

Species independent strength grading of structural timber

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Samenvatting

Hoewel hout als constructiemateriaal al millennia wordt toegepast, is het onderzoeksgebied naar de voorspelling van de sterkte van constructief hout nog in ontwikkeling. Op dit moment is de algemene opvatting dat het bepalen van de sterkte-eigenschappen van constructief hout per houtsoort moet plaatsvinden. Door deze sterkte-eigenschappen te koppelen aan kenmerken die aan het hout gemeten kunnen worden (hetzij visueel of machinaal) kan op sterkte gesorteerd hout aan de markt geleverd worden.

In potentie zijn er meer dan 1000 commercieel beschikbare houtsoorten waarvan het hout constructief gebruikt kan worden. Het grootste aantal hiervan zijn tropische hardhoutsoorten. Deze worden veelal toegepast wanneer een hoge sterkte en hoge duurzaamheid benodigd zijn. Tegenwoordig wordt in toenemende mate (tropisch) hout verkregen uit duurzaam beheerde bossen. Door deze manier van bosbeheer blijven de (tropische) bossen behouden en hebben ze een economische waarde voor de lokale bevolking. Een gevolg van deze manier van bosbeheer is dat er steeds meer onbekende houtsoorten in kleine hoeveelheden op de markt komen, waarvan de sterkte-eigenschappen moeten worden bepaald.

De huidige methoden voor de bepaling van de sterkte-eigenschappen van een houtsoort vereisen uitgebreide testen. Een probleem daarbij is dat het hout dat getest wordt representatief moet zijn voor het hout dat op de markt komt. Hierdoor moeten alle mogelijke variaties in de kwaliteit van het hout dat op de markt komt afgedekt worden.

Om het hout daarna in de praktijk toe te passen moeten sorteeregels worden opgesteld waaraan de sterkte-eigenschappen verkregen door testen zijn gekoppeld. Voor visuele sortering worden daarbij kenmerken als kwasten en draadverloop gebruikt. Bij machinale sortering worden bijvoorbeeld de dichtheid en de elasticiteitsmodulus gebruikt. Voor naaldhout is aangetoond dat machinale sterktesortering betrouwbaarder is en hogere opbrengsten in hogere sterkteklassen geeft dan wanneer visuele sterktesortering wordt toegepast. Voor tropisch loofhout is er het probleem dat het belangrijkste kenmerk voor de mechanische eigenschappen, het draadverloop, moeilijk te meten is in de praktijk. Daardoor is er slechts één visuele klasse voor tropisch hardhout gedefinieerd en is optimalisatie niet mogelijk.

Een oplossing voor bovengenoemde problemen kan houtsoortonafhankelijke sterktesortering zijn, waarbij alleen naar de invloed van de gemeten kenmerken wordt gekeken. Om dat te onderzoeken is in dit proefschrift de vraag gesteld wat de invloedrijke parameters zijn voor het ontwikkelen van houtsoortonafhankelijke sterktemodellen en of deze gekwantificeerd kunnen worden ten einde het veilig, economisch en duurzaam gebruik van naaldhout en hardhout in constructies te waarborgen.

Om deze vraag te beantwoorden is een database met een grote hoeveelheid testresultaten uit buigproeven onderzocht bestaande uit proefstukken van naaldhout, hardhout uit de

gematigde zones en tropisch hardhout. Deze database is de laatste 10 jaar in samenwerking met de Nederlandse industrie opgebouwd.

Op basis van literatuuronderzoek kan geconcludeerd worden dat de sterkte en stijfheid van foutvrij hout beide afhankelijk zijn van de dichtheid van het hout, onafhankelijk van de houtsoort. De natuurlijke spreiding voor de beide eigenschappen sterkte en stijfheid is onderling gecorreleerd, waardoor de stijfheid een goede voorspeller is van de sterkte van foutvrij hout. Op basis van de toegepaste mechanica zijn mathematische modellen opgesteld die de reductie van de sterkte en stijfheid als gevolg van de aanwezigheid van kwasten en draadverloop beschrijven.

Omdat de dichtheid de maximaal mogelijke basissterkte van het hout definieert is houtsoortonafhankelijke sterktesortering door middel van visuele sortering niet mogelijk. Bij visuele sortering is voor enkele naaldhoutsoorten en hardhout uit de gematigde zones de groeiringbreedte een maat voor de dichtheid: bij de meeste hardhoutsoorten is er geen significante correlatie.

Uit het onderzoek is naar voren gekomen dat het zeer moeilijk is om bij tropisch hardhout het draadverloop voor een destructieve buigtest goed in te schatten. Hierdoor kan de variatie in sterkte eigenschappen tussen partijen groot zijn. Om de sterkte van hout dat onder dezelfde handelsnaam op de markt gebracht wordt met voldoende veiligheid te bepalen, moet een reductiefactor worden toegepast op de testresultaten. Omdat niet bekend is hoe groot de variatie in draadverloop bij tropisch hout dat op de markt gebracht wordt is, is het niet mogelijk deze reductiefactor te bepalen.

Door middel van machinale sterktesortering is de variatie in draadverloop wel te detecteren. De reductie van de stijfheid (de elasticiteitsmodulus) is met dezelfde formule (de bekende Hankinson formule) te beschrijven als de reductie van de buigsterkte, alleen met andere waarden voor de constanten. Hierdoor zijn de elasticiteitsmodulus en de dichtheid samen de parameters die gebruikt kunnen worden voor machinale sterktesortering voor hout met draadverloop.

De reductieformule van de sterkte als gevolg van de aanwezigheid van kwasten heeft dezelfde vorm als die voor de reductie van de stijfheid door kwasten, met andere waarden voor de constanten. Hierdoor zijn elasticiteitsmodulus en de dichtheid samen ook geschikt voor houtsoortonafhankelijke machinale sterktesortering voor hout dat kwasten bevat.

Doordat de invloed van kwasten en draadverloop op de elasticiteitsmodulus niet te onderscheiden is, moet voor houtsoortonafhankelijke machinale sterktesortering het hout in twee groepen ingedeeld worden: hout waarbij kwasten het bezwijken veroorzaken en hout waarbij draadverloop het bezwijken veroorzaakt. Daarom is het nodig om naast de machinale metingen ook een visuele beoordeling uit te voeren, waarin voor de groep die met het draadverloopmodel gesorteerd wordt, gecontroleerd wordt dat er slechts kwasten met een gelimiteerde grootte aanwezig zijn. Verder dient door de visuele controle hout met andere kenmerken die niet door machinale metingen gedetecteerd kunnen worden zoals drukbreuk uit het sorteerproces verwijderd te worden. Een kenmerk als drukbreuk geeft een onvoorspelbare reductie van de sterkte en mag daarom in constructief hout niet aanwezig zijn.

De elasticiteitsmodulus kan in de praktijk op eenvoudige wijze door middel van trillingsmetingen bepaald worden.

Op basis van de mathematisch relaties tussen de kenmerken kwasten en draadverloop enerzijds en de dichtheid en elasticiteitsmodulus anderzijds is het mogelijk om voorspellingsmodellen op te stellen van de sterkte op basis van gemeten dichtheid en elasticiteitsmodulus. De waarden voor de buigsterkte, elasticiteitsmodulus en dichtheid voor de genormeerde sterkteklassen hebben betrekking op hout met een vochtgehalte van 12%. Om de testresultaten van hout dat met een ander vochtgehalte is beproefd naar dit referentievochtgehalte te kunnen omrekenen zijn correctiefactoren afgeleid. Voor constructieve afmetingen is geen correctie met betrekking tot de afmeting naar de referentieafmetingen noodzakelijk.

De vorm van de spreiding rond de voorspellingslijnen is theoretisch afgeleid op basis van de verdeling van de voorspellingswaarden. De vorm van de spreiding blijkt verschillend te zijn voor het voorspellingsmodel van hout met kwasten en voor het voorspellingsmodel van hout met draadverloop. Een methode om de vorm van de spreiding af te leiden op basis van de experimenteel verkregen data is opgesteld en geverifieerd.

Om het hout daadwerkelijk te sorteren moeten ‘settings’ worden bepaald. Dit zijn de limietwaarden in het voorspellingsmodel op basis waarvan het hout in een bepaalde sterkteklasse wordt ingedeeld. De sterkte waarden van hout kunnen alleen geverifieerd worden op basis van de eigenschappen van een partij die destructief getest is. Hierbij kan bij kleine aantallen in een sorteerklassen de karakteristieke sterkte waarde sterk variëren tussen geteste partijen. Om dit te ondervangen is een methode ontwikkeld waarmee op basis van de verdelingen van de voorspelde waarden en de spreiding in het voorspellingsmodel de karakteristieke waarde voor de sterkte kan worden bepaald. De karakteristieke waarde voor de sterkte van een sorteerklassen bij een geëiste waarschijnlijkheid kan hiermee onafhankelijk van het aantal proefstukken in een partij bepaald worden.

Met de ontwikkelde voorspellingsmodellen is het mogelijk om houtsoortonafhankelijk machinaal op sterkte te sorteren. Met name voor tropisch hardhout kan hiermee de afgegeven sterkteklasse op een betrouwbare manier worden bepaald en kan de opbrengst in de hogere sterkteklassen worden vergroot. Het resultaat draagt bij aan een economische, veilige en duurzame constructieve toepassing van hout.

Summary

Timber as a construction material has been used for millennia, but the research field covering the prediction of the strength of structural timber is still in development. Currently, the common conception is that the determination of strength properties has to be determined for every wood species individually. By combining these strength properties to features that can be measured at the timber (either visually or by machine measurements), strength graded timber can be supplied to the market.

Potentially, there are more than 1000 commercially available wood species, the timber of which can be used in structures. The largest amount of these wood species are tropical hardwoods. These wood species are often used when high strength and high durability are required. Nowadays, (tropical) timber is increasingly coming from sustainably managed forests. By application of this method of forest management, the (tropical) forests are preserved and have an economic value for the local population. A result of this approach is that more and more unknown wood species in small quantities are coming on the market, the strength properties of which have to be determined.

The present methods for the determination of strength properties of a wood species require extensive testing. An extra problem is that the timber that is tested has to be representative for the timber coming on the market. All future variations in the quality of the timber coming on the market have to be covered.

To be able to use the timber in structures, grading rules have to be formulated that are related to the strength properties, determined by tests. For visual grading, features like knots and slope of grain are used. For machine grading, for example, the density and modulus of elasticity are used. For softwoods it has been proven that machine grading is more accurate and gives higher yields in the higher strength classes in comparison with visual grading. For tropical hardwoods, a major problem for visual grading is that the most important feature for the mechanical properties, the slope of grain, is very difficult to measure in practice. For this reason, only one visual grade is defined for tropical hardwoods and optimisation is not possible.

A solution for abovementioned problems can be species independent strength grading, where only the influence of the measured features is taken into account, irrespective of the species. To investigate whether this would be possible, the research question dealt with in this thesis was: what are the influencing parameters for the development of species independent strength models, and can they be quantified to ensure safe, economic and sustainable use of softwoods and (tropical) hardwoods in structures ?

To answer this question, a database consisting of a large number of test results from bending tests on European softwoods, temperate hardwoods and tropical hardwoods was investigated. This database was built-up in the last ten years in cooperation with the Dutch industry.

Based on a literature survey, it was concluded that both the strength and stiffness of clear wood depend on the density of the timber, irrespective of the wood species. The natural variation in test values for both properties strength and stiffness are correlated. As a result, the stiffness is a good predictor of the strength for clear wood. Based on structural mechanics, mathematical models were formulated describing the reduction of strength and stiffness caused by the presence of knots and grain angle deviation.

Because the density defines the maximum possible basic strength of the timber species, independent strength grading by visual grading is not possible. For some softwood species and temperate hardwoods, the grow ring width can be a measure for the density. For the majority of hardwood timber, there is no significant correlation.

The examination of the visual measurement of the slope of grain has revealed that it is very difficult to accurately estimate the slope of grain for tropical hardwoods before a destructive bending test. As a consequence, the variation in strength properties between test samples from the same wood species can be very large. To determine the strength of timber brought on the market under the same trade name with sufficient safety, a reduction factor has to be applied to the test results. Because it is not known how large the variation can be in the slope of grain for tropical timber brought on the market under the various trade names, it is not possible to determine this reduction factor.

By means of machine strength grading it is possible to detect the variation in slope of grain. The reduction of the stiffness (the modulus of elasticity) can be described with the same equation (the well-known Hankinson equation) as the reduction of the bending strength, only with other constant values. Because of this, the modulus of elasticity and the density are parameters that, together, can be used for machine strength grading for timber showing grain angle deviation.

The reduction equation describing the strength due to the presence of knots has the same form as the reduction equation describing the stiffness due to the presence of knots, only with other constant values. Because of this, the modulus of elasticity and the density together are also the parameters suited for species independent machine strength grading of timber containing knots.

Because the influence of knots and slope of grain on the modulus of elasticity cannot be distinguished from each other in the modulus of elasticity measurement, timber has to be divided into two groups for species independent machine strength grading: timber for which failure is induced by knots and timber for which failure is induced by slope of grain. Therefore, it is necessary to perform a visual assessment, to check for the group containing grain angle deviation that only knots of limited sizes are present in the timber. Furthermore, the visual check has to ensure removal of pieces with features that cannot be detected by machine readings, such as compression failures. A feature like a compression failure causes an unpredictable strength reduction and is therefore not allowed in structural timber.

In practice, the modulus of elasticity can be determined in a simple manner by means of vibration measurements.

On the basis of mathematical relationships between on the one hand the features knots and slope of grain and on the other hand the density and the modulus of elasticity, it is

possible to formulate prediction models of the strength based on the measured density and modulus of elasticity of a piece. The values for the bending strength, the modulus of elasticity and the density for the standardized strength classes are related to timber with a moisture content of 12%. Correction factors have been derived to be able to adjust the test result of timber tested with a different moisture content to this reference moisture content, For structural sizes, no adjustments with regard to the reference sizes are necessary.

The shape of the scatter around the prediction lines is theoretically derived on the basis of the distribution of the prediction values. The shape of the scatter turns out to be different for the prediction model for timber containing knots and for the prediction model for timber containing grain angle deviation. A method to derive the shape of the scatter on the basis of available data has been formulated and verified.

To actually grade timber, "settings" have to be determined. These are limit values for the prediction values that determine which strength class the timber can be assigned to. The strength values of timber can only be verified on the basis of the properties of a sample that is tested destructively. For small numbers of pieces in a sample, the characteristic values of a strength grade can vary significantly between tested samples. To overcome this problem, a method was developed which takes into account the distribution of the prediction values and the scatter of the prediction model. The characteristic strength value of a strength grade for the required probability can be determined by it, irrespective of the number of pieces in a sample.

With the developed prediction models it is possible to perform species independent strength grading. Especially for tropical hardwoods, the assigned strength classes can be determined in a reliable way and the yield in the higher strength classes can be increased. The research results contribute to an economic, safe and sustainable application of timber in structural applications.

List of symbols

Greek letters

α	angle between the beam axis and grain direction, or the confidence level.
β	interaction factor for the shear strength in the Norris equation, or the reliability index, or the shrinkage coefficient (% per percent change of moisture content)
μ	mean value of a population
ρ	density (kg/m ³)
σ	stress (N/mm ²), or standard deviation of a population

Latin letters

a	distance between the support and point load in a four point bending tests (mm)
b	dimension perpendicular to the plane of the load of a specimen in in a four point bending tests (mm)
f	strength (N/mm ²)
f_m	bending strength (N/mm ²)
$f_{m,stat,mod}$	predicted values of the bending strength according to the mean regression line (N/mm ²)
f_t	tension strength (N/mm ²)
f_c	compression strength (N/mm ²)
f_v	shear strength (N/mm ²)
E	modulus of Elasticity (N/mm ²)
F	Force (N)
G	shear modulus (N/mm ²)
GKR	Group knot ratio
h	dimension in the plane of the load of a specimen in a four point bending tests (mm)
I	second moment of Inertia (mm ⁴)
IP_{fm}	Indicating Property, prediction value for the bending strength based on an equation with measured parameters (N/mm ²)
k_f	Ratio between the bending strength parallel and perpendicular to the grain

k_m	Ratio between the MOE parallel and perpendicular to the grain
$k_{s,tn}$	reduction factor describing the influence on the scatter between samples based on the number of samples for visual grading
l	span in a four-point bending test (mm)
l_b	length of a timber beam (mm)
m.c.	moisture content in %, the percentage of the weight of the water in a wooden piece, related to the weight of the wood with no water inside
MOE_{dyn}	Modulus of Elasticity determined from vibration measurements (N/mm ²)
MOE_{glob}	Modulus of Elasticity determined in a four point bending test in which shear deformation is incorporated (N/mm ²)
MOE_{loc}	Modulus of Elasticity determined in a four point bending test under pure bending (N/mm ²)
N	Number of samples
OLS	Ordinary least squares regression analysis
WLS	Weigthed least squares regression analysis.
n	Number of pieces in a sample
$p(i)$	factor indicating the probability for a prediction value for the bending strength that the actual bending strength is lower than a required value.
p_{char}	the probability that the actual bending strength of pieces in a sample graded between a lower and higher IP_{fm} -value is lower than the required value
PTL	Parametric Tolerance Level. The value for which, with a probability of α (the confidence level) the $p\%$ fractile of the underlying population is higher than this value.
r	correlation coefficient, in other books also denoted as ρ
r^2	coefficient of determination
S	Setting. Limit value for the IP_{fm} to grade timber in a strength class. When timber is graded to more than one strength class there will be more than one value of S , these are then called settings.
SKR	Single Knot Ratio
SoG	Slope of Grain, the tangent of α
s	standard deviation of a sample
t	thickness, smaller dimensions of a piece (mm)
w	ratio of the standard deviation of the residuals from a regression analysis and the prediction model values
$\bar{x}$	mean value of a sample
W	section modulus (mm ³)

Definitions

clear wood	pieces of wood with no strength reducing characteristics
structural timber	pieces of wood intended to be used in structures
microscopic level	level of wood where cells can be distinguished with a microscope
macroscopic level	level of wood where it is possible to retrieve clear wood pieces
gross level	level of wood where gross features as grain angle deviation and knots occur and can be distinguished with the naked eye.
visual grading	the process by which a piece of timber can be sorted, by means of visual inspection, into a grade to which characteristic values of strength, stiffness and density may be allocated
machine grading	the process by which a piece of timber can be sorted by a machine sensing, non-destructively, one or more properties of the timber, with any necessary visual inspection, into grades to which characteristic values of strength, stiffness and density may be allocated
settings	limit values for the prediction values of the bending strength, to grade structural timber in different strength classes.
tree species	trees sharing the same morphologic characteristics as leaves etc.
wood species	wood originating from a certain tree species
trade name	commercial name under which structural timber, coming from one or more wood species, is brought on the market
tropical hardwoods	wood of angiosperm trees of the botanical group dicotyledons whose natural distribution lies substantially south of the Tropic of Cancer and north of the Tropic of Capricorn
temperate hardwoods	wood of angiosperm trees of the botanical group dicotyledons whose natural distribution lies substantially north of the Tropic of Cancer and south of the Tropic of Capricorn
softwoods	wood of gymnosperms trees of the botanical group coniferales.

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Introduction

1.1 Sustainably produced (tropical) timber for structural applications

Timber has a long history as a construction material for structures made by humans. Throughout the centuries timber has proven to be an important building material. In this time era, there is an increasing interest for the use of timber as a building material. There are several reasons for this. On one hand, in the developments of engineered wood products and connections, there is a great variety in appearances, which addresses aesthetic demands. On the other hand, the use of timber plays an important role in the reduction of CO₂ emission in the building process. In a comparative study Gustavsson (2006) showed, that a multi-storey timber building could be CO₂-negative, in contrast to a concrete building.

A prerequisite for sustainable timber buildings is that the timber is produced in a sustainable way. This means that we want the total worldwide forest area to remain at least constant and the biodiversity of species to be maintained. When this is discussed, the problem of deforestation has to be addressed. Pictures of the tropical rainforest, with large areas where all trees are felled come to mind. The irresponsible use of the tropical forests has certainly led to the present situation where large areas of tropical rainforests are gone.

However, the situation today has changed in that sense that it is now possible to retrieve timber from rainforests managed in a sustainable way. These forests are certified by independent organisations like FSC (www.fsc.org) or PEFC (www.pefc.org). For a forest to be certified, it must be managed in such a way that it is maintained in a sustainable way. This means that only selective felling takes place; only trees of a certain diameter may be felled in a certain forest section, after which this section must be left alone for 30 years. Since tropical forests have a large diversity in tree species this means that the yearly production might include a large number of tree species.

Apart from this ecological aspect, there are also social requirements to secure the living conditions of the local population. Nowadays, most deforestation is not caused by timber production, but for instance by gaining agricultural areas (Lambrechts et al., 2009)

To ensure that timber used for constructions is from a sustainably managed source, the buyer of tropical hardwoods can demand that timber is delivered with a recognized certificate. The Dutch government has issued guidelines for sustainable purchasing of timber (www.tpac.smk.nl) in which recognized certificates are listed. These guidelines are

to be followed for all public contracts. This does not only concern tropical hardwoods, but also softwood from Europe and North-America.

The Netherlands Timber Trade Organisation (VVDH) has also committed itself to increasing the share of certified sustainably produced timber brought on the market. The report on the first half year of 2013 (Winterink, 2013) shows that for softwoods 96% of the imported timber and for hardwoods 54% of the imported timber is from a demonstrably sustainably managed forest with a Chain of Custody certificate. When these figures are compared with the situation in 2008 (softwoods 77% and hardwoods 19%), the impact of this policy is clear.

It can be concluded that the use of (tropical) timber from sustainably managed forests can have a positive impact on reducing CO₂-emission (Gustavsson, 2006) as well as on maintaining the forests with an economic benefit (www.fsc.org and www.pefc.org). To promote the use of sustainable tropical timber, the European Sustainable Tropical Timber Coalition (www.europeansttc.com) was founded, with stakeholders in governments and building companies.

A consequence of making use of sustainably managed forests is that a large number of tropical hardwood species become available on the market. These are generally called Lesser-Known Timber Species (LKTS), for which the properties have to be determined to be able to use them for structural purposes (Van Benthem en Bakker, 2011). Potentially, more than 1000 timber species are suitable for structural applications (Wagenfuhr, 2007). According to current regulations, it is required to determine the strength properties of each wood species separately. It would require an enormous amount of testing to determine the strength properties of all these species. This is a huge drawback in the economic use of these species. Another problem is that currently for tropical hardwoods only strength properties connected with visual assessments are available. This is not a very effective method, as will be shown in this thesis. The main objective of this thesis is therefore to investigate the development of species independent strength models based on objectively (mechanically) measured parameters.

In section 1.2 the backgrounds of the safe design of timber structures will be briefly explained and the consequences for the determination of the strength properties of (tropical) hardwood timber.

1.2 Grading of softwood and hardwood timber

The safety of a timber structure depends on a number of aspects, such as the correct mechanical modelling of the loads, a good prediction of the structural behaviour of the design of the connections, and good workmanship during the execution. This dissertation is restricted to the accurate and economic determination of the strength and stiffness properties and the density of timber to be used in structures.

The concept of reliability of structures according to NEN-EN-1990 can be described with the following formula:

$$P_f = \text{Prob}(R - E) \leq 0 \quad (1.1)$$

In equation (1.1) R (Resistance) and E (the load effect) are stochastic variables.

Equation (1.1) describes the probability that the stresses in a structural element due to the loads (E) are greater than the resistance strength (R) of the material the structural element is made of. The value of P_f is often represented as its transformation to the cumulative distribution function of the standardized normal distribution value, called the reliability index β .

The solution of equation (1.1) is a 3D problem that can be solved with a probabilistic approach. However, in the normal engineering practice according to the Eurocodes, the probability of failure is not calculated with a full probabilistic approach, but with a semi-probabilistic approach using characteristic values (E_k and R_k) together with a load and material factor. The engineer has to verify that E_k multiplied by a load factor does not exceed R_k divided by a material factor. These factors are calibrated to ensure that the required reliability index value is reached for different load situations.

Figure 1.1 shows the relationship between the characteristic strength value and the design strength value, where the design strength value is the characteristic value divided by the material factor. α_R is a FORM (First Order Reliability Method) sensitivity factor.

From Figure 1.1 it is clear that when the material factor is a fixed value, and R_k is a fixed percentage fractile of the distribution, the variability in timber strength properties influences the reliability of the structure. Two different strength distributions can have the same characteristic value, but different mean and standard deviations. To justify a chosen material factor, the scatter in standard deviations of the distribution of different batches of timber must be limited. This can be achieved by grading the timber.

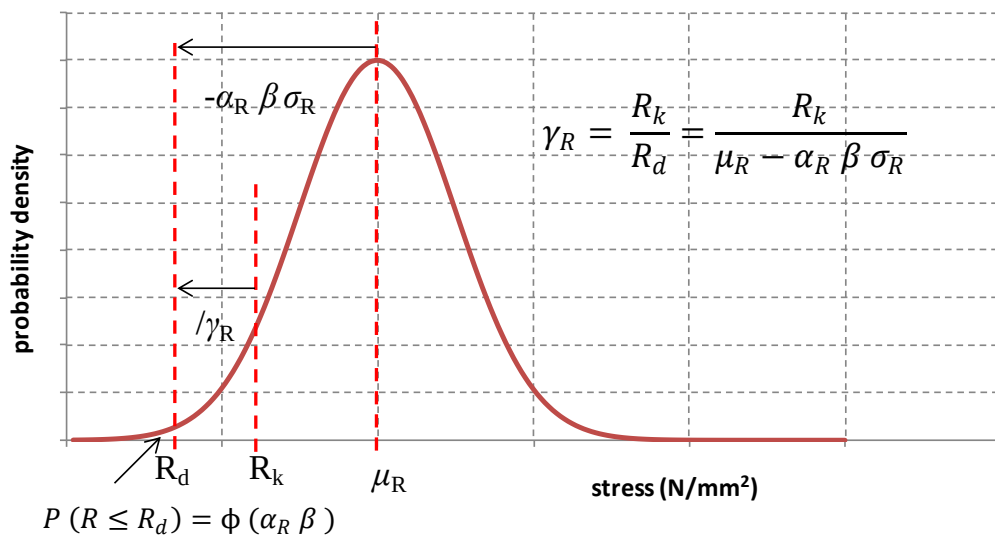


Figure 1.1. Relationship between the characteristic strength value and the design strength. Re-sketched after Vrouwenvelder (2008).

A typical aspect of timber is that the material properties can have different values depending on indirect factors such as size, moisture content and duration of the loads. These factors depend on the design and climate conditions the structure is placed in. In structural design calculations, these factors are integrated in the design equations (Eurocode 5 for timber structures) that an engineer has to apply. It is therefore very important that these indirect factors are correctly derived from material tests.

This dissertation deals with the derivation of material properties and the way they are influenced by the indirect effects of size and moisture content, and the way they are influenced by directly measurable non-destructive properties. As explained in figure 1.1 the value of R_k (which is called the characteristic value) is used by an engineer as input in calculations. In NEN-EN 1990 this value is defined as the 5% fractile of the strength distribution of a material. It is this value that has to be determined in material tests on structural timber when strength is concerned. However, if timber would be used without any selection then - due to its large natural variation in strength properties - the material factor would have to be very high to ensure the required safety. Or another option would be to change the percentile of the characteristic value. To overcome this problem, the timber is selected in groups by a process called grading.

The process of grading can be defined as the sorting of timber beams into groups to which the same strength properties can be assigned. This sorting takes place on the basis of parameters (which we will call grading parameters) that have an effect on the strength properties. By defining different levels of parameter values, individual beams can be assigned to various grades that have different strength properties. There are two grading methods: visual grading and machine grading.

Visual grading takes into account visible strength reducing parameters, such as the size and amount of knots, or the slope of grain. Machine grading makes use of parameters produced by machine readings like the weight (by which the density can be calculated) or by readings (through vibration or bending) by which the modulus of elasticity can be calculated. The grading method and the parameters have an influence on the yield of the grading process, which is defined as the amount of timber that can be assigned to the different grades. This is because the predictability of the parameters on the strength properties differs. In general, the parameters used by machine grading have better prediction capabilities than the parameters used in visual grading. To be able to perform the grading process, prediction models have to be derived. The grades are mostly related to a predefined strength class. Predefined strength classes with its properties as listed in European standard EN 338.

The effect of the grading process of a batch of timber is illustrated by figure 1.2. The strength distribution of the total ungraded population is the outside line. During grading, the beams are assigned to three grades (a), (b) and (c). The 5% fractiles are indicated with vertical dashed lines. The grading has two effects: the 5% fractiles of grades (b) and (c)

are higher than the 5% fractile of the ungraded population and the variability in strength properties of the three grades is much lower than that of the ungraded material. This results in a more economic use of the timber.

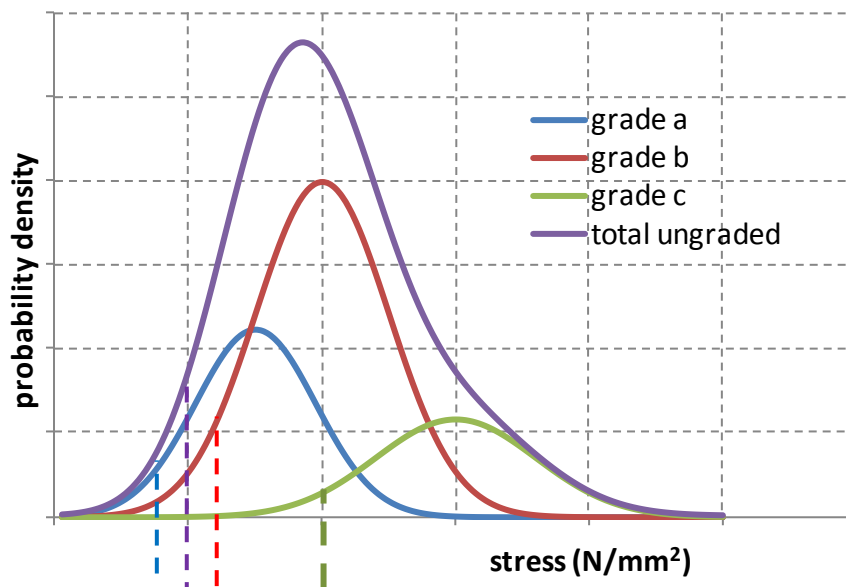


Figure 1.2. The effect of strength grading (re-sketched after STEP 1,1995).

Defining the characteristic value as the 5% fractile of a distribution is not as simple as it seems. To be able to do this, first the distribution type (parametric: normal, lognormal, Weibull or non-parametric: ranking) has to be determined. The selected distribution type will affect the grading result. It seems logical to determine the distribution type for every new dataset that fits best. However, in standards often a distribution type for a certain mechanical property is prescribed.

Timber is a natural material and is produced in nature as trees. By felling these trees and sawing them in dimensions for structural use they become timber. There are more than 100.000 wood species (Hajela, 2008), but when the amount is restricted to species suitable for use in structures and from which timber can be economically produced a number of 1000 species can be assumed (Wagenfuhr, 2007). Looking at the anatomy (the way the wood cells are structured) a division can be made between softwood and hardwood species. Botanically, they can be distinguished by the presence of needles (softwoods) or leaves (hardwoods). The largest amount of the 1000 potential species for structures are hardwood species. Tropical hardwood species are used when high strength and high durability is required. Tropical hardwood can be defined as wood of angiosperm trees of the botanical group dicotyledons whose natural distribution lies substantially south of the Tropic of Cancer and north of the Tropic of Capricorn. Examples of these structures are lock gate doors or timber guard rails. For buildings, usually softwood is used. See figure 1.3.



Figure 1.3. An industrial building from glued laminated softwood timber (left) and lock gate doors from tropical hardwood timber (right)

It would be practical if for all beams of every wood species the same grading parameters and the same parameter levels could be used. Unfortunately, for visual grading this is not possible. This is illustrated in figures 1.4 and 1.5. In softwood, the presence of knots is mostly the failure initiating parameter. In tropical hardwoods, the presence of knots is very rare and the slope of grain is the most critical parameter. The same knot indicator gives different strength levels for graded and destructively tested samples for different species. Therefore, the strength properties of the same visual grade can be different for different species. Only limited combinations of species for which the strength properties are the same for a visual grade are used. In North-America, the softwood species spruce, pine and fir are combined, mostly for practical reasons, where it is accepted that the species with the lowest strength properties are governing in the strength properties assignment.



Figure 1.4. Spruce tree (picea abies) (left) and typical failure mechanism for a spruce timber beam (right)



Figure 1.5. Cumaru tree (dypterix spp.) (left) and typical failure mechanism for a cumaru timber beam (right)

For machine grading, the same argument is used only for combining spruce and pine or spruce and fir. As a consequence, for every wood species used in structures, visual and machine grades are connected to strength classes (EN 1912 and EN 14081-4). Unless specifically mentioned, it is not allowed to mix species in the grading process.

This means that for deriving the strength properties, each wood species is basically regarded as a different material for which the strength properties have to be derived by an extensive testing program. A factor that not has been discussed is the representability of the tested samples for the whole population. This is done by defining the strength properties not only to the species, but also to the areas where they grow. All these aspects particularly hinder the introduction of a large number of “lesser known” species that are the result of the felling process in sustainably managed forests.

Another consequence of the focus on softwood research is that the indirect factors size, moisture content and duration of load are determined by research on softwoods. In the calculation rules of timber structures NEN-EN 1995-1-1 (2005), which is commonly referred to as Eurocode 5 (EC5), these factors are considered to have the same influence on softwoods as hardwoods, which is not based on research.

1.3 Research question

The current methods for the determination of the strength properties for wood species require extensive testing. To make sure that the strength properties are comparable, these values must be adjusted to a reference size, moisture content and load duration, which are the (safely applied) reciprocals of the calculation factors in EC5. These adjustment factors are also a result from research mainly on softwood and it is unclear if these factors are correct for hardwoods. For visual grading the problem with tropical hardwoods is that the main strength influencing parameter, the grain angle deviation, is very difficult to measure. As a result, with visual grading only one visual grade and connected strength class can be defined for a species. At present, there is no hardwood species accepted to be used in machine grading under the current European standards. Ravenshorst et al. (2004)

suggested that a species independent strength grading approach might be a solution to apply machine strength grading for tropical hardwoods. Considering the situation described in the previous sections, the main question in this dissertation can be formulated as follows:

What are the influencing parameters for species independent grading models and how can they be quantified to ensure safe, sustainable and economic use of softwood and hardwood timber in structures?

The originality in the work described in this dissertation lies in the approach to the described problems. The approach to combine strength predicting properties independent of the species with the intention of using them in machine grading has not been successfully applied before. The originality lies in the attempt to predict the strength of structural timber based on physical properties independent of the species, taking into account the mechanics behind the occurring failure modes. To investigate the possibilities of this approach a synthesis of data of softwood and hardwood species will be made. By combining softwoods and hardwoods the range of the strength properties is much larger, which could also add knowledge for the accuracy of calculation rules for timber structures.

1.4 Dissertation outline

In figure 1.6 the outline of the dissertation is illustrated.

In chapter 2, the aspects of using wood as a structural material are explained and an overview is given of the historical development in the assignment of strength properties to structural timber. In chapter 3, the dataset is presented on which the modelling in this thesis is based. This is a unique dataset containing 20 tropical hardwood species. This data was collected in the last 15 years in the Netherlands in cooperation with the industry. Besides the dataset of tropical hardwoods there are datasets of temperate hardwoods and European softwoods, which will be used for comparison. Another part of chapter 3 gives the description of the test methods to determine the strength properties according to the current European standards, and the methods to measure the strength predicting parameters in the grading process (of either visual or machine grading). The statistical methods are further elaborated to be used for species independent strength grading in chapter 6.

In chapter 4, the basic test results and relationships between measurable characteristics and laboratory tests are presented and analysed.

In chapter 5, the theory is developed to formulate species independent strength models. The failure mechanisms of clear wood and of timber with gross features as knots and grain angle deviation are studied. The influence of the gross features on the strength and stiffness of timber are described by physical models. Adjustment factors for size and

moisture content are determined and the developed models are verified on a dataset of softwood timber and on a dataset of tropical hardwood timber. In chapter 6, the developed models from chapter 5 are applied on the datasets listed in chapter 3 to perform species independent grading. In chapter 7 the result of the research is discussed and conclusions are drawn.

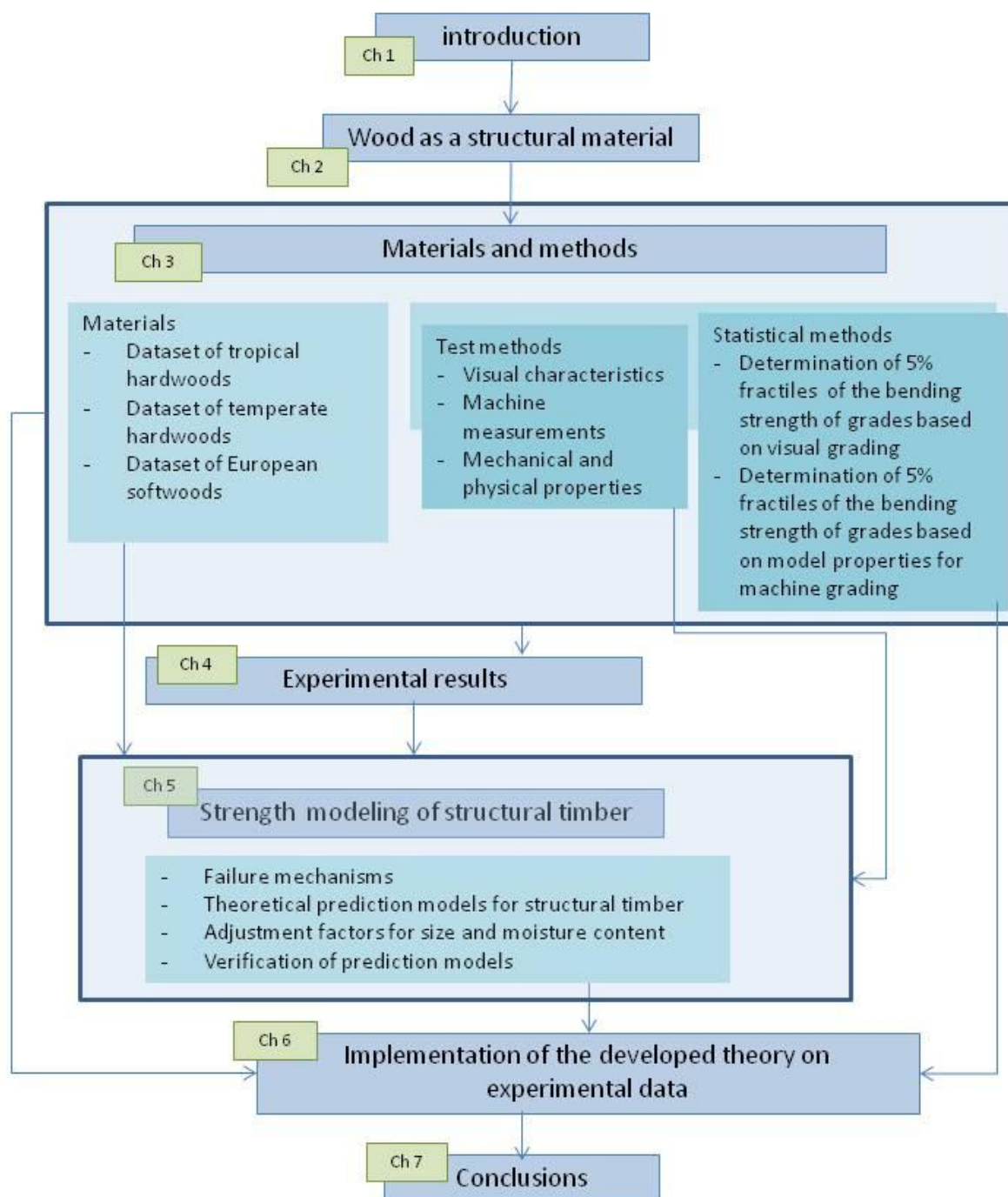


Figure 1.6. Outline of this dissertation.

Wood as a construction material

2.1 The source and structure of wood

2.1.1 The tree

Wood is converted from trees. Biologically, trees are a specific form of plants, with a special aspect: their stem. From this stem, the product that we call timber is converted. However, in practice the difference in the use of the terms wood and timber is not always clear. In this thesis, wood is defined as the basic material from the stem of the tree. The products after it is processed in sizes fit for structural use and strength properties are assigned to it are defined as structural timber. In this thesis structural timber is always rectangular sawn. In figure 2.1 the main parts of a tree and their functions are shown.

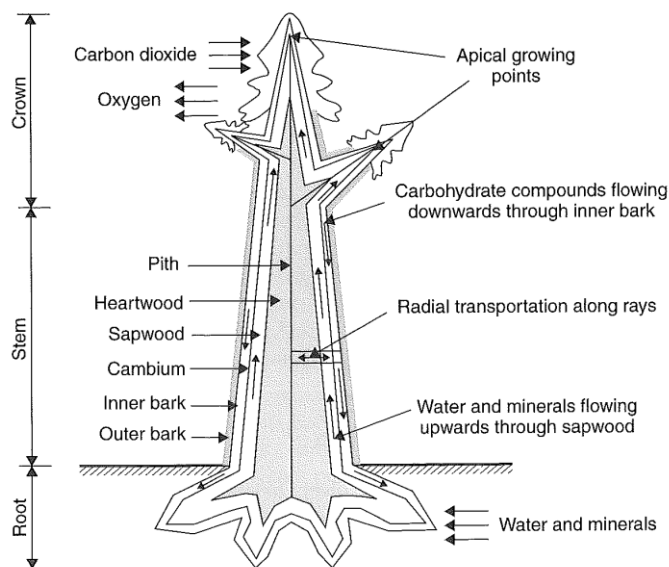


Figure 2.1. Main parts of a tree and their functions (from Smith et al, 2003)

The stem provides the mechanical resistance of the tree for the self-weight and environmental loads, and therefore makes that the timber from it is designed by nature to carry loads. For commercial timber, the source species can be divided in two main groups, the Coniferales (gymnosperms) and Dicotyledons (angiosperms). What in practice is called softwood timber belongs to coniferous trees and what is called hardwood timber

belongs to dicotyledonous trees. For commercial timber, a division can be made in hardwood from the temperate zones and from tropical zones. In appearance, the softwood trees have needles and the hardwoods have leaves.

In the classification system of plants, all tree species are named by their scientific name. This classification system is based on morphological features of the tree (such as fruits, leaves and flowers). This means that the determination of the tree species can only be done by investigation of the tree. To determine the tree species by examination of a wood sample is very difficult (Dinwoodie, 2000). The diversity of tree species is the result of an evolutionary process, whereby angiosperms have evolved from softwoods and have a more complex cellular structure. The classification of plants knows a hierarchy such as kingdom, order, family, genus and species. Wood anatomical features are not always distinctive enough to determine these different levels. Timber is traded under commercial names (trade name), and it normally consists of one or more tree species. In some cases the tree species can be determined from wood anatomical features, but in many cases of hardwood only on genus or family level a determination of the timber is possible. According to EN 14081-1 the tree species has to be given in the trade documents, but this is, especially for (tropical) hardwoods, an unrealistic requirement.

2.1.2 The structure and strength of wood

There are different scale levels at which timber can be examined to explain the properties at product level intended for structural use. In figure 2.2, the levels at timber which can be classified and analysed are shown.

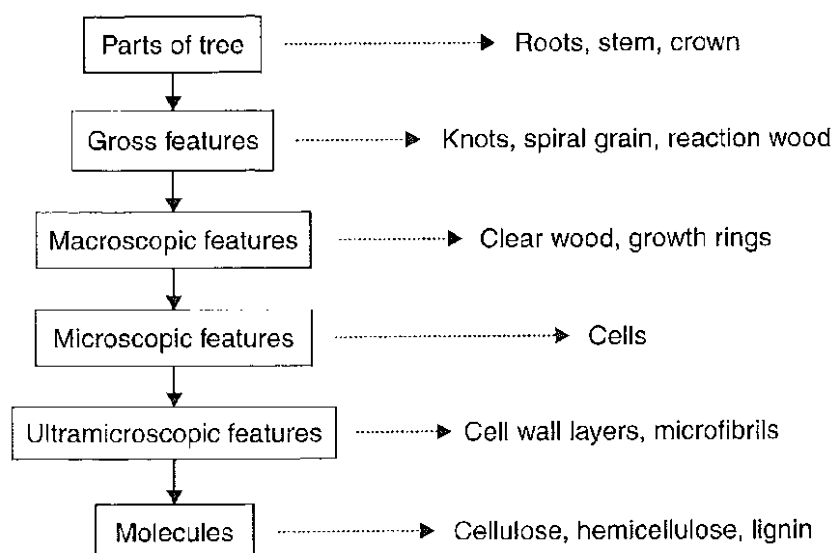


Figure 2.2. Levels at which wood structure can be classified and analysed (from Smith et al., 2003).

In figure 2.3 the microscopic, macroscopic and gross levels are shown for the species massaranduba (*Manilkara bidentata*)

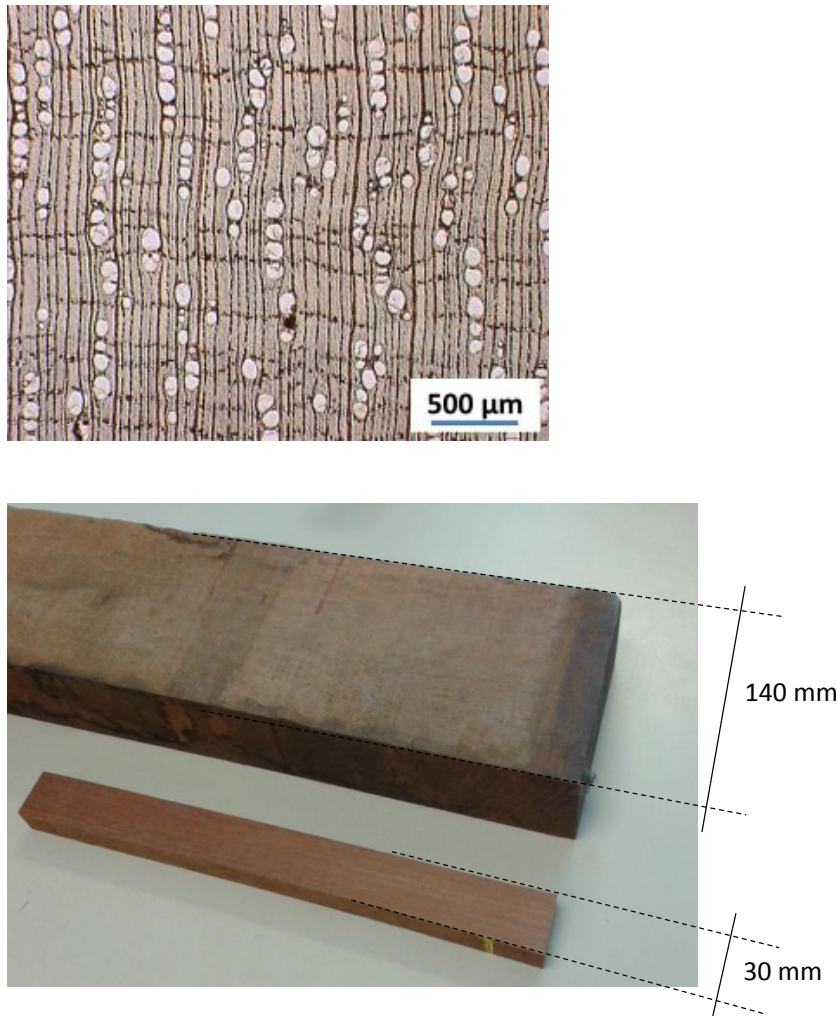


Figure 2.3. *Massaranduba* (*Manilkara bidentata*) at microscopic level (above, cross section), at macroscopic level (below, small clear piece) and gross level (below, large piece)

In this thesis, the main purpose is to predict the strength properties at the level of gross features for structural timber based on measurements made at this level. Measurements at the macroscopic level can be useful to understand and predict the influence of gross features and will be studied in this thesis. The level of microscopic features, ultramicroscopic features and molecules is part of the specialism of wood material science. In this section, a brief description at the cell level will be given to evaluate if they can affect the mechanical properties at timber level (gross features). This does not mean that at ultramicroscopic level no influence is expected. In many literature (for instance Dinwoodie (2000) and Thelandersson et al. (2003)) the direction of the microfibrils in the S2-layer of the cell wall is believed to have an important influence on the strength properties of the cell wall, but this property cannot be distinguished from the influence of gross features on strength measurement of the product timber. Their influence will be

implicitly incorporated in measurements at gross level or could be the cause of the natural variation timber has.

At microscopic level, a tree trunk is composed of millions of individual woody cells, that are organized in recognizable patterns varying with the species (Kollmann and Côté, 1968). In gymnosperms there are two basic types of cells and in angiosperms four types of cells. They have to perform the following functions: storage of organic substances, conduction upwards of dilute mineral solutions, and support of the crown. Storage is performed by parenchyma cells in both softwood and hardwood, tracheids perform the support and conduction function in both softwoods and hardwoods. In hardwoods, there are additionally also fibers for support and vessels (pores) for conduction.

Most cells are many times longer than broad. The long cells, which are arranged longitudinally (in the direction of the stem), make up the bulk of the wood and provide 'grain' to the material.

The wood cells with the conduction function are positioned near the bark of the stem and are called sapwood. The cells in the centre of the stem are called heartwood. The durability of sapwood and heartwood is very different, but for the mechanical properties this makes no difference, assuming that they have the same moisture content.

Although there are no fibers in softwoods, there are general terms for both softwoods and hardwoods that use the word fiber, for instance the term fiber saturation point, which indicates the moisture content at which the cell walls are saturated. Water in wood is first absorbed by the cell walls and therefore affects the mechanical behaviour. When the fiber saturation point is reached, the moisture content can increase, but the water will then be in the lumen of the cell and is called free water, with no influence on the mechanical properties.

The strength and stiffness of the cells is determined by the cell wall thickness. The density of the cell walls seems to have a rather constant value of 1500 kg/m^3 (Dinwoodie, 2000). Therefore, this is also the maximum possible density at gross level of the timber if the cell walls would be that thick that no inner opening would be left. Then there would be no room for conduction. Therefore, species with these densities do not exist. Apart from the cell itself, also the longitudinal connection between the cells might be a governing factor for the strength and stiffness of the system of cells.

The growing pattern of the cells (circularly grown, many times longer than broad with the bulk of the material in longitudinal direction) causes wood to be anisotropic, or more specific orthotropic with different mechanical properties in 3 directions. See figure 2.4

In figure 2.4 (left), a part of a tree stem is cut and the surfaces of the 3 main directions are indicated:

- X is the longitudinal direction parallel to the stem axis
- R is the radial direction perpendicular to the stem axis from the pith to the bark
- T is the tangential direction perpendicular to the stem axis and perpendicular to the radial direction.

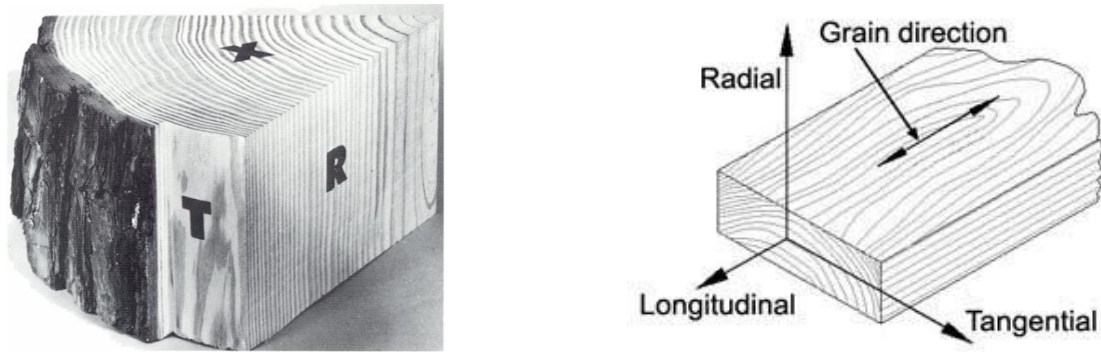


Figure 2.4. Main directions of wood in a tree (left) and in a wooden board (right)

From a stem from figure 2.4 (left), a piece of wood can be cut according to figure 2.4 (right). At macroscopic level this is called clear wood. At microscopic level wood is certainly not homogeneous, but at macroscopic level this could be assumed. For clear wood, the grain direction (the direction of the tracheids, vessels and fibers) is exactly parallel to the longitudinal axis of the board. And for clear wood there are no gross features such as knots present.

The ratio between the strength parallel and perpendicular to the grain can be a factor thirty (Kollmann and Coté, 1968). From figure 2.4, the strength ratios of clear wood in different directions become clear when wood is modelled as a bundle of straws. Parallel to the grain in longitudinal direction, the tension strength depends on the strength of tracheids and fibers. In longitudinal compression strength depends on the stability (for buckling) of the tracheids and fibers. In the direction perpendicular to the fibers, the tension strength is governed by the strength of the cell walls or the connection between the cells in transverse direction. For compression perpendicular to the grain the strength is governed by the deformation of the cells and is therefore also a stability failure. For shear along the grain, the strength of the connection between the tracheids and fibers is governing. Based on these analogies, brittle behaviour is expected under tension and shear and a more or less plastic behaviour under compression.

For the strength properties of small size clear wood several databases worldwide are available. Clear wood is mostly tested in small sizes (cross section 20 mm x 20 mm, span 360 mm in a four-point bending test, or 50 mm x 50 mm, span 700 mm in a third-point bending test) because it is difficult to obtain it in larger sizes. When it is assumed that the amount of cell wall material determines the strength and stiffness of clear wood, then this will appear in the relationship of the density with these properties. For clear wood there are several studies and databases showing that the bending strength and MOE of clear wood can well be explained with the density. Armstrong et al. (1984) defined formulas for the bending strength and the modulus of elasticity based on the density (or specific gravity) with different constants for softwoods and hardwoods.

In figures 2.5, 2.6 and 2.7, the values as listed in the Houtvademecum (Wiselius, 2010), are presented. These values are based on a literature search of tests on small clear specimens worldwide of softwoods and hardwoods. The values presented here are for tests at a moisture content between approximately 12% and 15%. The datapoints in the figures represent mean values of 160 hardwood species and 32 softwood species.

Because the sizes of the test specimens and the loading configurations might differ, the coefficient values of the regression lines cannot easily be compared with other databases and certainly not with structural timber. The datapoints given in figures 2.5 and 2.6 give the mean values for strength and stiffness against the mean density for a large number of wood species, both softwoods and hardwoods. The regression lines are forced through the origin. It can be observed that a linear relationship exists between density and strength and between density and MOE. This supports the assumption that there is a basic strength and stiffness of the woody cell wall material, independent of the wood species and that purely the amount of cell wall material determines the strength and stiffness. The amount of cell wall material is of course directly related to the density. The good correlation between MOE and bending strength is then a result of the fact that both can be well predicted by the density. This will be further elaborated in chapter 5.

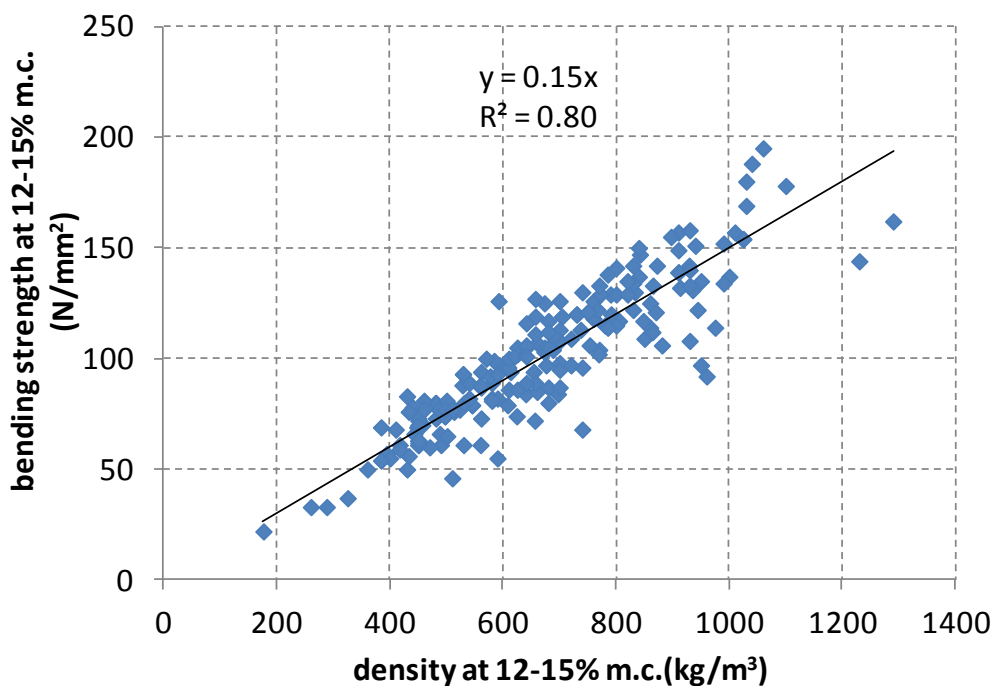


Figure 2.5. Mean bending strength values plotted against mean density values for clear wood for 192 softwood and hardwood species

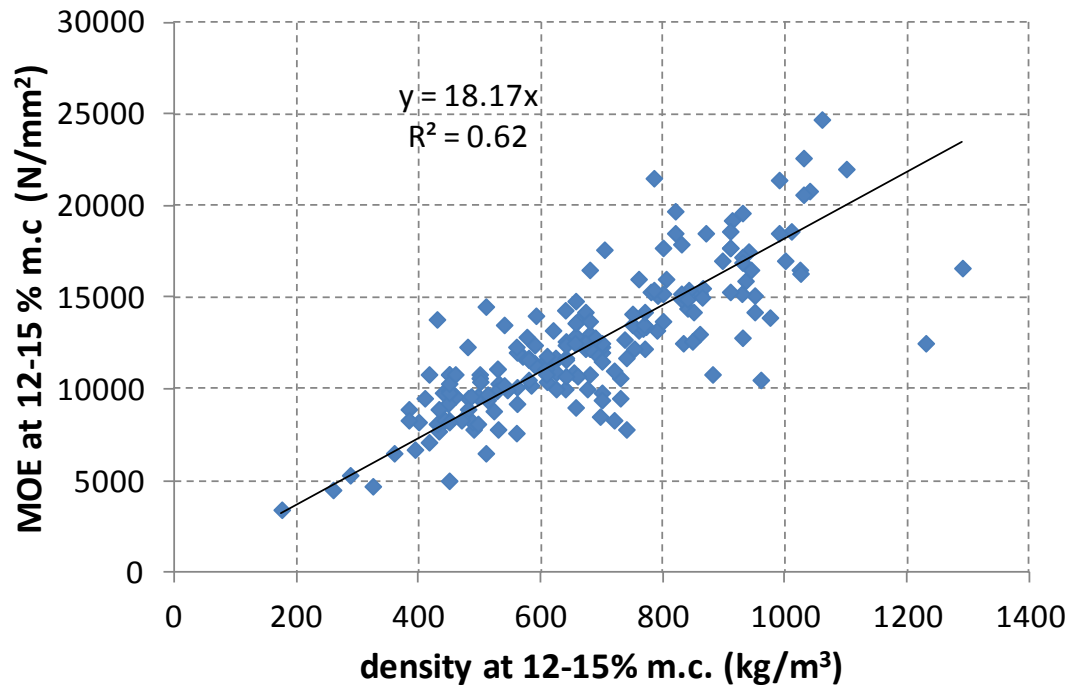


Figure 2.6. Mean Modulus of Elasticity values plotted against mean density values for clear wood for 192 softwood and hardwood species

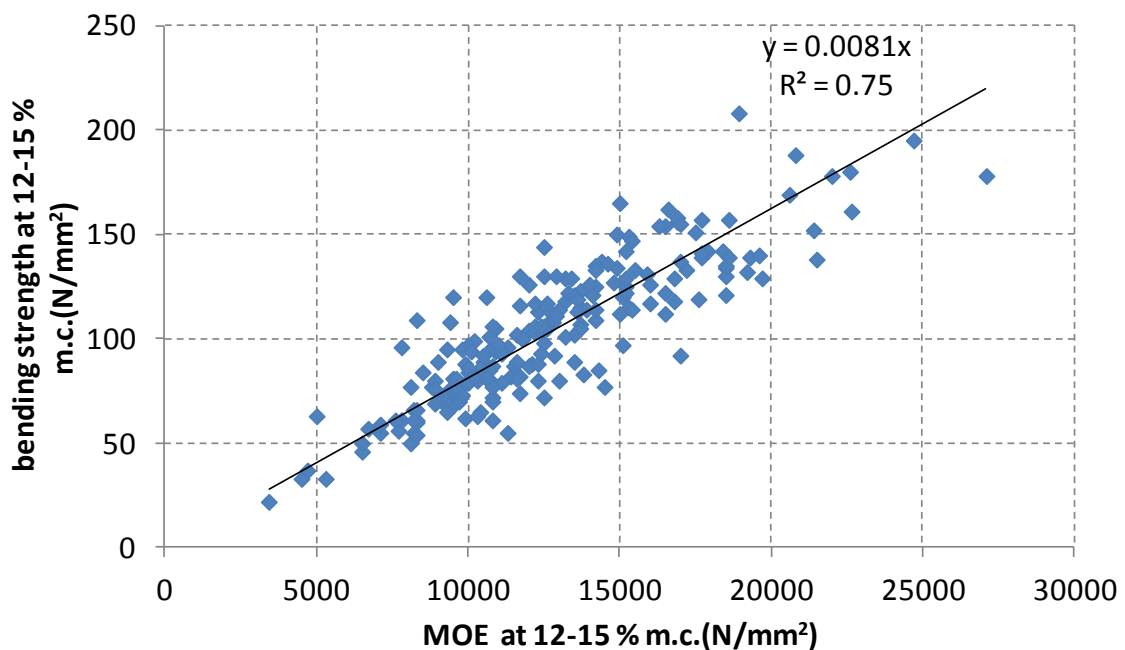


Figure 2.7. Mean bending strength values plotted against mean Modulus of Elasticity values for clear wood for 192 softwood and hardwood species

At timber level gross features are present. These gross features can have a great influence on the mechanical properties. The strength properties of small clear wood should therefore

not be used for timber of structural sizes. The differences between clear wood and structural timber are the influence of gross features, and the effects of size and moisture content.

The most important gross features are:

- Knots. Knots are the remains of branches in the stem. The branches start from the pith and develop in radial direction of the stem. This means that in the board, the grain direction of the knot material is perpendicular to the grain direction of the longitudinal direction of the board. A knot can have very different shapes depending on the growth development and the cutting pattern of the board. Another effect of the knot is that the main grain direction of the board will deviate around the knot.
- Grain angle deviation. For clear wood the grain angle deviation from the board longitudinal axis is zero. Apart from local grain deviation due to the presence of knots, there can also be a global grain angle deviation that is present over the whole length of the board. Other types of grain angle deviation are spiral or cross grain, where the grain angle differs in radial direction on the tangential surfaces. The grain angle deviation can also be presented as slope of grain (SoG). The slope of grain is the tangent of the grain angle deviation from the longitudinal beam axis.
- Cracks and fissures. Due to moisture content changes, internal stresses can occur that may cause cracks and fissures along the grain.
- Brittleheart or compression failures. These are failures in longitudinal direction that can occur due to growth stresses to impact loads. The result can be a crack perpendicular to the longitudinal axis of the board, leading to a very low strength.

At gross level, timber is therefore anisotropic but also inhomogeneous.

For clear wood it is proven that there is a size effect for properties with brittle failure as for instance the bending strength. E.g. Bohannon (1966) showed that larger sizes lead to lower strengths. Also the moisture content clearly has effect on the strength and stiffness. This effect is present until the cell walls are saturated with water. This is called the fiber saturation point (FSP). Above FSP the water cannot be absorbed by the cell walls and is free water in the timber, where it has no effect anymore on the mechanical properties. In the Wood Handbook (Ross et al., 2010) for a large number of species the strength and stiffness values for clear wood above FSP and at 15% m.c. are given. At 15% m.c., the bending strength can be up to 1.4 times as high than above the fiber saturation point.

The effect of size and moisture content for clear wood cannot be applied on structural timber because the effect of the gross features might interact with the effects of moisture content and size.

2.2 Timber as an engineering material

For engineering purposes, a concept of timber for structural applications has to be adopted. In the system of Eurocode 5 the following simplifications are made:

- The timber properties are assumed to be constant over the length of a structural element. Therefore, timber is assumed to be homogeneous in its properties.
- Timber is assumed to have strength and stiffness properties in the directions parallel and perpendicular to the longitudinal direction of the element.

This does not mean that in the process of assigning these properties (the grading process) more accurate models of timber cannot be used, incorporating knots, grain angle deviation, etc. In this thesis the concept of assuming similar mechanical properties in the planes perpendicular to the grain will be adopted.

A linear relation between strains and stresses according to Hooke's law is assumed for tension, compression and bending. (Remark: As will be shown in chapter 5 in compression and bending tests there is a linear relation visible, and after the elastic limit there is also a non-linear phase. However, Eurocode 5 does not take this phase into account in the calculation rules).

For engineering purposes the following strength and stiffness properties must be determined experimentally to describe the properties of timber: $f_{t,0}$, $f_{t,90}$, $f_{c,0}$, $f_{c,90}$, $f_{m,0}$, $f_{v,0,90}$, $f_{v,90,90}$, E_0 , E_{90} and $G_{0,90}$. The index 0 refers to the property parallel to the grain and the index 90 refers to the property perpendicular to the grain. Although these properties are independent from each other, some experimental relations are found that are used for strength assignments. In a testing program, only the mechanical property $f_{m,0}$ and E_0 are determined together with the physical property density (ρ). All other properties are derived from these three tested properties to define strength classes with strength and stiffness profiles. The accompanying equations are based on experimental experiences, mainly on softwoods. Whether these equations are applicable for (tropical) hardwoods for all properties is questionable.

A special remark has to be made on the index 0 in the engineering properties. With the index 0, is meant parallel to the longitudinal axis of the timber element. This does not have to be the angle of the grain with the longitudinal axis of the timber element (although it is specifically mentioned in EC 5 that this means parallel to the grain). The limits for the grain angle of the timber pieces are considered in the grading process. Timber with grain angles within certain limits is tested and is assigned to properties denoted with index 0. Later in this thesis, when a model for the strength properties of timber is developed, the real grain angle with the longitudinal axis is considered.

Strength under an angle with the grain.

The strength class profiles give the strengths parallel and perpendicular to the grain. For the strength under an angle with the grain, two failure criteria are used in EC5, the criteria of Hankinson and of Norris.

In figure 2.8, the strength distribution of a piece of timber subjected to a force under an angle of the grain is explained. This could be a piece subjected to an axial force over the entire height, but could also be the bottom slice of a beam under bending, small enough to assume a constant axial stress.

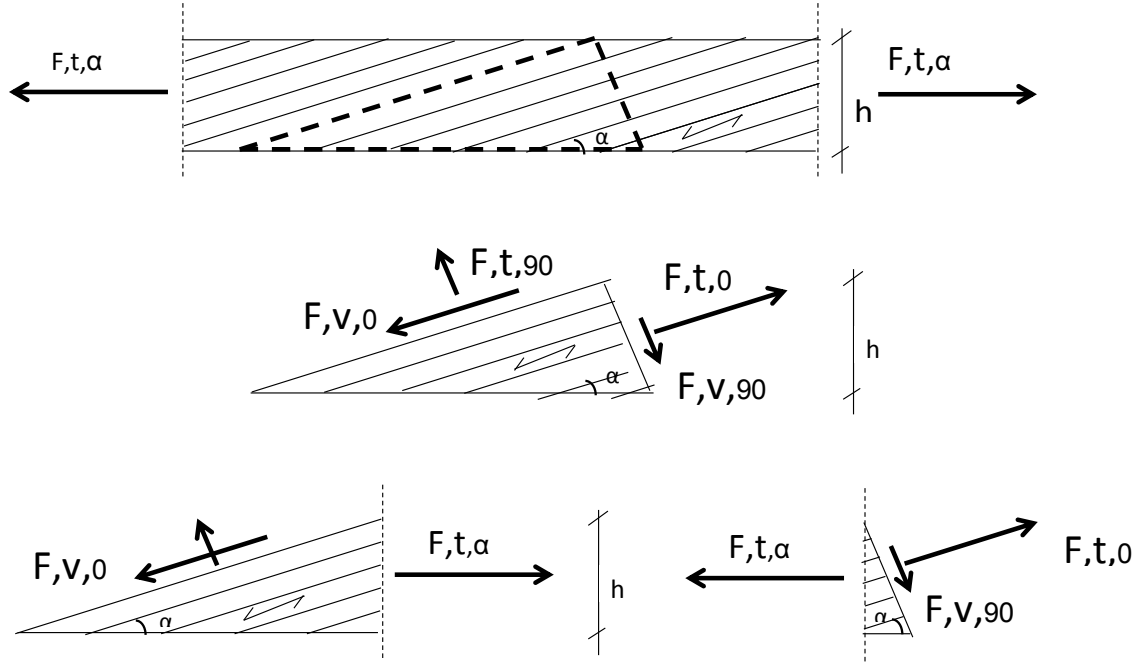


Figure 2.8. Strength distribution in a timber piece subjected to a force under an angle with the grain.

For timber beams subjected to tension with the grain angle deviating α from the beam axis, the stresses can be calculated directly from equilibrium equations for any value of the grain angle α .

The equations are:

$$\sigma_{t,0} = \sigma_{t,\alpha} \cos^2 \alpha \quad (2.1)$$

$$\sigma_{t,90} = \sigma_{t,\alpha} \sin^2 \alpha \quad (2.2)$$

$$\tau_{v,0} = \sigma_{t,\alpha} \sin \alpha \cos \alpha \quad (2.3)$$

The strength $f_{t,\alpha}$ can then be expressed as a function of the grain angle deviation α and the tension strength parallel and to the grain, perpendicular to the grain and shear strength parallel to the grain.

$$f_{t,\alpha} = \frac{f_{t,0}}{\cos^2 \alpha} \quad (2.4)$$

$$f_{t,\alpha} = \frac{f_{t,90}}{\sin^2 \alpha} \quad (2.5)$$

$$f_{t,\alpha} = \frac{f_{v,0}}{\sin \alpha \cos \alpha} \quad (2.6)$$

The equation that is used to describe the strength of timber under an angle with the grain is known as the Hankinson equation. It was first presented in a paper issued by the chief of air service in 1921 (Hankinson, 1921). This paper dealt with the compression under an angle with the grain. The following equation was presented:

$$f_{c,\alpha} = \frac{f_{c,0}}{k \sin^2(\alpha) + \cos^2(\alpha)} \quad (2.7)$$

With

$$k = \frac{f_{c,0}}{f_{c,90}} \quad (2.8)$$

Equation (2.7) was derived by curve fitting of test results on softwood and temperate hardwood species. The formula fitted well, both on the elastic limit as on the ultimate (plastic) limit. In the article where the Hankinson formula is proposed, the author gives an overview of the interaction formulas used before. The Hankinson formula was superior to the previous ones. Tension under an angle with the grain is not specifically mentioned in EC 5, but in for instance the Dutch National Annex, the Hankinson formula is used for tension under an angle with the grain. In EC 5, the Hankinson formula is used on graded timber, but it can also be used to model the strength of individual pieces since this is the way the formula was derived.

Equation (2.7) can be elaborated to a failure criterion, which gives the interaction between stresses parallel and perpendicular to the grain. This is shown for a timber piece under tension.

$$f_{t,\alpha} = \frac{f_{t,0}}{\frac{f_{t,0}}{f_{t,90}} \sin^2(\alpha) + \cos^2(\alpha)} = \frac{f_{t,0} f_{t,90}}{f_{t,0} \sin^2(\alpha) + f_{t,90} \cos^2(\alpha)} \quad (2.9)$$

$$f_{t,\alpha} \left(\frac{f_{t,0} \sin^2(\alpha) + f_{t,90} \cos^2(\alpha)}{f_{t,0} f_{t,90}} \right) = f_{t,\alpha} \left(\frac{\cos^2(\alpha)}{f_{t,0}} + \frac{\sin^2(\alpha)}{f_{t,90}} \right) = 1 \quad (2.10)$$

When $f_{t,\alpha}$ is replaced by $\sigma_{t,\alpha}$ the failure criterion becomes:

$$\sigma_{t,\alpha} \left(\frac{\cos^2(\alpha)}{f_{t,0}} + \frac{\sin^2(\alpha)}{f_{t,90}} \right) \leq 1 \quad (2.11)$$

$$\frac{\sigma_{t,\alpha} \cos^2(\alpha)}{f_{t,0}} + \frac{\sigma_{t,\alpha} \sin^2(\alpha)}{f_{t,90}} \leq 1 \quad (2.12)$$

$$\frac{\sigma_{t,0}}{f_{t,0}} + \frac{\sigma_{t,90}}{f_{t,90}} \leq 1 \quad (2.13)$$

This means that the Hankinson equation is a linear interaction equation between the stresses parallel and perpendicular to the grain. The influence of shear is not taken into account.

Several researchers have reported that the assumption of the Hankinson equation gives good results for practical applications. For instance Pope et al. (2005) performed tests on pieces taken from spruce scaffold boards with a range of grain angle deviations. The pieces were tested on bending. They found a good fit with the Hankinson equation.

The second failure criterion used in EC 5 is the Norris criterion. Where Hankinson describes a linear relationship, Norris (1962) describes a quadratic relationship. Norris proposes equation (2.14).:

$$\left(\frac{\sigma_{t,0}}{f_{t,0}}\right)^2 + \left(\frac{\sigma_{t,90}}{f_{t,90}}\right)^2 + \left(\frac{\tau_0}{f_{v,0}}\right)^2 \leq 1 \quad (2.14)$$

Whereas the Hankinson equation is experimental, the Norris equation is based on physical (constitutive) relations for orthotropic materials.

In EC 5, this relationship is used in the verification of tapered beams. The reference document by Riberholt (1979) states that the Norris criterion is only valid for glued laminated timber. Riberholt states that this has to do not specifically with the failure criterion but with the determination of stresses in a tapered edge which is assumed as an orthotropic continuum, which might give a good description for glulam (with relatively small inhomogeneities), but not for sawn timber beams with greater inhomogeneities.

To predict the failure bending strength, the axial strengths parallel and perpendicular to the grain and the shear strength are required. Riberholt concluded that it seemed that the shear strength is larger when it is combined with compression stresses perpendicular to the grain than when it is combined with a tension stress perpendicular to the grain. In Möhler (1979) this was confirmed by performing a regression analysis on test results with the Norris formula as basis. It was found that the Norris formula fitted well on the compression side when the shear strength value was doubled as compared to the tension side. This finding is integrated in extra factors in the EC 5 verification formula for tapered beams. Some tests were reported where the Norris criterion suited better for glulam than for sawn timber.

The Norris equation can also be rewritten as a strength function based on the grain angle deviation α with the beam axis and the strengths $f_{t,0}$ and $f_{t,90}$, and shear strength $f_{v,0}$:

$$f_{t,\alpha} = \frac{1}{\sqrt{\left(\frac{\cos^2(\alpha)}{f_{t,0}}\right)^2 + \left(\frac{\sin^2(\alpha)}{f_{t,90}}\right)^2 + \left(\frac{\sin(\alpha)\cos(\alpha)}{\beta f_{v,0}}\right)^2}} \quad (2.15)$$

The factor β is then also to be determined from experiments and can be different for shear in combination with tension parallel and perpendicular to the grain in comparison with shear in combination with compression parallel and perpendicular to the grain.

Both failure criteria are intended to be used for forces acting under an angle with the beam axis in EC 5, but they can also be used in the grading process, where the occurring stresses are parallel to the beam axis, but the grain angle deviates from the beam axis. In chapter 5 it will be investigated which failure criterion is most suited to be used in the modeling in this thesis.

Size effects on structural timber

Wood is considered as a brittle material under tension and as an elasto-plastic material under compression. Weibull (1939) developed a theory for homogeneous brittle materials that describes the effect of size on the strength distributions. The theory is based on the weakest link, viewing the material as a summation of elements with the same probability of failure. Assuming a serial system, the strength will be reduced when more elements are present. The size effect can occur in all three dimensions width, height and length. Since in the testing of the bending strength of timber the ratio of the span length and the height of the specimen is kept constant, these two cannot be distinguished. Then, for this size effect, either the height or the span length can be used. The size effect can be modeled as follows:

$$\left(\frac{f_{m,2}}{f_{m,1}}\right) = \left(\frac{h_1}{h_2}\right)^k \quad (2.16)$$

Where $f_{m,1}$ is the strength at a chosen reference height h_1 . This means that for any other height h_2 the bending strength $f_{m,2}$ can be calculated with:

$$f_{m,2} = \left(\frac{h_1}{h_2}\right)^k f_{m,1} \quad (2.17)$$

It must be emphasized that the Weibull theory predicts differences in the strength distributions of a homogeneous brittle material. The factor k is related to the cumulative percentage level of the strength distribution of a material for a certain height.

The same formula can be set up for the width and length, but these effects are not included in the Eurocode 5.

The above formula might be valid for timber without knots, for timber with knots it is debatable. The timber elements then are no longer homogeneous, so a pure Weibull size effect is not to be expected. However, there can be a size effect that has a physical reason: the relative size of the knots will decrease when specimens are cut from a larger element. On the other hand, a larger beam might also increase the probability of the presence of the pith of a beam, which has a lower strength and stiffness. In Rouger (1995), values for size factors found by different researchers are listed. More recently, Denzler (2007) studied the size effect based on visual grading for structural timber. Denzler concludes that there are

counteracting effects, and that an effect might be present for average strength values, but this effect might not be present on the 5% fractile. An average Weibull factor of $k=0.10$ was found for both knot-free and knotty specimens. The recommendation was that the size factor could be removed from the design codes for softwoods with knots. Stapel and Van de Kuilen (2013) compared different visual grading methods for softwoods and showed that the size effect is dependent on the method of quantifying knots.

For this research, the main purpose is not to find the size effect for visual grades, but only for machine grading and in particular to find the differences between softwoods and hardwoods. Therefore, the size effect that arises after machine grading has to be investigated. This will be done in chapter 5 for a dataset of softwood (containing knots and no grain angle deviation) and a dataset of hardwood (containing grain angle deviation and no knots). The size effect will be studied for the developed models in this thesis and is to be used in machine grading.

A size effect for the MOE is not expected. A size effect could only be present when for instance the density would vary considerably over the height of the specimen. In practice the variation in density between beams is larger than within beams.

Effect of moisture content on structural timber

The effect of moisture content on the bending strength and stiffness is of particular importance for tropical hardwoods, because the moisture content at testing is mostly higher than the reference moisture content of 12%. The effect of moisture content on the MOE is quite clear, also for structural timber. Research has shown that this effect occurs for both the static MOE (derived from laboratory bending tests) and for the dynamic MOE (derived from the stress wave velocity and density). Gerhards (1975) found for sweetgum that the stress wave velocity decreased with increasing moisture content. Unterwieser and Schickhofer (2011) repeated this research for spruce and found that the dynamic MOE decreased at FSP to approximately 85% of the 12%-value and above FSP remained practically constant.

Green and Evans (1992) conclude from studies on softwoods species that the change in strength distributions with the change in moisture content is a function of the quality of the timber. However, when only the 5% fractile strength level is regarded, this values does not necessarily increase with decreasing moisture content. With the quality of softwood is meant the presence of knots relative to the dimensions. For tropical hardwoods, very little data on structural timber are available. Since in tropical hardwoods knots are rare, the effect on strength might be different.

2.3 Determination of mechanical properties of structural timber

2.3.1 General principles

Because wood is a natural material, there is a great variability in mechanical properties of structural timber coming on the market. To make economic use of structural timber, it is divided into different groups to which strength properties are assigned. The divisions are based on characteristics that influence the mechanical properties. This process of dividing timber into different groups is called grading. The two common methods are visual grading and machine grading, which will be explained in the next sections.

To assign strength properties to grades, a testing program has to be performed in which material is tested that is representative for the timber coming on the market in the future. The objective of the testing program is therefore to predict with a high degree of certainty the strength properties of grades that incorporate the possible scatter to be expected within that grade. Possible causes of the scatter include aspects as:

- The growth conditions of the trees, which can be influenced by forest management and climate.
- The tree species.
- The processing of the tree to timber.
- The accuracy of the grading method.
- The influence of moisture content.
- The influence of size.

To identify the scatter within a grade in the testing program, the following requirements are made to the sampling according to the European system (EN 14081-1):

The testing program must include:

- Sufficient samples from the entire growth region the strength class assignments will be valid for.
- Timber specimens that are representative for the regular production.
- Identification of the tree species tested for which the strength class assignments will be valid.
- A range of sizes and moisture contents for which the strength class assignments will be valid.
- Sufficient specimens within a sample to state reliable strength properties.

It may be clear that it is very difficult to distinguish all possible causes of scatter in advance and set-up a testing program incorporating all possible influences. The determination of the sampling program is therefore one of the most delicate activities and is subject of many discussions.

As described in section 2.2, in Europe, strength class profiles are defined to make application easier in practice, but one's own profiles can also be defined. In both cases,

the testing program should determine the characteristic values of the bending strength, the modulus of elasticity and the density. The characteristic values are defined as:

- The 5% fractile for the bending strength
- The mean value for the modulus of elasticity
- The 5% fractile value for the density.

The other mechanical properties may be derived from these values.

The characteristic values of the grade of a species have to be calculated by combining the test results of the samples. In the next sections it will be explained how these are combined for use with visual grading and for machine grading.

The strength properties according to EN 338 are given for a reference moisture content of 12% and a reference depth of 150 mm. The test results have to be adjusted to these reference conditions before the characteristic values are determined. The methods and factors to combine samples and to adjust the test results for moisture content and size are ongoing subject of discussion. In this thesis the situation described in the European standards, valid in 2013, is taken as a basis.

The test set-ups used for the determination of the material properties used in this thesis are described in chapter 3.

2.3.2 Strength properties based on visual grading

Visual grading is the oldest method to predict the mechanical properties of timber. The method is based on identifying the strength and stiffness reducing features and puts limits to the extent they occur in a piece of timber. According to EN 14081-1 (CEN, 2011), visual strength grading is defined as: “The process by which a piece of timber can be sorted, by means of visual inspection, into a grade to which characteristic values of strength, stiffness and density may be allocated.” In the accompanying note it is mentioned that electronic or mechanical instruments can be used to assist the visual grader in this process. The most influencing features are knots and grain angle deviation. Another parameter that can be used is ring-width, which can be a measure for the density, depending on the wood species. The visual grading rules have been developed on national levels and until now it has not been possible to agree on common European visual grading rules.

When all the features have been measured, a linear prediction model can be defined, with the strength, stiffness or density as the dependent parameter and the characteristics as independent parameter. The most critical value of that feature of a specimen within the tested area is used as input. With a multiple linear regression, the correlation coefficient r or the coefficient of determination r^2 can be calculated. Hanhijarvi (2005) gives an overview of the increase of the r^2 -value, by adding more features as parameters in the regression. The number of grades (and different strength class assignments for each grade)

that can be defined depend on the range of the magnitude the features appear. This is explained in figure 2.9.

A common measure for the influence of knots is the knot ratio. The simplest definition of this is the size of the knot perpendicular to the length axis divided by the depth of the beam at the cross-section where the knot occurs. The depth is the dimension of the beam in the loading direction, this is normally the largest dimension of the cross section. (When the largest dimension is used as depth this is called edgewise bending. For the tests performed in this thesis all beams were tested edgewise). The largest value for this measure on either side of the beam within the test length is used. In the top of figure 2.9 an example is given for a test set with a knot ratio range of 0 to 0.6. The plot shows a contour of the data points knot ratio-bending strength for every test specimen. This plot is typical for softwood. In practice, three is the maximum number of grades that can be distinguished. In this case, in figure 2.9, two grades are defined with knot ratios between 0-0.2 for the highest grade and between 0.2-0.3 for the lowest grade. The pieces with a knot ratio higher than 0.3 are called reject: no strength properties can be assigned to them. The data points containing the lowest 5% bending strength values in every grade are marked with a black triangle. The horizontal line above these triangles gives the 5% fractile of the grade at the intersection point with the vertical axis. There must be sufficient distance between the 5% fractiles of the grades to make the grading useful. This plot is typical for softwood species and some temperate hardwood species. At the bottom of figure 2.9, a typical situation for tropical hardwoods is given. In most hardwood species, knots are rare in the timber retrieved from the trees. (This is due to the fact that the trees from hardwood species normally have a larger diameter and that the crown is not used to convert it to structural timber). When this is the case, only one visual grade can be distinguished.

In practice, a predicting model based on a multiple regression of the features as parameters is not used for visual grading. Using this model would mean that the visual grader has to enter the parameter values in a software program which then calculates the predicted strength and stiffness values. In practice, the visual grader checks all the features. The feature that is present with a magnitude that is connected to the lowest visual grade (the grade connected to the lowest strength properties) is governing. The timber beam is assigned to that critical visual grade and the connected strength class.

In appendix B the requirements for visual grading according to the Dutch standard NEN 5493 for tropical hardwood are given.

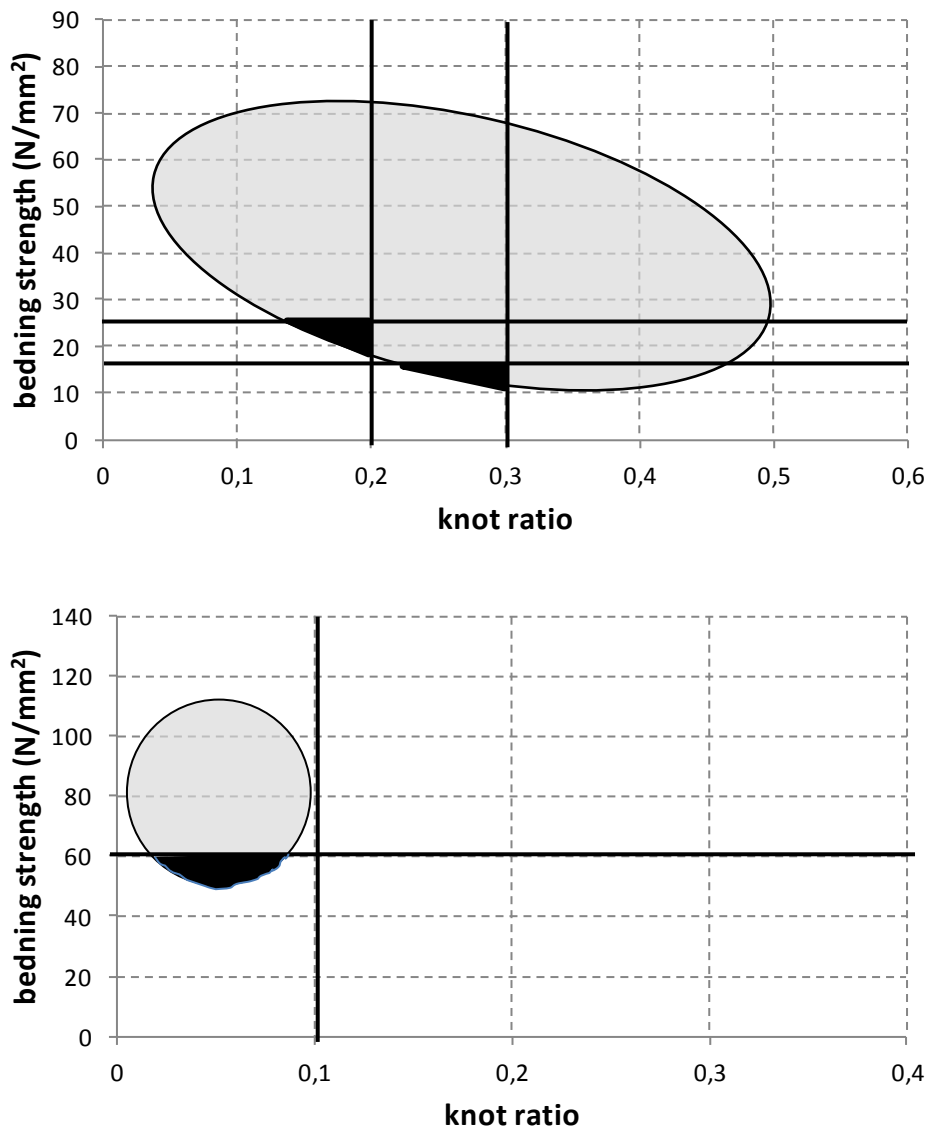


Figure 2.9. Typical regression plots of the bending strength against the knot ratio for softwoods (top) and hardwoods (bottom).

2.3.3 Strength properties based on machine grading

According to EN 14081-1 machine strength grading is defined as: “process by which a piece of timber can be sorted by a machine sensing, non-destructively, one or more properties of the timber, with any necessary visual inspection, into grades to which characteristic values of strength, stiffness and density may be allocated.” Machine grading also requires a so-called visual override. This means that each machine graded piece has to be checked for strength reducing features that cannot be detected by the machine. These can be soft rot or insect damage, fissure lengths and distortions, but also abnormal incidental defects like compression failures.

Strength grading machine can use the concept of a machine controlled system or an output controlled system. For machine controlled systems, the settings (grading limits) are determined in a research plan comprising representative testing of the timber to be graded in future. With output controlled systems, only a small testing plan is performed to determine the initial settings, and during production, on a regular basis pieces are taken from production and tested. When necessary, the settings are updated. Output controlled systems are mainly used in situations where over time a relative homogeneous production is present. This implies that the same species and same dimensions are graded over a long period of time. Machine controlled systems are used where a wider variation of species and sizes is expected. In Europe, only machine controlled systems are used. Because this thesis deals with the possibility of defining species independent strength models, it will focus on models intended for machine controlled systems.

Machine strength grading uses properties derived by electronic measurements on the timber to predict the strength without damaging the timber. The measurements can be deformation, vibration, the transmission of irradiation etc. These measurements are used to calculate material properties like the modulus of elasticity, the density, dimensions and moisture content. In Hanhijarvi (2005), an overview of non-destructive measurement techniques is given. The calculated material properties from non-destructive measurements are used to create predicting models. It has been shown that material properties from NDT-measurements have much better predicting capabilities than measurements performed in visual grading. As a result, there is less scatter around the prediction line and a higher yield in higher grades is possible. In figure 2.10 typical results for machine grading are given. These figures can be compared with figure 2.9 for visual grading.

In the top and bottom figures 2.10, typical data clouds are shown for the relationship between the modulus of elasticity and the bending strength. At the top, a typical result is given for softwoods. The scatter is much less than for visual grading and therefore it is possible to grade in a higher strength class than with visual grading and because of the higher accuracy also with higher yields. At the bottom of figure 2.10 a typical result for a sample size with a limited number of specimens for a single tropical hardwood species is given. Because the range in both MOE and bending strength is more limited than for the softwood in the top of the figure, the prediction capability is low. This is one of the motives to investigate species independent modelling in this thesis.

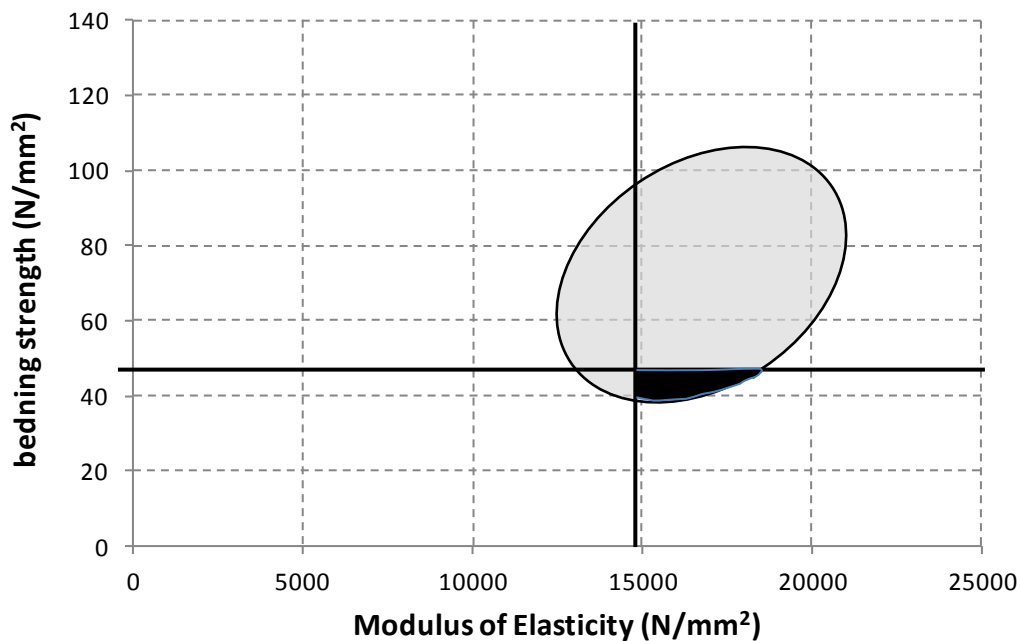
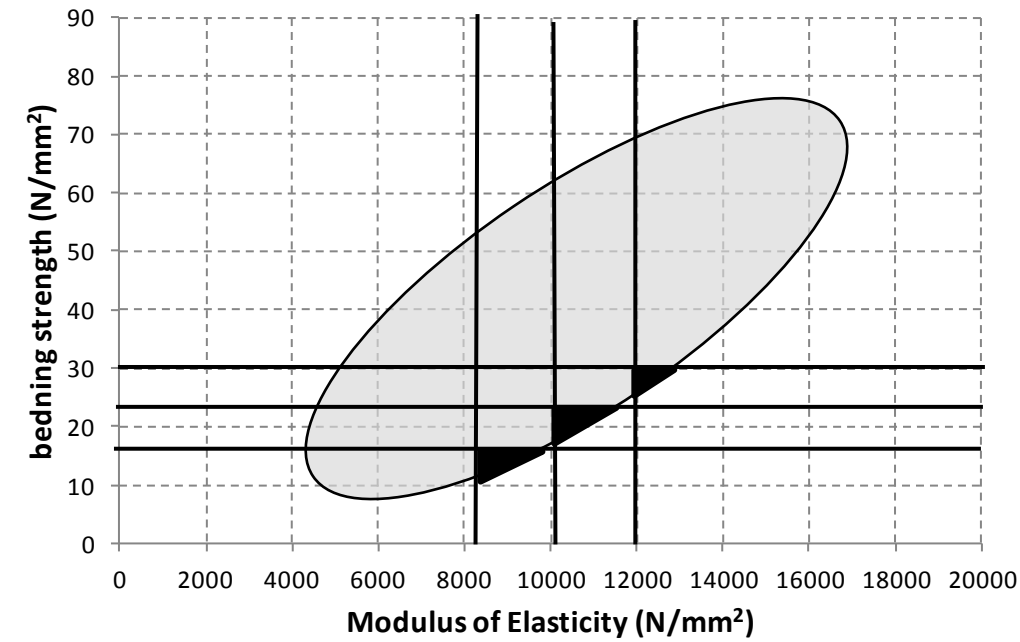


Figure 2.10. Typical regression plots of the bending strength against the modulus of elasticity for softwoods (top) and hardwoods (bottom).

For machine grading it is possible to derive predicting models with multiple parameters, because the software integrated in the machine is capable of calculating these and send signals to the marking unit to mark the piece with the right strength class on the basis of the model prediction compared with the settings. Settings for different strength class combinations can be made. In this way e.g. higher yield in a high grade or less amount of pieces that are rejected can be optimised.

The procedure for deriving settings for machine grading is given in European standards EN 14081-1 (2011) , EN 14081-2 (2012) EN 384 (2010) and EN 408 (2012)

2.4 Overview of standardised methods for determining 5% fractiles based on one sample.

To evaluate which strength class may be assigned to a strength grade (either determined by visual or machine grading) the characteristic values of the strength grade have to be determined. Structural timber is susceptible to a variation in strength properties. This has to be taken into account when the characteristic values of a strength grade are determined. The objective is to give a good estimation of the characteristic value of the population from the test data. “Good” can be difficult to define, because whatever statistical method is used, all methods make certain assumptions about the underlying population which are very difficult to verify. In this section the statistical methods and backgrounds for deriving the characteristic value according to several worldwide standards for timber are analysed when one or more test samples for a specific (visual) grade are present. In section 3.3, the statistical methods used in this thesis to determine the characteristic values of timber for visually graded samples are presented. Section 3.4 will focus on the derivation of characteristic values based on a prediction model. A method has been developed by the author under the assumption that the distribution properties of prediction model values and the scatter of model errors gives more information about the scatter in strength properties and that this information can be used to determine the characteristic value. In this chapter, the focus will be on strength, for which the 5% fractile is accepted worldwide as characteristic value.

2.4.1 Parametric methods

There is a difference between calculating the 5% fractile of a sample property simply based on the available data and estimating the 5% fractile of the population the data is sampled from. For the first approach, the 5% fractile can be calculated using the distribution the data follows. This is called the 5% point estimate. When the data follows a normal distribution, the t-distribution could be used. However, for engineering purposes, the second approach is necessary. In this section the parametric methods according to EN 14358 (2007) and ASTM D2915 (2003) are described. They both follow the same approach for determining the Parametric Tolerance Limit (PTL). The PTL can be defined as the value for which, with a probability of α the $p\%$ fractile of the underlying population is higher than this value. The assumption is made that the underlying population is normally distributed. α is called the confidence level.

For calculations with the available sample data the standard deviation has to be multiplied by a factor k (the confidence level factor) and subtracted from the mean:

$$PTL = \bar{x} - ks \quad (2.18)$$

Where

$\bar{x}$ is the sample mean value

s is the sample standard deviation

k is the factor determined from the non-central t-distribution based on α , p and n , the number of specimen.

From the definition of the PTL follows the requirement:

$$Prob(\bar{x} - ks \leq \mu - z_p \sigma) = \alpha \quad (2.19)$$

Where

μ is the mean of the underlying population

σ is the standard deviation of the underlying population

z_p is the value of the normal distribution for the $(1-p)^{\text{th}}$ -fractile.

p is the fractile to be evaluated. For $p=0.05$ the value $z_p=1.645$.

The outcome of the derivation of the factor k out from equation (2.19) is given in equation (2.20):

$$k = (NCT^{-1}(z_p \sqrt{n}, n-1)(\alpha))/\sqrt{n} \quad (2.20)$$

Where

NCT^{-1} is the inverse of the non-central t-distribution

n is the number of datapoints in the test sample.

The factor k is for the percentile value that is evaluated (z_p) therefore depending on the required confidence level α and the number of datapoints n in the test sample.

ASTM D2915 gives a table with values of k for various values of α and p . EN 14358 gives values for k for $\alpha=0.75$ and $p=0.05$ only.

The above procedure is also applicable when the underlying population is assumed to be lognormally distributed. Then for all the data the natural logarithmic value is taken. These transformed data can be assumed to come from a normally distributed population and the confidence level factor k can be applied on them. After the PTL of the transformed data is calculated, the exponential value has to be taken to find the PTL of the untransformed data.

ASTM D2915 leaves open which underlying distribution should be used for strength properties. EN 14358 states that for strength a lognormal distribution should be assumed.

2.4.2 Non-parametric methods

By non-parametric methods is meant that the distribution properties of the test data are not used in the determination of the 5% fractile. This method is used by ranking the test values in ascending order and selects the appropriate order value for the required test value. Contrary to parametric methods therefore only the lower tail values are used. Non-parametric methods might be an alternative when the distribution of the test data is not known or no clear distribution can be fitted.

In EN 384, the 5% fractile of a sample is the order number $n/20$, where n is the number of pieces in the test sample. If this is not an actual test value, then linear interpolation between the two adjacent values is required. A confidence interval is not given, which means that this actually is a point estimate of the 5% fractile.

When a non-parametric tolerance limit (NTL) is to be determined with a confidence level α , tables are available in ASTM D2915. These are based on Guttman (1970). He describes and proves the work of Wilks (1941), who showed that distribution free tolerance intervals can be described by the Beta distribution. The only requirement is that the probability density function has to be continuous, which means that every X_{i+1} value must be higher than the X_i value, where X_i is the order number of the test value. Guttman (1970) also presents the approximation equation given by Scheffé and Tukey (1944), for which he reports that the outcome deviates from the exact solution by less than 0.1 % only. This equation is given here as equation (2.21).

$$n = 0.25(\chi^2_{1-\gamma, 2m})^{-1} \left(\frac{1+\beta}{1-\beta} \right) + 0.5 * (m - 1) \quad (2.21)$$

Where

n is the required sample size when the m^{th} order is to be used for the $(1-\beta)^{\text{th}}$ percentile value with a confidence level of γ .

$(\chi^2_{1-\gamma, 2m})^{-1}$ is the inverse of the cumulative distribution function (CDF) of the chi-squared distribution for a probability of $1-\gamma$ with $2m$ degrees of freedom.

β in equation (2.21) is related to p as defined below equation (2.19) by : $\beta = 1-p$. When p is inserted in equation (2.21) , this becomes:

$$n = 0.25(\chi^2_{1-\alpha, 2m})^{-1} \left(\frac{2-p}{p} \right) + 0.5 * (m - 1) \quad (2.22)$$

With

p is the probability to be evaluated and

α is the required confidence level

For $\alpha=0.75$, $\alpha=0.95$, $\alpha=0.99$ and $p=0.05$ the required number of specimens n when order value m is to be used is calculated with equation (2.22) and tabulated in ASTM D2915.

The influences of sample size on for instance the tolerance limits of samples from a population can also be investigated with Monte-Carlo methods, whereby sampling simulations are made on existing or simulated populations.

The derived characteristic values and Tolerance Limits are derived for the situation that one random sample is taken from a normally distributed population. With more than one sample, the accuracy of the estimation of the population tolerance limit increases, and therefore the k -factor could decrease.

2.5 Overview of standardised methods for determining 5% fractiles based on N samples.

In the previous section, the focus was on the determination of the values of interest of a population, based on one sample. It was thereby assumed that the tested sample was representative for the entire population, meaning that the tested dataset is randomly drawn from a population that is homogeneous in itself. For timber to be tested for strength class assignments, it can be questioned how the definition of the population is and whether this population is homogeneous. Therefore it is recommended that more than one sample is taken from the population. This introduces the following questions:

- How should these samples be drawn from the population?
- How should the test results be combined?

In order to answer these questions we first have to define the population. In Europe, the visual and machine grading systems are based on an initial testing plan. From the population samples are drawn that are representative for the present and future production. When samples are drawn from a single population, the test data will represent the expected scatter. The question is how the population is defined:

- Timber from a species from a specified forest area?
- Timber with the same trade name? There are situations that the timber is sold under a trade name, where the timber can be of the same genus, but coming from different tree species.
- Timber from a species from a sawmill?
- Timber from a species from a number of sawmills?
- Timber delivered to a building site?

It may be not clear in advance that the samples to be combined are coming from the same population. In fact, it may happen that samples from different populations are merged and strength properties are derived as if they were coming from one population. This may happen for instance with a tropical hardwood species coming from two regions, which we want to market as one population (the wood species), when in fact there are two populations (for instance because there are different growth conditions in the two

regions). Two populations could also be created by the way of processing. When there is a limit for a visual strength class for the slope of grain of a maximum of 1:10, for a wood species from the same region one batch could be sawn very precisely with a very low slope of grain ($<1:20$), whereas in the other batch the slope of grain varies from 1:15 to 1:10. Both batches comply with the visual grading rules for the grade, but because the grain angle deviations are not randomly equally distributed over the two batches, they could strictly speaking also be regarded as two different populations, although they will be put on the market as coming from the same population.

The question whether samples are drawn from the same population is very difficult to answer, but is mostly handled in a practical way. Standards have to deal with these practical differences. Basically, there are two approaches:

1. The samples with the weakest strength properties are governing.
2. The strength properties of the samples are averaged.

Approach 1 of course is the safest and approach 2 the more economical one. The consequences of the choice for either approach 1 or 2 could also be integrated in the safety factors that are applied. However, according to the author, at present the choice for approach 1 or 2 is based more on expert judgment than a thorough statistical analysis.

Combining of samples according to current standards

In the pre-standard ISO/CD 12122-1(2012), the samples with the weakest strength properties are governing for the population. By a chi-squared test it has to be evaluated whether the samples are coming from a homogenous population. This is done by first determining a provisional 5% fractile with a 75% confidence interval based on all data together. For every sample, the percentage of pieces below this value is calculated and a chi-squared test is performed on these values compared with the expected amounts of pieces (which is 5% for every sample). When for a set of samples the chi-squared test is not significant at the 0.01 level the hypothesis that the samples could come from the different populations is rejected. In that case these samples are combined to one sample that is used in further calculations (In this thesis they sometimes will be called homogeneous, although this is strictly not the outcomes of the statistical test performed). The procedure starts with the 2 weakest samples. When the chi-squared test is not significant at the 0.01 level, then stepwise samples are added (in order of the 5% fractiles of the samples) until the chi-squared test is significant at the 0.01 level (then the hypothesis that the samples could come from the different populations cannot be rejected). The samples for which the chi-squared test was not significant are assumed to be able to be combined to one sample and are regarded as a homogeneous population. The 5% fractile with a 75% confidence interval based on the merged data of these samples is then determined and this value is the 5% fractile of the population the samples represent.

The approach of EN 384 is based on the average strength of the samples. The mean of the 5% fractiles of all samples, weighted according to the number of pieces in each sample, is calculated. If this mean value is higher than 1.2 times the 5% fractile of the weakest sample, then a characteristic value of 1.2 times the 5% fractile of the weakest sample has to be used. That value has to be multiplied by a factor k_s , taken from a figure depending on the number of samples and the number of specimens n in the sample with the lowest number of specimens in it. The factor k_s varies from 0.78 ($n=40$) to 0.9 ($n>200$) for one sample, from 0.88 ($n=40$) to 0.96 ($n=150$) for three samples and is 1.0 (for $n>40$) for 5 samples.

The background for the k_s factor can be found in Fewell and Glos (1988). To take into account the variability between the 5% fractiles of subsamples, 20 subsamples of 100, 200 and 300 pieces were randomly selected from a parent sample of 652 pieces of European redwood/whitewood. The result is shown in figure 2.11. This figure was adopted and modified to the k_s factor, and included in EN 384.

Evaluating the background of the k_s -factor, it can be concluded that it was based on simulations on a homogeneous population, therefore a lot of the questions for defining a population are not incorporated in the derivation of the k_s -values.

Apparently, the standard authors were aware of this fact and therefore introduced the requirement that the mean of the 5% fractiles of the samples may not be higher than 1.2 times the 5% fractile of the weakest sample.

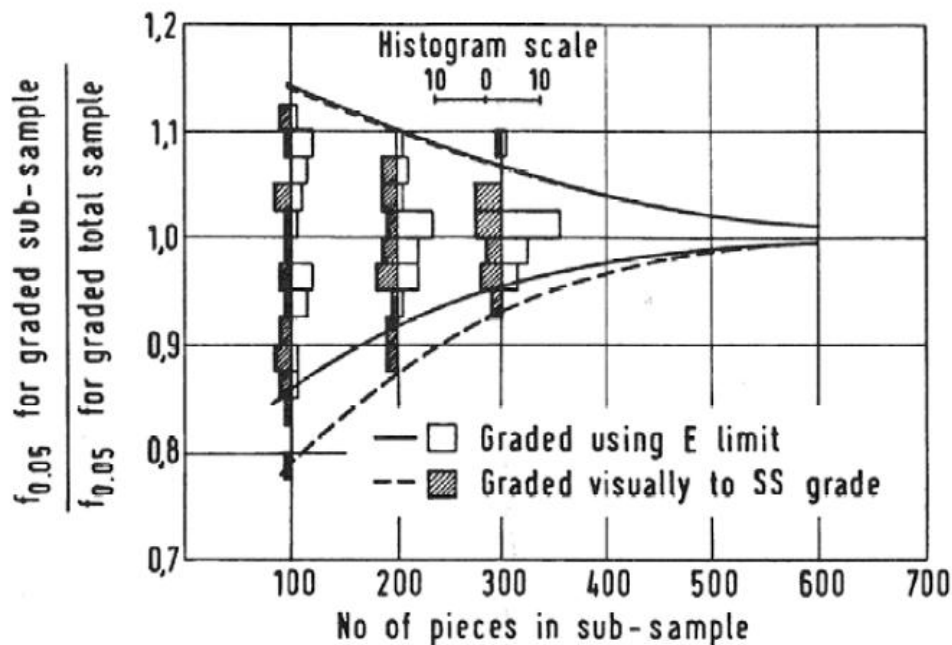


Figure 2.11 Ratios of bending strength 5% fractiles bending strength of randomly selected sub-samples from a parent sample of 652 pieces. Taken from Fewell and Glos (1988).

2.6 Historical development of strength properties values for structural hardwood timber based on visual grading

In the 20th century the standardisation of grading procedures and property assignment of structural timber has evolved, and is not finalised yet. The main topic of discussion throughout history has been how to develop test methods and grading rules in such a way that they reflect the strength properties of timber used in construction. Since timber is a natural product, solid timber is the outcome of the growth process of trees and the processing of the timber. The challenge for the researchers has been to address the variability that is a consequence of these processes and to provide industry with safe and economically useable strength figures.

Historically, strength properties have been derived from samples of small clear test samples. To determine the full size properties, adjustment factors have been derived. Nowadays, full size testing is the main method.

The first standards with visual requirements for structural timber as a building material in The Netherlands were developed in the 1920's (N1012:1927). The standard N1055:1955 included both design rules as material properties tables. The strength values given were allowable stresses. In N1055:1955, strength values for oak and djati (teak) were given. The source of these strength values is not known. The timber had to comply with the visual sorting criteria according to N1012:1932.

In the early 1960s, a number of tropical wood species were tested at the TNO Houtinstituut (reported in Houtinstituut TNO (1961,1961,1961,1962,1962)) in bending for both structural sizes (50 x 150 mm) and small clear pieces (50 x 50) cut from them. The structural sizes were tested in a four-point bending test with a span of 3000mm ($l/h=20$), where the small clear pieces were tested in a three point bending test with a span of 700 mm ($l/h=14$). Also, compression tests were performed. 50 specimens of every species were tested. These species were tola branca, iroko, keruing (yang), peroba de campos and basralocus.

To address the variability of species iroko and basralocus, the timber was supplied by respectively 5 and 4 different suppliers. It was noticed that a small number of pieces were left out of the analysis because anomalies were detected before any visual quality rules were applied. All specimens were graded according to N1012:1940 and the non-conforming specimens were left out of the analysis.

Based on the comparisons between the ratio of the 1-percentile values of the structural sizes and the small clears, a factor between 0.69 and 0.88 was found. A factor of 0.75 was determined as an average ratio between small clears and full size test data for all species, which was called the quality factor. (The possible influence of loading configurations and size was not taken into account).

In the Dutch design standard for timber structures of 1972 NEN 3852 (1972), the list of species was extended as compared to the 1955 version.

The backgrounds for these assignments were given in Houtinstituut TNO (1963).

In Govers (1966) an overview is given of the procedures for deriving strength properties from small clear and structural sizes. Various research organisations used different statistical methods:

- Forest product research laboratory (FPRL), USA. Bending test results were carried out on small clear specimens. The basic strength (which could be compared with the characteristic value as we use nowadays) was the 1-percentile value, assuming a normal distribution. To make the step to full size specimens (which was called commercial quality), the 1% fractile was multiplied by a factor of 0.75 which was called the quality factor. A safety factor of 2,25 was used.
- Division of Forest Products (DFP), Australia. Bending test results were carried out on small clear specimen. Also the 1% fractile was determined. This value was multiplied by 9/16 to address the long-term strength of timber and by 4/5 to address errors in design and execution.
- Timber Research And Development Association (TRADA), United kingdom. Bending tests were carried out on timber of commercial quality. The basic stress was the 2.5% fractile. This value was multiplied by 9/16 to address the long-term strength of timber and by $\frac{3}{4}$ as a safety factor.

In Houtinstituut TNO (1963) the procedure of FPRL was applied. The full size test results were used to determine the quality factor for which an average of 0.75 was found. From data from literature on small clear specimens, the 1% fractile were determined assuming a normal distribution. Then, this value was multiplied by the quality factor and divided by a safety factor of 2.25. The values were given based on data on green (wet) small clear data. The allowable stresses for hardwoods in TGB 1972 NEN 3825 were given for use in wet conditions only. Therefore, no correction factor was given for use in dry conditions. Based on this report, a list of 15 hardwood species with strength values were incorporated in NEN 3825. These species were: tolabrance, sipo, European oak, wane, yang(keruing), iroko, niangon, afzelia, kopie, peroba de campos, basralocus, bilinga, merbau, azobé and Demerara greenheart.

For TGB 1990 (NEN 6760), the system of limit state design was applied. This implied that for the material strength properties the characteristic values were now given, and safety factors and modification factors had to be applied in the calculations by the engineer. A test standard for timber based on structural sizes was then developed (NEN 5498:1991) which became the mandatory test method in The Netherlands. Based on international research, the size factor was introduced in timber design standards. In the Dutch standard, the reference test set-up now was a four-point bending test with $l/h = 18$ with $h=200$ mm as the reference height and l being the span.. The bending test value had to be readjusted to 200 mm by dividing it by the factor $(200/h)^2$. The characteristic value was defined as the 5% fractile determined by the method of ranking. As a result, the test data based on small clear specimens could no longer be used anymore. Based on historical structural sized data on azobé in the period 1970-2000, the strength class for azobé with

the connected visual grading rules was determined. The backgrounds for this assignment are given in Van de Kuilen and Blass (2005).

With the introduction of the Eurocodes the European test standards were introduced. The main difference with the Dutch standard NEN 5498 was the reference height. According to the European standard EN 384, the reference height is 150 mm, so all test results have to be adjusted to this size.

Strength class assignments based on (national) visual grades are listed in the European standard EN 1912 (2012). This standard is a collection of strength class assignments of both softwood species and hardwood species with their connected visual grades. Their assignments are based on full size data except for a number of hardwood species, which will be discussed in the next section.

A development in Europe is the introduction of harmonised standards. With the introduction of the system of Eurocodes, the derivation of strength properties is harmonised. Before the Eurocodes were developed on a national level strength class profiles were defined. Also for machine grading national standards were written. The Eurocodes (with NEN-EN 1995-1-1:EC 5 for Timber Structures) are a set of design and verification rules, that refer to harmonised product standards for the material properties. The objective of the harmonised product standards is to ensure that the material properties are derived in a similar manner all over Europe. The harmonised product standard EN 14081-1 was developed for this purpose. This includes a harmonised testing standard (EN 408) and a harmonised standard for determining the characteristic strength values (EN 384) of tested and non-tested properties. Also, a European standard with a strength class system (EN 338) was developed. Although not mandatory, this strength class system is used by most producers all over Europe. In appendix C, the strength classes according to prEN 338 (CEN, 2014) and the characteristic values for the bending strength, modulus of elasticity and density are presented. In this thesis these strength class profiles will be used to assign graded timber to strength classes.

For visual grading, the characteristic values have to be assigned to visual grades. So far, it has not been possible to develop visual grading rules European wide.

To overcome this problem for visual grading, the standard EN 1912 was developed, which connects the visual grading rules used by an individual country or a group of countries to the European strength class system. The testing and derivation of characteristic values is done according to the European standards. In principle the assignments are based on full size testing.

There are, however, some critical points in the present assignments, particularly in the allowed growth areas for a visual grade. They are sometimes very large and are not always supported by testing over the whole growth area.

For hardwoods there is a mixture of assignments based on full size specimens and on small clear specimens. Especially some species graded under UK rules are not based on

full size specimens. A part of the UK data for small clear specimens is summarized in Dinwoodie (2000).

It can be questioned whether it is possible to make the connection between mean values of small clears and the full size data for tropical hardwoods. In the past, the quality of the full size timber was closer to clear wood than what presently comes on the market. Also the most important visual characteristic responsible for the reduction of strength, the grain angle deviation, is very difficult to measure.

2.7 Discussion

- Over the last century (tropical) hardwoods have been used in Europe in structural applications where high strengths and high durability are required. In The Netherlands this meant application in hydraulic structures like bridges, lock gate doors, fenders etc.
- The number of (tropical) hardwood species to be used in structural applications will increase due to the use of timber from sustainably managed forests.
- For tropical hardwoods, only relatively small amounts of test data are available for timber of structural sizes.
- Because grain angle deviation, which is the governing strength reducing characteristic for (tropical) hardwoods, is difficult to measure, a larger variation between samples fulfilling the same visual grade can be expected than for softwoods.
- Very little is known about the effect of size and moisture content on the strength and stiffness properties on tropical hardwoods of structural sizes.
- One of the biggest problems in assigning strength properties to structural timber is to define the population the tested timber is representative for. This is subject of ongoing research and is not solved yet.
- To be able to develop species independent strength models, it is therefore necessary to be able to predict the variability in strength properties of the entire population of timber.
- Because of the variability in strength properties and the relatively low number of pieces in a strength grade, the choice of the statistical method to determine the 5% fractile of a sample or a number of samples greatly affects the outcome.

Materials and methods

3.1 Materials

To be able to develop species independent strength models, the developed models have to be derived and tested with data from non-destructive and destructive experiments. For this purpose, a large number of tropical hardwood species has been tested in the Netherlands in the recent years in cooperation with the industry. Also, a smaller number of temperate hardwoods has been tested. Furthermore, a dataset of European softwoods is available to derive species independent strength models. Although the most promising application of species independent strength grading is foreseen for tropical hardwoods, temperate hardwoods and softwoods are included to test the derived models also on these datasets.

Dataset of tropical hardwoods

In table 3.1, the collection of tropical hardwoods is given.

Table 3.1. Dataset of tested samples of tropical hardwoods

Sample ID	Species trade name	Botanical name	Origin	Dimensions	n=
AV1	angelim vermelho	<i>Diniza excelsa</i>	Brazil	60 x 150	40
AV2	angelim vermelho	<i>Diniza excelsa</i>	Brazil	60 x 150	49
AV3	angelim vermelho	<i>Diniza excelsa</i>	Brazil	60 x 150	50
AV4	angelim vermelho	<i>Diniza excelsa</i>	Brazil	60 x 150	57
AV5	angelim vermelho	<i>Diniza excelsa</i>	Brazil	60 x 150	57
CUM1	cumaru	<i>Dypterix spp.</i>	Brazil	65 x 155	40
CUM2	cumaru	<i>Dypterix spp.</i>	Brazil	50 x 105	44
CUM3	cumaru	<i>Dypterix spp.</i>	Brazil	50 x 130	47
CUM4	cumaru	<i>Dypterix spp.</i>	Bolivia	55 x 155	40
CUM5	cumaru	<i>Dypterix spp.</i>	Peru	55 x 155	49

MAS1	massaranduba	<i>Manilkara bidentata</i>	Brazil	50 x 150	41
MAS2	massaranduba	<i>Manilkara bidentata</i>	Brazil	40 x 100	54
MAS3	massaranduba	<i>Manilkara bidentata</i>	Brazil	60 x 145	51
MAS4	Massaranduba	<i>Manilkara bidentata</i>	Brazil	60 x 150	55
MAS5	massaranduba	<i>Manilkara bidentata</i>	Brazil	50 x 100	48
AZ1	azobé	<i>Lophira alata</i>	West Africa	50 x 150	46
AZ2	azobé	<i>Lophira alata</i>	Cameroon	65 x 150	79
AZ3	azobé	<i>Lophira alata</i>	Cameroon	50 x 110	30
GR1	greenheart	<i>Ocotea rodiaei</i>	Surinam	50 x 110 , 50 x 150	20
GR2	greenheart	<i>Ocotea rodiaei</i>	Guyana	50 x 110	43
GR3	greenheart	<i>Ocotea rodiaei</i>	Guyana	70 x 160	27
GR4	greenheart	<i>Ocotea rodiaei</i>	Guyana	30 x 75	103
OK1	okan /denya	<i>Cylicodiscus gabunensis</i> <i>Harms</i>	Ghana	50 x 150	42
OK2	okandanya	<i>Cylicodiscus gabunensis</i> <i>Harms</i>	Cameroon	65 x 150	44
OK3	okan /denya	<i>Cylicodiscus gabunensis</i> <i>Harms</i>	Cameroon	50 x 100	50
KA1	karri	<i>Eucalyptus diversicolor</i>	South africa	60 x 150	38
NA1	nargusta	<i>Terminalia amazonia</i>	Honduras	80 x 150	41
PI1	piquia	<i>Caryocar spp.</i>	Brazilie	70 x 150	39
VI1	vitex	<i>Vitex spp.</i>	Solomon Islands	105 x 150	40
BAS1	basralocus	<i>Dicorynia guianensis</i> Amsh	Suriname	50 x 150	37
BAN1	Bangkirai	<i>Shorea spp.</i>	Indonesia	40 x 100, 50 x 75, 75 x 150	94
SV1	sucupira vermelho	<i>Andira spp.</i>	Brazil	50 x 130	54
CR1	castana rosa	<i>Pouteria oppsitifolia</i>	Brazil	60 x 150	50
LA1	louroa marela	<i>Ocotea spp.</i>	Brazil	60 x 155	50
LF1	louro faia	<i>Euplassa spp.</i>	Brazil	60 x 150	48
PU1	purpleheart	<i>Peltogyne spp.</i>	Brazil	60 x 150	45
TV1	tauari vermelho	<i>Cariniana spp.</i>	Brazil	75 x 145	46
FA1	favinha	<i>Entorolobium spp.</i>	Brazil	63 x 150	50
SA1	sapupira	<i>Hymenolobium spp.</i>	Brazil	67 x 160	48
FP1	favinha prunelha	<i>Pseudopiptadenia spp.</i>	Brazil	65 x 165	50
BIL1	bilinga	<i>Nauclea diderrichii</i>	Cameroon	65 x 150	49
BIL2	bilinga	<i>Nauclea diderrichii</i>	Cameroon	50 x 100	44
EV1	evuess	<i>Klainedoxa gabonensis</i> Pierre	Cameroon	50 x 95	50
EV2	evuess	<i>Klainedoxa gabonensis</i> Pierre	Cameroon	65 x 150	49
TA1	tali	<i>Erythrophleum ivorense</i>	Cameroon	65 x 150	44
TA2	tali	<i>Erythrophleum ivorense</i>	Cameroon	50 x 100	46
Total					2218

Dataset of temperate hardwoods

In table 3.2 the collection of temperate hardwoods is given.

Table 3.2. Dataset of tested samples temperate hardwoods

Sample ID	Species trade name	Botanical name	Origin	Dimensions	n=
O1	oak	<i>Quercus robur</i>	Poland	80 x 150	39
O2	oak	<i>Quercus robur</i>	Germany,France	40 x 100, 50 x 75, 75 x 150	117
R1	robinia	<i>Robinia pseudoacacia L.</i>	Hungary	50 x 150	39
C1	chestnut	<i>Castanea sativa</i>	Italy	50 x 100, 80 x 80	292
				Total	487

Dataset of European softwoods

In table 3.3 the collection of European softwoods is given.

Table 3.3. Dataset of tested samples European softwoods

Sample ID	Species trade name	Botanical name	Origin	Dimensions	n=
S1	spruce	<i>Picea abies</i>	Belgium	60 x 125, 75 x 150	288
S2	spruce	<i>Picea abies</i>	South-Germany	36 x100, 36 x 150, 50 x 100, 60 x 150	180
S3	spruce	<i>Picea abies</i>	Germany-Czech Republic	50 x 150, 70 x 173, 95 x 245, 145 x 285	243
S4	spruce	<i>Picea abies</i>	North-Italy	40 x 122, 53 x 116, 60 x 146	271
S5	spruce	<i>Picea abies</i>	Estonia	50 x 100, 64 x 150	103
S6	spruce	<i>Picea abies</i>	Latvia	45 x 110, 64 x 150	104
S7	spruce	<i>Picea abies</i>	Sweden	50 x 118	50
S8	spruce	<i>Picea abies</i>	Russia,Vologda	45 x 78	49
D1	douglas	<i>Pseudotsia menziesii</i>	Netherlands	78 x 100, 70 x 135, 70 x 170, 75 x 190	356
L1	larch	<i>Larixspp</i>	Netherlands	75 x 150	40
L2	larch	<i>Larix siberica</i>	Russia	50 x 70, 75 x 200, 75 x 200	100
				Total	1784

3.2 Test methods

3.2.1 Methods for testing of mechanical and physical properties

In this section, the test set-ups used for the determination of mechanical and physical properties is described. In principle, the test set-ups as described in the European test standard EN 408 are followed together with the provisions of EN 384, except where they were not suitable for the tested dataset.

Bending strength test set-up:

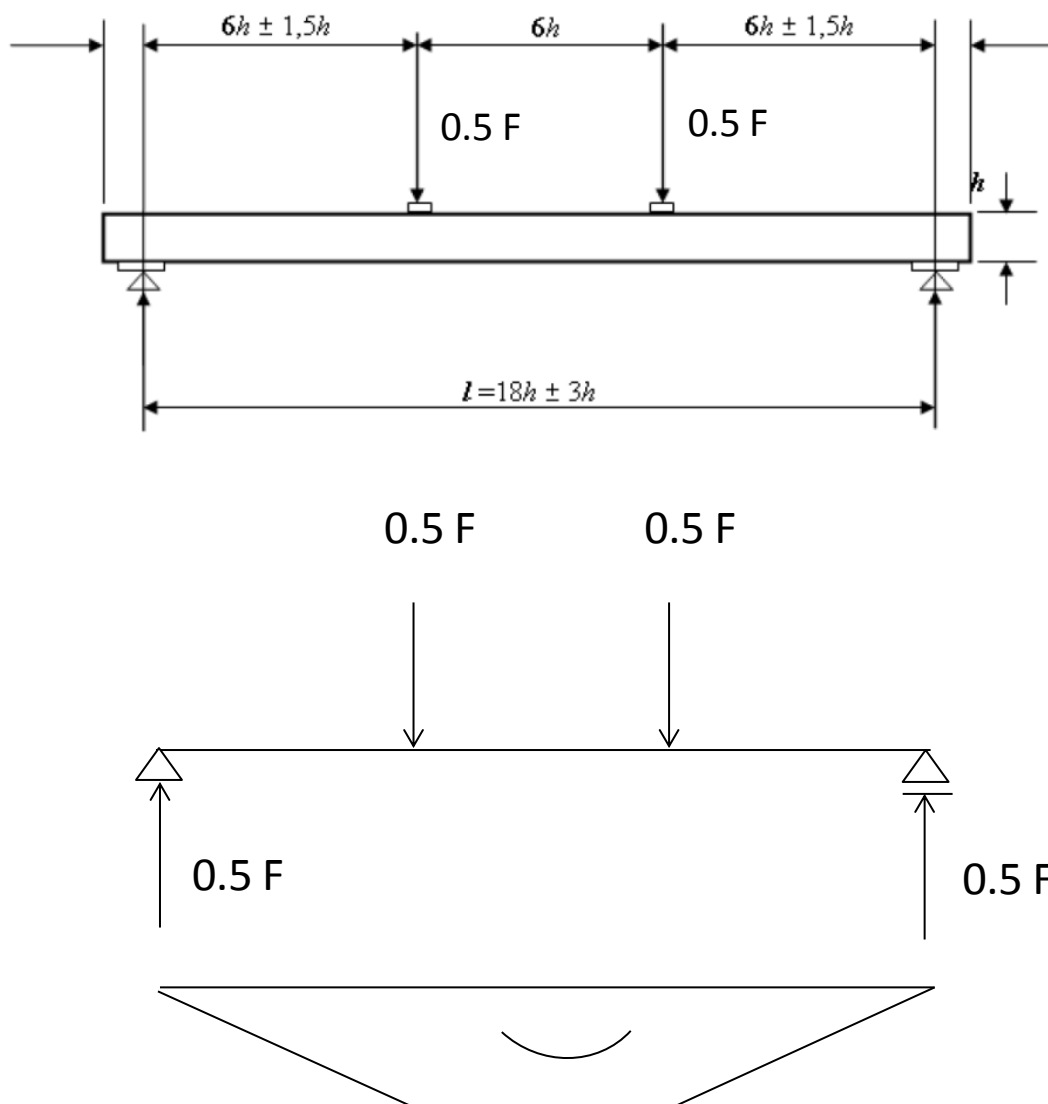


Figure 3.1. Test set-up for determining the bending strength from a four point bending test according to EN 408 (above) and the mechanical scheme for the test set-up (below).

For the bending tests, a four-point bending test with a span of 18 times the depth and the point loads at $6h$ ($=a$) from the supports was applied. For softwoods, mostly an overlength of 1 meter on both sides was used. The critical section, being the position with the

governing visual defect, was placed in the center part between the two load heads, when possible. For tropical hardwoods, the critical section is difficult to visually assess. Therefore, for tropical hardwoods, the total length of the pieces used was the span length plus an overlength of 1 or 2 times of the depth.

The top side of the test pieces in the test set-up was randomly selected. The load head speed was chosen in such a way that the pieces broke in approximately 5 minutes.

The bending strength is then calculated with:

$$f_m = \frac{3 Fa}{bh^2} \quad (3.1)$$

Where F is the total maximum load (the summation of the two point loads) and a the distance from the support to the point load, b and h are thickness and depth respectively.

The adjustments for size according to EN 384 were not followed. The reason is given in section 2.2. Because the size effect could depend on the grading method, and therefore on the parameters used in the strength predicting models, this effect for the models developed in this thesis will be investigated in chapter 5.

The adjustment for moisture content according to EN 384 was not followed either. Before testing, the pieces were conditioned at 65% relative humidity and 20 °C for 2 weeks. According to EN 384, no adjustment for the bending strength should be carried out when the moisture content differs from 12%. However, whether this is correct for tropical hardwoods is questionable. Softwoods are normally tested at 12% moisture content. For tropical hardwoods this is usually higher. Because the reference moisture content for which the strength properties are given is 12% m.c., the values have to be readjusted to this m.c. In chapter 5, the adjustments for moisture content will be derived for the used material.

Local Modulus of Elasticity:

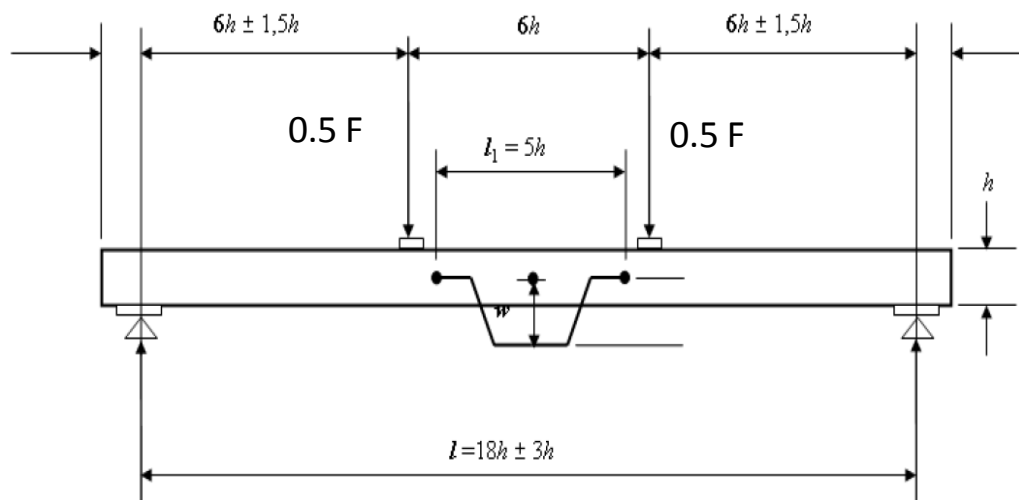


Figure 3.2. Test set-up for the determination of the local modulus of elasticity according to EN 408.

The test configuration is the same as that for the bending strength. The deflection of the test pieces in the centre over a distance of $5h$ within the point loads (where a constant moment occurs) in the neutral layer is measured by means of LVDTs (Linear Variable Differential Transformer) at both sides of the test piece. The local modulus of elasticity is then calculated with structural mechanics according to the following formula

$$MOE_{loc} = \frac{al_1^2(F_2 - F_1)}{16I(w_2 - w_1)} \quad (3.2)$$

Where $a = 6h$ (the distance from the supports to the point loads), I is the second moment of inertia, $l_1 = 5h$, F_2 and w_2 are the total force and the deflection at 40% of the maximum load and F_1 and w_1 are the total force and the deflection at 10% of the maximum load.

This derived value is called the local modulus of elasticity because it is determined by measurements on a limited part of the test piece.

No size effect is expected for the Modulus of Elasticity (MOE). For the adjustments to 12% moisture content the provisions of EN 384 will not be followed, but they will be derived in chapter 5.

Global Modulus of Elasticity:

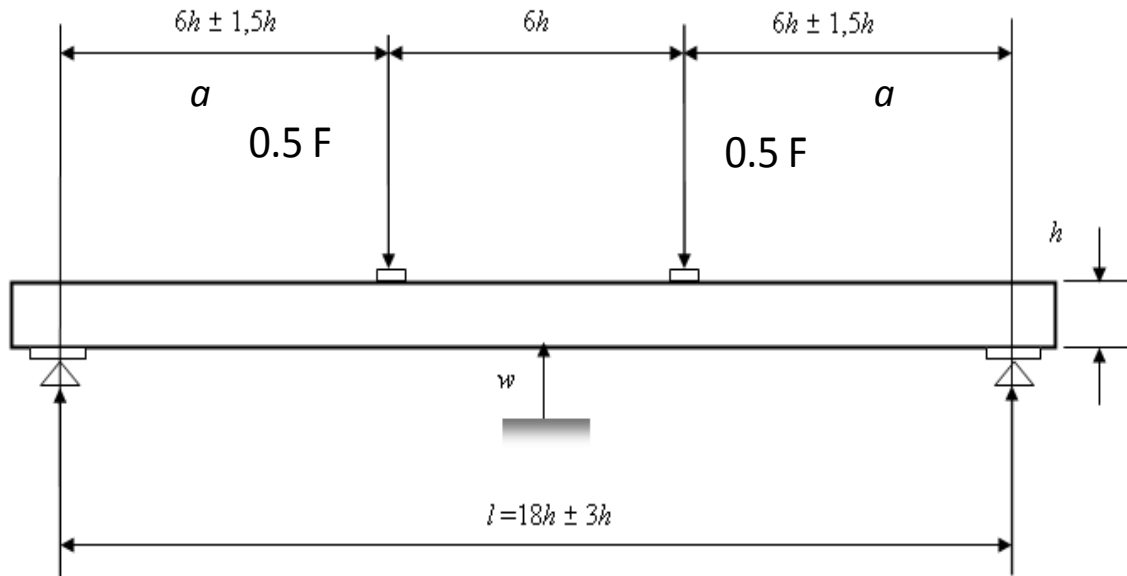


Figure 3.3. Test set-up for the determination of the global modulus of elasticity according to EN 408.

The test configuration is the same as that for the bending strength and the local modulus of elasticity. The difference with the local modulus of elasticity is the position of the measurement of the deflection. In this case, the deflection is measured at midspan on the test piece relative to the supports. The deflection of the test piece was measured at the bottom of the test pieces with an optical laser as indicated in figure 3.3. The measured

deformation includes any local indentations that might occur at the supports and loading points, and deformations of the supports themselves. Small steel plates of lengths not greater than half the depth of the test piece were inserted between the piece and the loading heads or supports to minimize local indentations. These steel plates were also present for the tests to determine the bending strength and the local modulus of elasticity. Usually, all deformations and forces to determine the local and global modulus of elasticity, were recorded in one test. The global modulus of elasticity is then calculated with structural mechanics according to the following formula:

$$MOE_{glob} = \frac{l^3(F_2 - F_1)}{bh^3(w_2 - w_1)} \left[\left(\frac{3a}{4l} \right) - \left(\frac{a}{l} \right)^3 \right] \quad (3.3)$$

Where $a = 6h$ (the distance from the support to the point load), F_2 and w_2 are the total force and the deflection at 40% of the maximum load and F_1 and w_1 are the total force and the deflection at 10% of the maximum load.

Equation (3.3) only takes into account the deformation caused by bending, although there is also shear deformation. As a consequence of neglecting the deformation due to shear, the global modulus of elasticity is lower than the local modulus of elasticity. However, using standardised shear modulus values cannot explain the differences found. A purely experimental relationship was presented in Ravenshorst and Van de Kuilen (2009), and for the material from section 3.1 the relationship will be presented in chapter 4.

The local *MOE* is the *MOE* due to pure bending and this value is used by the engineer in design calculations. Therefore, when only the global *MOE* is measured (which is easier to perform) a conversion equation to the local *MOE* is required.

Dynamic Modulus of Elasticity:

For grading purposes, in practice the Modulus of Elasticity can be measured in different ways. For instance, in bending machines the flatwise (with the small side of the piece placed in the direction of the load head) Modulus of Elasticity is calculated from the deformations in a three-point bending set-up over a span of normally 900 mm. The piece is fed through the machine and the lowest calculated flatwise *MOE* over the length of the beam is used as grading parameter. Nowadays, most grading machines use the method of calculating the *MOE* from vibrations measurements. In this thesis, this method is used for determining the *MOE* for grading purposes. Both methods described above are called the dynamic Modulus of Elasticity. The dynamic modulus of elasticity is called dynamic because they are determined in a very short time period while the tested piece is in movement, in contrast with the local and global Modulus of Elasticity which are called static *MOEs*. The static *MOE* is determined in a laboratory test set-up with a target testing time to failure of about 300 seconds. With dynamic methods, the *MOE* can be determined with measurements that take place in a very short time. In this thesis, the dynamic method of introducing stress waves and measuring the frequency response is used. The actual

measurements take place in less than 0.2 seconds. When in this thesis the dynamic *MOE* is mentioned, the *MOE* retrieved by introducing stress waves is meant.



Figure 3.4. Test set-up for MOE_{dyn}

The dynamic modulus of elasticity MOE_{dyn} is based on vibration measurements of a beam in which a longitudinal stress wave is initiated. In the beam in figure 3.4 the shape of a measuring signal of the acceleration of a beam end in time is drawn. From these vibration measurements the first natural frequency is determined by using a Fourier Transformation. The MOE_{dyn} can be calculated according to the following physical law:

$$MOE_{dyn} = \rho c^2 \quad (3.4)$$

With ρ as the density in kg/m^3 and c as the wave speed in m/s . The unity of MOE_{dyn} is then $\text{kg}/(\text{m} \cdot \text{s}^2)$. This can be rewritten as $(\text{kg} \cdot \text{m}/\text{s}^2) \cdot (1/\text{m}^2)$, which is N/m^2 . The normal unity for the *MOE* is in N/mm^2 ; therefore, value the found in N/m^2 has to be multiplied by $1\text{E-}6$.

This E_{dyn} is a general property of solid materials and its applicability on wood was already known in the first half of the 20th century (Kollmann and Cote, 1968). Görlacher (1990) showed that this principle could also be used in strength grading of structural timber. After that, industrial application has been proven in practice and nowadays most strength grading machines make use of this measurement technique. Theoretically, the beam should be free from supports, but practice has shown that when the beam is supported locally by rubber or wooden sticks, the difference with the free supported beam can be neglected compared to the total amount of uncertainties involved in the grading process.

The wave speed can be calculated from the first natural frequency determined from a Fourier Transformation of the measurement data, which can be accelerations or displacements, measured with an appropriate sensor on either cross end of the beam. The wave speed can be calculated with:

$$c = 2fl_b \quad (3.5)$$

Where f is the first natural frequency in Hz, and l_b is the length of the beam in meters.

The adjustments for moisture content for the dynamic *MOE* that are used in thesis will be derived in chapter 5.

For this thesis, the MTG handheld from Brookhuis Micro Electronics was used to perform the measurements for the calculation of the dynamic *MOE*. Some softwood batches were also measured in an in-line version with the same principle, the mtgBatch from Brookhuis

Micro Electronics. Both systems are approved to grade structural timber with CE-marking in compliance with the European standard EN 14081-1.

Moisture content

In EN 408 it is stated that test pieces should be conditioned at an environment of 20 (+/- 2)°C and 65(+/-5) % relative humidity until they attain a constant mass. For most softwoods, in this environment the timber achieves a moisture content close to 12%. However, tropical hardwoods are mostly used and supplied with a high moisture content and therefore also tested with this high moisture content. To prevent distortions due to shrinkage and drying cracks, tropical hardwoods are stored either outdoors under a cover or in a climate chamber at 20°C and 85% relative humidity. Before testing, the specimens are kept in an environment at 20°C and 65% relative humidity for two weeks.

In accordance with EN 13183-1, the moisture content has been determined on a section free from knots and resin pockets with a length of 25 mm, which is often called the oven-dry method. The test pieces are dried at 103°C until the mass is stable (then the moisture content is 0%). The moisture content at the test can then be calculated from the wet and dry mass with:

$$m. c. = 100 \frac{mass_{wet} - mass_{dry}}{mass_{dry}} \quad (3.6)$$

The moisture content is therefore expressed as the amount of moisture (water) in the piece related to the dry mass and is a percentage measure. This means that the moisture content can have a value higher than 100%, which can be the case for softwoods in the standing tree and directly after felling.

Density

The density is determined by dividing the mass of each piece by its volume. In this thesis the density is used as a grading parameter. According to EN 408, the density of the test piece has to be determined on a section taken from the test piece. For structural timber, this has to be from a full cross section, free from knots and resin pockets. A normal length of this cross section is 25 mm. This density has to be used in the strength class assignments.

According to EN 384 it is also permitted to determine the density from the mass and volume of the whole specimen and to adjust to the density of defect-free specimen by dividing it by 1.05.

For strength class assignments, the density in this thesis is determined as follows:

- For softwoods and temperate hardwoods, the density is determined from a knot-free section of 25 mm length, if available. Otherwise, the density is determined by dividing the mass by the volume of a piece divided by 1.05.
- In tropical hardwoods, knots are very rare. The density is determined by dividing the mass by the volume of a piece without any further correction.

Because the reference density in the strength class tables to be used by the engineer in design calculations is given at 12% moisture content, the density has to be adjusted to this reference moisture content.

The density at a certain moisture content is

$$\rho_{mc} = \frac{G_{m.c}}{V_{m.c}} \quad (3.7)$$

With $G_{m.c.}$ as the mass of the piece and $V_{m.c.}$ as the volume of the piece.

When the moisture content of a piece of timber changes, then two effects occur:

- The dimensions of the piece of timber changes due to shrinkage or swelling by ΔV in %.
- The mass of the of the piece of timber changes by ΔG in %.

The target moisture content is normally lower than the measured moisture content. Then, when ΔV and ΔG are positive, the relationship between the density at the target moisture content and at the measured moisture content is:

$$\rho_{mc,measured} = \rho_{target,mc} \frac{(1 + \frac{\Delta G}{100})}{(1 + \frac{\Delta V}{100})} \quad (3.8)$$

The density at a target moisture content can be formulated according to equation (3.9)

$$\rho_{target,mc} = \rho_{mc,measured} \frac{(1 + \frac{\Delta V}{100})}{(1 + \frac{\Delta G}{100})} \quad (3.9)$$

ΔV can be described as: $\Delta V = \beta_v * (mc_{measured} - mc_{target})$ in %

Where $mc_{measured}$ cannot be higher than the fiber saturation point (FSP), because above that moisture content timber does not change its volume. β_v is the percentage volume change per percent change of the moisture content according to equation (3.10)

$$\beta_v = \beta_r + \beta_t + \beta_l - \frac{\beta_t \beta_h}{100} \quad (3.10)$$

Where:

- β_r . Is the radial shrinkage (in % per percent change of the moisture content)
- β_t . Is the tangential shrinkage (in % per percent change of the moisture content)
- β_l . Is the longitudinal shrinkage (in % per percent change of the moisture content)

ΔG can be described as: $\Delta G = (mc_{measured} - mc_{target})$, in %, because the change in weight is linear with the change in moisture content (the weight of the water).

The density at the target moisture content can then be calculated with the following formula for the moisture content below fiber saturation point:

$$\rho_{target,mc} = \rho_{mc,measured} \frac{(1+0,01*\beta_v*(mc_{measured}-mc_{target}))}{(1+0,01*(mc_{measured}-mc_{target}))} \quad (3.11)$$

The density at the target moisture content can then be calculated with the following formula for the moisture content above fiber saturation point.

$$\rho_{target,mc} = \rho_{mc,measured} \frac{(1+0,01*\beta_v*(mc_{FSP}-mc_{target}))}{(1+0,01*(mc_{measured}-mc_{target}))} \quad (3.12)$$

In Rijdsdijk and Laming (1994) the fiber saturation points and the shrinkage coefficients are given for 145 softwood and hardwood species. In this thesis, as a mean approximation for all wood species, a fiber saturation point of 25% m.c. is used. As an average value for β_v a value of 0.5% per percent moisture content change is used as approximation for all species for moisture contents between 12% and 25%.

3.2.2 Methods for measuring visual characteristics

In appendix B it is listed which visual characteristics are addressed in the visual grading standard for tropical timber. The main strength reducing characteristics are:

- Knots
- Grain angle deviation (or slope of grain)
- Fissures
- Compression failures
- Wane

In principle, the first two, knots and grain angle deviation can be quantified and used to make models to assign pieces into different strength classes. Fissures, compression failures and wane reduce the strength regardless of which strength class the timber is assigned to when their influence is greater than the influence of knots and grain angle deviation. Therefore, grading standards limit the occurrence of these characteristics to a level where they are assumed to have no influence on the strength. For fissures, limits are given in length and depth, but compression failures and wane are not allowed at all in the Dutch grading standard NEN 5493 for hardwood timber to be used in hydraulic structures. The exact magnitude of fissures, compression failures and wane is therefore not recorded for the investigated timber pieces. They are only assessed as acceptable or not. In this section, some examples are given. In appendix B pieces are listed that are not acceptable to be used in structures and were removed from the analysis.

Knots

The influence of knots is calculated as a ratio of their size perpendicular to the beam axis with the thickness or height of the beam. In figure 3.5 examples are given. The knot ratio (SKR) is then:

$$\text{single knot ratio (SKR)} = \max\left(\frac{d_1}{h}, \frac{d_2}{t}\right) \quad (3.13)$$

The knot with a maximum value of the knot ratio within the critical test area is the recorded knot ratio for a beam.

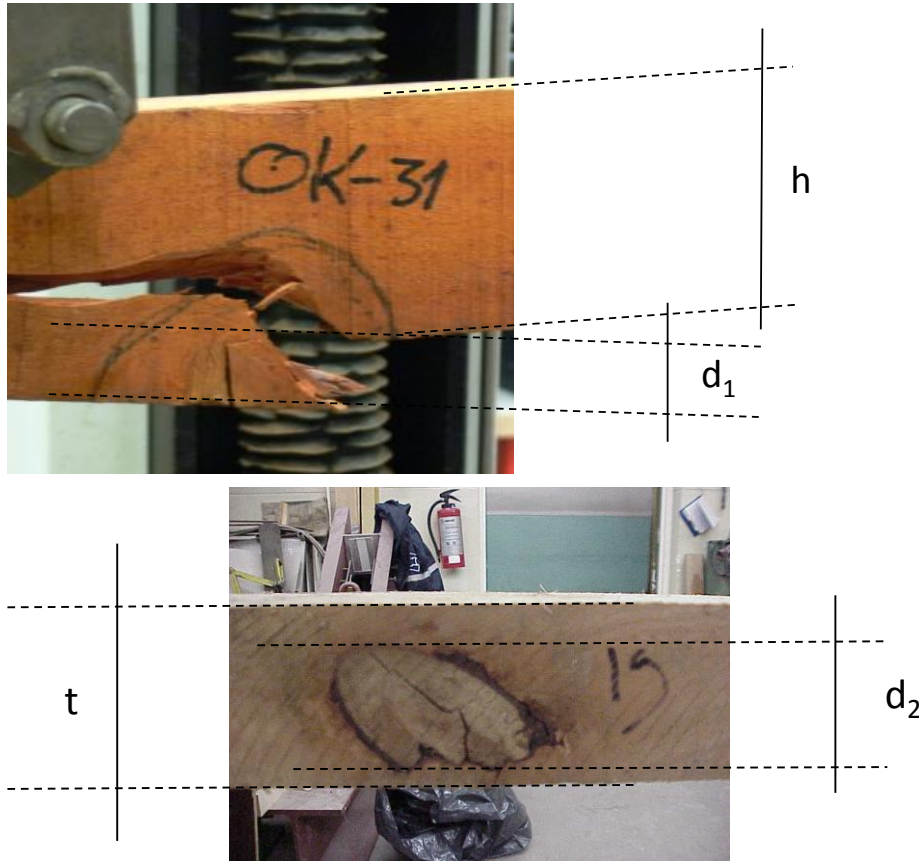


Figure 3.5. Maximum knot on the height (above) or on the thickness of the beam (below)

For one dataset, softwood species douglas with ID D1, the group knot ratio was recorded. In this case, over a part of 150 mm of the length of the beam, the knot sizes perpendicular to the beam axis were summed (overlapping parts were counted only once) and divided by the circumference.

$$\text{Group knot ratio (GKR)} = \max\left(\frac{d_1+d_2+\dots+d_i}{2h+2t} \text{ over 150 mm length} \right) \quad (3.14)$$

In figure 3.6, an example is given. When for instance knot d_1 is also visible on the other side of the beam, this size has to be summed also. At the top thickness of the beam two knots are visible. The overlapping size over the thickness is counted.

For a beam the GKR part of 150 mm in the critical section with the maximum value of group knot ratio is recorded.

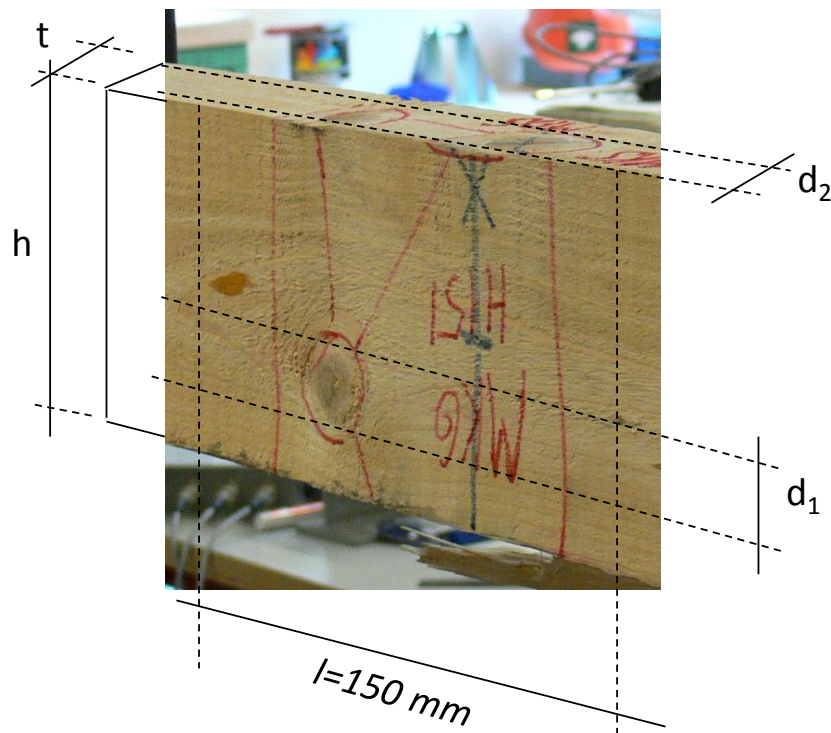


Figure 3.6. Beam with multiple knots over a length of 150 mm.

Grain angle deviation or slope of grain

In practice, for grading purposes the grain angle deviation has to be measured on the beam. In figure 3.7 this is denoted as α_2 . Normally, in grading standards, the slope of grain is used. This is the tangent of the angle, so for α_2 it is h_2/l_2 . This slope of grain, however, is very difficult to measure in practice. The slope of grains (SoG) given in this thesis are measured after the destructive test. The recorded slope of grain in figure 3.7 after the destructive test was h_1/l_1 .

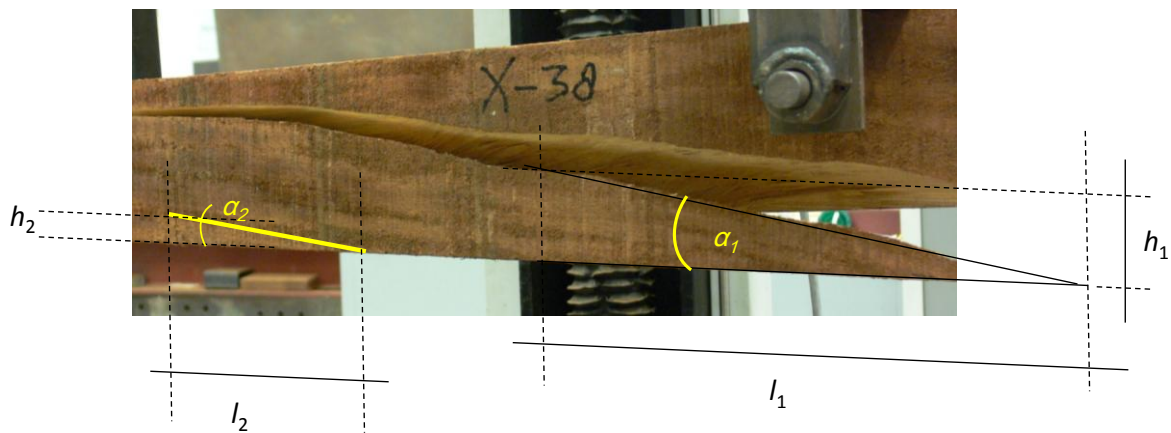


Figure 3.7. Measurement of slope of grain before and after the destructive test.

For the data listed in this thesis the slope of grain measured in the plane parallel to the direction of the load is given. Of course the slope of grain is a 3D phenomena, and to address this, also the slope of grain in the plane perpendicular to the direction of the load should be measured. However, as will be explained later, measuring the slope of grain visually is a difficult task and 3D slope of grain measurements on limited samples were not very successful (in terms of improving the correlation of the slope of grain with the bending strength).

Compression failures

Compression failures give an unpredictable reduction of the strength and a beam containing compression failures is not allowed in structural timber. This has to be detected by a visual override on the timber. At the top of figure 3.8 an example of a compression failure in a timber beam detected by visual inspection is shown. This beam should not be used in structures and is not included in the analysis. At the bottom of 3.8 the effect of a compression failure in a destructive test is shown. In this case, the unpredictable influence led to a very low bending strength.



Figure 3.8. Compression failure in a timber beam (left) and the effect of a compression failure in a bending test (right).

Fissures

Fissures are allowed when they are limited in length along the beam axis and the depth through the thickness. Large fissure might affect the bending strength, but could also affect for instance the shear strength. The fissures should not pass through the thickness (half the thickness is used as a maximum in standard EN 14081-1) and be limited in length. In figure 3.9, examples are given of fissures that are not allowed.



Figure 3.9. Examples of fissures that are not allowed in structural timber. On the left, an end fissure passes through the thickness. On the right, a fissure running out of the top side of the beam.

Wane

Wane gives a reduction of the nominal cross section, because the beam is sawn from a part of the stem near the bark. The reduction in cross section gives a reduced section modulus and therefore a reduced strength capacity compared with a full cross section. Another point is that the presence of wane also means that non-durable sapwood is present, which is not allowed in application for hydraulic structures. Figure 3.10 gives an example of wane in a timber beam.



Figure 3.10. Wane in a timber beam.

Other anomalies.

Timber with any other anomalies that might affect the strength should not be used as structural timber. An example is given in figure 3.11.



Figure 3.11. Anomaly in a timber beam that might affect the strength in an unpredictable way. Timber with these kinds of anomalies should not be used as structural timber

3.3 Statistical method to determine the 5th percentile values of visual strength grades of a timber species based on N samples

In section 2.5, the procedures for determining the 5% fractile of visual strength grades based on more than one sample were discussed. However, as they were presented, they are not suited to be used to process the test results tropical hardwoods with more than one sample.

A reason for this is that especially for tropical timber, it can be questioned whether the samples can be regarded as a homogeneous population. Therefore, this uncertainty should be addressed in the determination of a timber species. A timber species is hereby defined as the trade name under which timber is brought on the market. A timber species might be timber of the same genus, but from different species. The determination of the species is very difficult when it must be based on the timber pieces themselves (instead of the tree). This uncertainty should be investigated. The current k_s factor in EN 384 is not suited to describe this uncertainty because it is based on a random sampling from a homogeneous population. In this section, a $k_{s,tn}$ (tn =trade name) will be derived based on experimental data of timber with trade names cumaru and massaranduba.

This factor $k_{s,tn}$ should be applied to the 5% fractile of the investigated dataset and depends on the number of samples.

The objective of this thesis is to compare the suitability of visual grading and machine grading for species independent grading. Therefore, the methods should be comparable. To make them comparable, the following principle is defined: The characteristic values of each batch graded with either visual or machine grading should meet the required characteristic values of the strength class assigned to this batch. As a consequence of this principle, then, for the determination of the 5% fractile of the tested samples, the 5% fractile value of the weakest samples that can be considered as a homogeneous population

has to be used. (It is noted that this approach differs from the standardised procedure in Europe for visual grading, where an average 5% fractile is used).

Therefore the following methods have to be developed:

- Determination of the weakest homogeneous samples
- Determination of the 5% fractile of the homogeneous samples
- Determination of the factor $k_{s,tn}$ that has to be applied on this 5% fractile.

Determination of the weakest samples that are combined as a homogeneous sample

In the pre-standard ISO/CD 12122-1 it is proposed to use the chi-squared test to determine whether samples can be regarded as a homogenous population. However, this standard gives an indication for a minimum of 100 pieces in each sample, where in practice a lower number of pieces in a sample is used. Furthermore, ISO/CD 12122-1 uses the number of pieces in each sample below the 5% fractile of the total sample to evaluate and compare them with the expected values. This can lead to a very low number or low percentage values and even numbers or percentages of zero. According to De Vocht (2006) the following requirements to perform a chi-square test have to be fulfilled:

- All expected cell frequencies must be equal to or higher than 1.
- A maximum of 20% of the cell frequencies may be between 1 and 5.

To overcome this problem, not the number or percentage of pieces below the 5% fractile of all samples, but the percentage below the mean value of all pieces will be used to compare the samples.

The following procedure is followed.

- For all samples, calculate the 5% fractile of the samples with a 75%- confidence level based on a normal distribution.
- Rank the samples according to these values. The sample with the lowest 5% fractile is called the weakest sample.
- Determine the mean value of all samples
- Determine the percentage of pieces below the mean value for all samples. These are the observed (O) percentage values.
- The expected percentage values (E) for all samples are the sum of the observed percentage values divided by the number of samples
- The test value Z is calculated according to equation (3.15):

$$Z = \sum_i^N \frac{(O_i - E_i)^2}{E_i} \quad (3.15)$$

Where

O_i is the observed percentage value, E_i is the expected percentage value, and i is the sample number.

- The significance level sig is then calculated as the critical value of the chi-squared distribution for the value of Z with $N-1$ degrees of freedom.

In ISO/CD 12122-1, a significance level of 0.01 is proposed, which means that when the calculated significance is higher than 0.01, the samples are regarded to be able to be combined in the determination of the 5% fractile.

When then the significance level is lower than 0.01, the strongest sample is removed and the procedure is repeated until the significance level is higher than 0.01. The samples included in this test are assumed to be homogenous and will be used to determine the 5% fractile of the total dataset. When also for the two weakest samples the significance level is lower, then only the weakest sample will be used to determine the 5% fractile of all samples.

Determination of the 5% fractile of the combined samples

This is a further elaboration of the approach with the confidence level factor k for one sample assuming a parametric distribution as described in section 2.4.1. The derivation of the value for the confidence level factor k for more than one sample it is assumed that the samples are drawn from one population that is normally distributed.

When more than one sample is taken, the precision of the estimate will increase. To formulate k -factors for the case with more than one sample, we have an estimate of the standard error around the point estimate, and assume that this standard error is normally distributed.

Again, the parametric tolerance limit is defined as:

$$PTL = \bar{x} - ks \quad (3.16)$$

In this formula, the mean value of $\bar{x}$ will tend to μ (the mean of the underlying population) and the mean value of s will tend to σ (the standard deviation of the underlying population). The variance of equation (3.16) can then be calculated with the formulas for error propagation. The variance of the equation is: $var(PTL) = var(\bar{x}) + k^2 * var(s)$. The variance of $\bar{x}$ is σ^2/n and the variance of s is $\sigma^2/(2(n-1))$ (See Sclove, 2005). The variance of the PTL becomes:

$$var(PTL) = \frac{\sigma^2}{n} + k^2 \frac{\sigma^2}{2(n-1)} \quad (3.17)$$

This is only true when $\bar{x}$ and s are independent, which they are not. However, this is not a problem for large samples. So equation (3.17) is an approximation valid for larger samples (sample size larger than 10).

Then the standard error becomes:

$$SE(PTL) = \sigma \sqrt{\frac{1}{n} + \frac{k^2}{2(n-1)}} \quad (3.18)$$

We have N samples and expect that the mean values of the 5% tolerance limits of these samples calculated with $\bar{x} - k_1 s$ tend to the mean of the 5% PTL of the population and that values of the samples are scattered around this value with a standard deviation of $SE/\sqrt{N}$. In that case, the equation 2.19 becomes

$$\text{Prob} \left(\frac{\sum_1^N (\bar{x} - k_1 s)}{N} - k_2 \frac{SE}{\sqrt{N}} \leq \mu - z_p \sigma \right) = \alpha \quad (3.19)$$

Where

k_1 is the value of the normal distribution for PTL limit of the population (=1.65 for the 5% PTL)

k_2 is the value that has to be determined.

We can define

$$\bar{X}_N = \frac{(\sum_1^N \bar{x}_i)}{N} \quad (3.20)$$

And

$$\bar{S}_N = \frac{(\sum_1^N s_i)}{N} \quad (3.21)$$

In this case $k_1 = z_p = 1.65$ ($p=0.05$, but now $\alpha=0.5$ because we want the variation around the mean value of the Tolerance Limit; this gives $k_1 = 1.65$). By taking $\alpha=0.5$ it is assumed that the distribution of $SE(PTL)$ is symmetric, which is not strictly true. However, for sample sizes of $n > 40$, this is regarded as acceptable) and the SE can be estimated by $\bar{S}_N * A$, with A is

$$A = \sqrt{\frac{1}{n} + \frac{k_1^2}{2(n-1)}} \quad (3.22)$$

Then:

$$\text{Prob} \left(\bar{X}_N - \bar{S}_N \left(z_p + k_2 \frac{A}{\sqrt{N}} \right) \leq \mu - z_p \sigma \right) = \alpha \quad (3.23)$$

$$\text{Prob} \left((\bar{X}_N - \mu + z_p \sigma) / \bar{S}_N \leq z_p + k_2 \frac{A}{\sqrt{N}} \right) = \alpha \quad (3.24)$$

$$\text{Prob} \left((\bar{X}_N - \mu + z_p \sigma) / (\bar{S}_N / \sqrt{n}) \leq (z_p + k_2 \frac{A}{\sqrt{N}}) \sqrt{n} \right) = \alpha \quad (3.25)$$

$$\text{Prob} \left((\bar{X}_N - \mu + z_p \sigma) / (\sigma / \sqrt{n}) / (\bar{S}_N / \sigma) \leq (z_p + k_2 \frac{A}{\sqrt{N}}) \sqrt{n} \right) = \alpha \quad (3.26)$$

The term

$$\left(\frac{\bar{X}_N - \mu + z_p \sigma}{\sigma / \sqrt{n}} \right)$$

is the normal distribution with mean $z_p \sqrt{n}$ and standard deviation 1. Then equation 3.26 becomes:

$$\text{Prob} \left(\frac{N(z_p \sqrt{n}, 1)}{(\bar{S}_N / \sigma)} \leq (z_p + k_2 \frac{A}{\sqrt{N}}) \sqrt{n} \right) = \alpha \quad (3.27)$$

The term $(z_p + k_2 \frac{A}{\sqrt{N}}) \sqrt{n}$ then can be described by the non-central distribution:

$$(z_p + k_2 \frac{A}{\sqrt{N}}) = (NCT^{-1}(z_p \sqrt{n}, n-1)(\alpha)) / \sqrt{n} \quad (3.28)$$

k_2 can then be calculated for the situation $N=1$ for a certain n and applied for other N (for $N=1$ k can be calculated with equation 2.20):

$$k_2 = (k - z_p) \frac{\sqrt{1}}{A} \quad (3.29)$$

Then $k_{N,n}$ can be calculated with:

$$k_{N,n} = z_p + k_2 \frac{A}{\sqrt{N}} = z_p + \frac{(k - z_p)}{\sqrt{N}} \quad (3.30)$$

The 5% *PTL* with a 75% confidence of the population for N samples is then calculated with:

$$5\%PTL_N = \bar{X}_N - \bar{S}_N(z_p - k_2 \frac{A}{\sqrt{N}}) = \bar{X}_N - \bar{S}_N k_{N,n} \quad (3.31)$$

For the 5% *PTL* $z_p = 1.65$.

In table 3.4, values for $k_{N,n}$ are given, based on the number of samples (N) and the number (n) of specimens in the sample (for this value of n the sample with the lowest amount of specimens is governing) :

Table 3.4 Values for $k_{N,n}$ depending on the number of samples N and the sample size n

n	N				
	1	2	3	4	5
10	2.08	1.96	1.90	1.87	1.84
20	1.92	1.84	1.81	1.79	1.77
30	1.86	1.80	1.77	1.76	1.75
40	1.83	1.78	1.75	1.74	1.73
50	1.81	1.76	1.74	1.73	1.72
100	1.76	1.73	1.71	1.70	1.70
150	1.73	1.71	1.70	1.69	1.69
200	1.72	1.70	1.69	1.69	1.68
250	1.71	1.69	1.69	1.68	1.68
300	1.71	1.69	1.68	1.68	1.68
500	1.69	1.68	1.67	1.67	1.67
1000	1.68	1.67	1.67	1.66	1.66

Determination of the factor $k_{s,m}$ that has to be applied on the 5% fractile of the homogeneous samples with the lowest strength.

A number of samples are drawn from an inhomogeneous population. It is unclear however whether the weakest possible sample based on the visual grading rules is present in the dataset. Therefore the derived 5% fractile based on the tested dataset must be reduced by a factor addressing this uncertainty to transfer this value to the population. This factor can only be derived from experimental research. Therefore, in this section, the datasets of cumaru (CUM1-CUM5) and massaranduba (MAS1-MAS5) as presented in table 3.1 are used to investigate this. An overview of the strength properties of the tested data is given in chapter 4. The strength properties of table 4.11 are used to investigate the $k_{s,m}$ factor.

The procedure followed was:

- For the timber with trade names massaranduba and cumaru testing programs were simulated. It could be that instead of 5 samples cumaru only 1,2,3 or 4 sample(s) were (was) tested. Those samples could be randomly taken from the 5 possible samples. This means that in total there are $2^5 - 1 = 31$ combinations in which the samples could be tested. For instance, 5 different combinations are possible for 4 samples to be tested in a testing program, 10 different combinations are possible for 3 samples to be tested, etc. The same number of combinations is possible for massaranduba.
- For every combination, the weakest homogeneous subsample(s) was (were) determined with the method presented in this section.

- For every combination, the 5% fractile of this (these) weakest homogeneous subsample(s) was determined with the method presented in this section. The lowest 5% fractile of all combinations is assumed to be the most conservative 5% fractile that can be expected from a testing program for timber with that trade name.
- For all test combinations the 5% fractile of the weakest homogeneous subsample(s) of each combinations is divided by the 5% fractile of the most conservative 5% fractile of all combinations. These are called k_s -ratios.

The calculated k_s -ratios are plotted in figure 3.12.

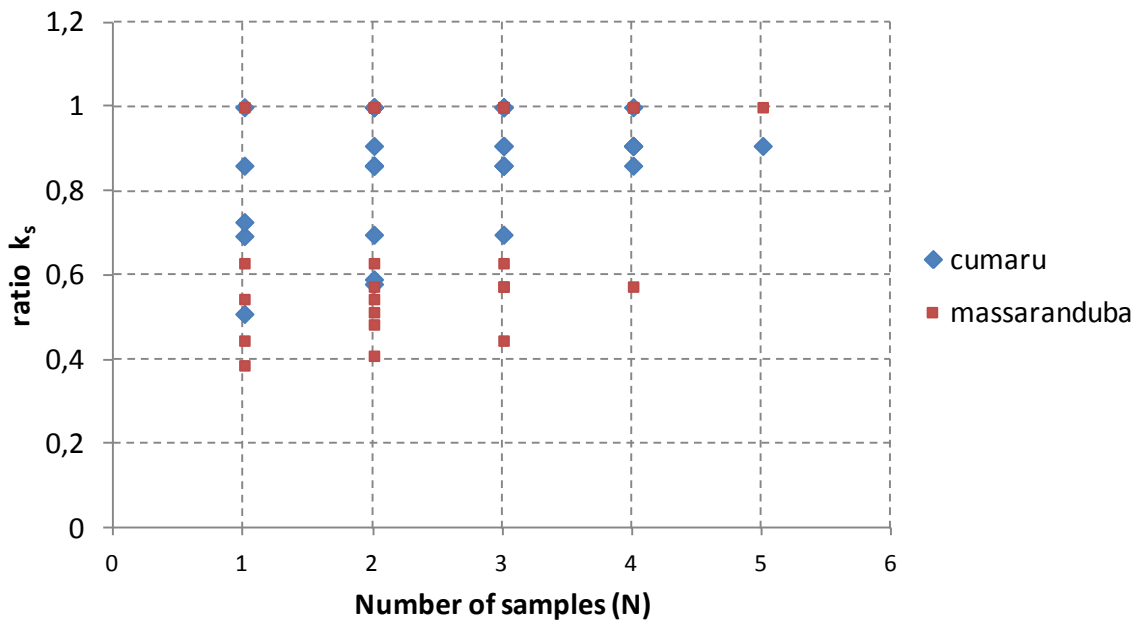


Figure 3.12 . k_s -ratios plotted against the number of samples for cumaru and massaranduba.

A low k_s – value indicates that the 5% fractile found for a test sample combination is much higher than the possible lowest 5% fractile that may occur and therefore should be multiplied by this factor. The lower contour of figure 3.12 should therefore be followed. Equation (3.32) for the k_s factor is proposed. This factor is called $k_{s,tn}$ because it addresses the variation of tree species (s) that might be involved for timber traded under this commercial trade name (tn). The constant 0.38 is taken as the lowest value for N-1. A regression analysis through the lowest points for every gives too high values for N=2 and N-3, therefore the value 0.1 is determined by trial and error to get safe values for these datapoints.

$$k_{s,tn} = 0.1 \ln (N) + 0.38 \quad (3.32)$$

Equation (3.32) is plotted in figure 3.13.

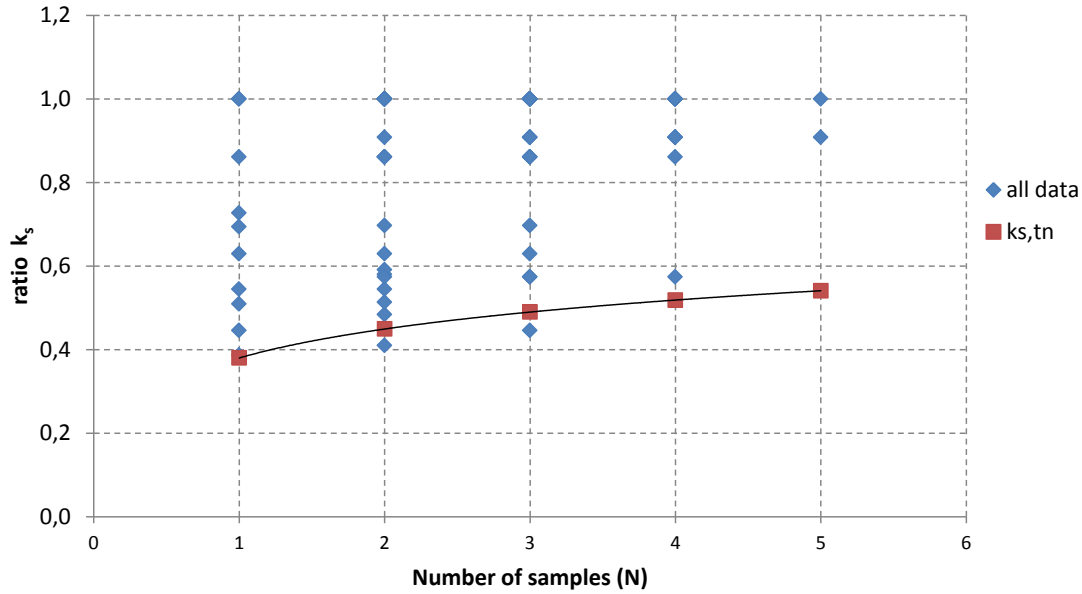


Figure 3.13 . k_s -ratios for all data of cumaru and massaranduba and $k_{s,tn}$ according to equation (3.32) plotted against the number of samples.

Figures 3.12 and 3.13 show that the factor $k_{s,tn}$ becomes higher when more samples are used. However, the fact that the $k_{s,tn}$ differs between the timber with the two trade names means that the uncertainty incorporated (different species, different quality) is not the same. For the investigated trade names massaranduba is governing. For equation (3.32) $k_{s,tn}$ will give a value of 0.61 for 10 samples.

One important aspect which will also be addressed in chapter 4 is that visual grading based on strict limits for the slope of grain, is almost impossible to perform in practice. To ensure a reliable method for visually graded timber based on the slope of grain, a reduction factor $k_{s,tn}$ as presented is required. However, this would mean that for a number of trade names for tropical timber much lower strength values have to be used as compared with the outcome of presently known tests results on timbers with those trade names.

This means that other methods are necessary to strength grade tropical timber. This could be done either by improving the measurement of slope of grain by for instance machines that measure this with laser dots. The other option is to use machine grading based on density and MOE_{dyn} . This last option will be studied further on in this thesis.

3.4 Determination of 5% fractiles based on model properties

3.4.1 Introduction

The methods described in the previous sections are used for samples that are visually graded. The samples are graded according to predefined limits in visual grading rules. Then, based on these samples, the strength class of the species is determined. For machine grading the working method is the other way around. The strength classes to which the

timber has to be graded to are chosen and limit values for machine measurements have to be derived. These limits are called settings. To derive these methods, an extra step is necessary, which is the development of a predicting model. By using a predicting model more knowledge is available of the nature of the variation and in that way the population can be described better. In this section, a method for deriving settings is proposed, which is based on the properties of the prediction model. The properties of the prediction model are the equation for the prediction line for the model values with the standardised tested values and the scatter around this prediction line.

The mechanical properties of timber can be predicted by non-destructive measurements, which principle is used in machine grading. However, the mechanical properties have a certain random error around their predicted values, which has to be accounted for in the derivation of settings for strength grading machines.

The determination of settings for machine strength graded timber in Europe is defined in EN 14081-2. The method is based on the derivation of a prediction model for the destructive strength values by using a predicting indicating property (*IP*) based on non-destructive measurements. The *IP* can be one single predicting property like the MOE, but can also be a value calculated by an equation including several predicting properties. The model is created by combining data of a number of samples originating from a growth area for which the settings will be derived. Limit values (called 'settings') for the *IP* for every strength grade have to be determined. In short, the settings are determined by assigning each single piece to a strength class based on the *IP* of the piece, followed by a verification of the characteristic values. When required values are not reached, the procedure is repeated by adapting the settings until they do.

In Ziethen and Bengtsson (2011), a number of weaknesses in the current standardized procedures are listed. Main problem is the sensitivity for small differences in the number of pieces in the assigned grades. The reason for this is that settings have to be determined based on the existence of data in the assigned strength grades by trial and error. This is explained in figure 3.14.

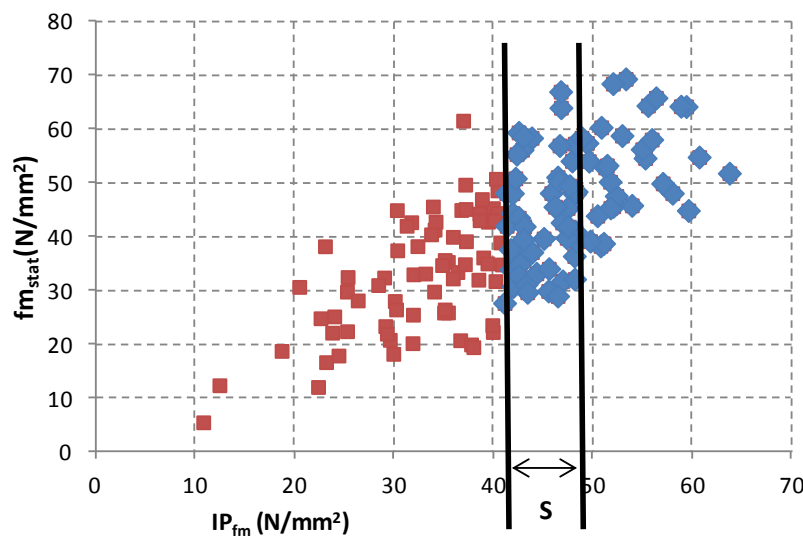


Figure 3.14. Limit values *S* to be found by trial and error for grade C30.

In figure 3.14 the data with bending test values are plotted against the predicted values (IP_{fm}) based on mechanical non-destructive measurements. It is now the objective to find a limit S for IP_{fm} whereby a piece that has an IP_{fm} -value higher than S is assigned to strength class C30. This limit value S is called a setting. When more than one strength class is graded there will more limit values, then called settings. To find the limit value according to the current standardized method the value of S has to be shifted along the horizontal axis in such a way that the 5% fractile of the test pieces assigned to the grade (the blue datapoints) is at least 30.0 N/mm². At present, this has to be done by the non-parametric method of ranking without confidence interval, but the next finding is also valid for any other method described in section 2.4. It turns out that when the limit S in figure 3.14 is shifted from 41.0 N/mm² to 47.0 N/mm² the 5% fractile of the assigned pieces stays just under 30.0 N/mm². When the limit value S is chosen above this value, for instance 47.1 N/mm², then the 5% fractile of the assigned pieces goes up to 36.4 N/mm². This makes the process of deriving settings very sensitive. A major disadvantage is that it does not make use of the information regarding the scatter around the model in a direct way. The objective of this section is to present a method that takes the properties of the prediction model into account.

Statistical distribution of timber strength grades

The statistical distribution of timber strength grades has been studied by a number of authors. Besides the shape of the distribution, also the difference between the target characteristic values and the obtained characteristic values was studied. The target characteristic value is the value required for the strength class the grade is assigned to (for Europe the target characteristic values are given in EN 338). The obtained characteristic values are the 5% fractiles calculated from the data assigned to a specific strength class. Sørensen and Hoffmeyer (2001) analysed a number of datasets to evaluate which distributions fitted best to the bending strength values of the assigned grades of mostly Norway spruce. They found that 2-parameter Weibull and normal distributions gave the best fit for the data, but that the obtained 5% fractiles varied significantly from the required target values. Van de Kuilen and Blass (2005) found that for the tropical hardwood species azobé a normal distribution fitted best. These results are in contradiction with the lognormal distribution to describe timber properties, which is proposed by the JCSS Model code (Anonymous, 2006).

Chui et al. (1991) studied the influence of sample size on the accuracy of the characteristic values of machine strength grades by simulating samples of different sizes from a dataset of 972 softwood pieces. The settings were derived for 2 grades. The pieces of the simulated samples were graded according to the settings and the 5% fractiles of the assigned grades were calculated. They found that for instance for samples sizes of 100 pieces the 5% fractiles of individual simulated samples could deviate around the mean 5% fractiles of the simulated samples up to 20% for a 90% confidence interval, depending on the method of determining the 5% fractiles.

Pellicane and Bodig (1981) studied the influence of sampling size on the 5% fractiles and also on the whole sample distribution by taking random samples from a known large population (3000 softwood pieces), containing different visual grades and sizes. The graded datasets fitted to a Weibull distribution. They analysed random samples from both ungraded and graded datasets. For a confidence level of 95% they found the following relation for the average error E as a function of sample size n over the whole distribution : $E = \frac{S}{\sqrt{n}}$, with for S a value of 110 for the ungraded samples and around 103 for the graded samples. This means that for graded pieces with a sample size of 100 the expected error is 10.3 percent, for a sample size of 40 pieces 16.3 percent and for 20 pieces 23.0 percent for a confidence interval of 95%. A confidence interval of 95% means that for 95% of test samples the empirical distribution function will be within the error margin of the entire population values. Sample sizes of 40 and even 20 in a strength grade can occur in the present procedure of deriving settings.

Summarizing, it can be concluded from literature (Chui et al, (1991) and Pellicane and Bodig (1981)) that the verification of the 5% fractiles of a population on a random sample with the required characteristic value is a delicate process.

Principles for machine strength grading based on regression analysis

The prerequisite for the derivation of settings for machine strength grading is the existence of a predicting model between the IP and the destructive test data, which can be determined by a regression analysis. A statistical requirement for performing a regression analysis is that regression residuals should have equal variance (Box et al.(1978)). Eurocode NEN-EN 1990 gives some guidance for the statistical determination of resistance models (section D8). However, NEN-EN 1990 gives assumptions that are uncertain to be true for structural timber. For example, NEN-EN 1990 assumes that the resistance function is a function of a number of independent variables, which is not true for timber when the density and MOE are used as prediction parameters. Furthermore the method of NEN-EN 1990 gives a value for the resistance for a specific value of the resistance function. For timber, this would be very uneconomic because of the larger scatter than for instance for steel. Another point is that the shape of the scatter around the regression line is assumed to be known in NEN-EN 1990, which is not clear for structural timber. However, the work in this thesis can be regarded as an elaboration of the principle use of resistance models according to NEN-EN 1990, specifically for structural timber. In this thesis prediction models will be formulated based on linear regression and the scatter around the regression lines that can be expected will be investigated. This will be done in chapter 5.

In the next section a method to determine limit values for the prediction functions are derived based on the principles of linear regression, taking into account the scatter in the model properties. An equal variance of the regression residuals is assumed to explain the principle of the method. In chapter 6 the presented method will be extended by the shape of the regression residuals found in chapter 5.

3.4.2 Derivation of the method

Model basis

The assumption of the proposed method in this section is that mechanical properties of timber can be described by a statistical distribution, whether the population is graded or not. The procedure for deriving settings is based on three principles:

- the distribution of the non-destructive grading parameter values can be described by a known statistical distribution.
- the regression residuals should be normally distributed around zero.
- the regression residuals should have an equal variance.

For further reading about regression analysis reference is made to for instance (Box et al, 1978) and (Freund et al., 1998). The principle of linear regression used in this thesis is explained in appendix A.

The proposed method consists of the following elements:

- a method for the determination of characteristic values and settings
- a method to construct the expected cumulative distribution of the destructive test values of the assigned grades
- a method to construct confidence intervals for the theoretical strength grade distributions.

The accuracy of the estimation of the 5% fractiles for strength depends on the assumptions made for the modeling. To verify the proposed method with test data from assigned grades, an approach has to be chosen that takes the expected variability of the test data into account. Comparing the calculated 5% fractiles of the test values of the assigned grades according to the present standardized methods of section 2.4 with the required values will not take the expected variability into account. It is more useful to verify whether the cumulative distribution of the test data assigned to a strength grade could be a realization of the expected theoretical cumulative distribution. This is the objective of the 3rd element of the proposed method, to construct confidence intervals for the theoretical strength grade distributions. With these confidence intervals, the cumulative distribution of the graded test data is then compared.

In this section, only the (bending) strength will be investigated, because this is the most important parameter, and mostly governing, but the method can also be applied to the Modulus of Elasticity and the density.

To determine the theoretical 5% fractile, the method presented by Ravenshorst and Van de Kuilen (2008, 2010) is extended. The method contains a p -value and a p_{char} -value to evaluate the 5% fractiles of a strength grade, which will be explained in this section. A regression model is derived between the bending test values obtained in static laboratory tests and predicted bending strength values. The predicted bending strength is called the

Indicating Property and is denoted by IP_{fm} . The measured bending strength values are denoted by fm_{stat} . The IP_{fm} values can be based on the value of a single property like the dynamic modulus of elasticity or a combination of properties. In this section it is assumed that IP_{fm} is written as the predicting value of fm_{stat} , and that the relationship between fm_{stat} and IP_{fm} is linear. To achieve this, the definition of IP_{fm} can be a linear equation consisting of one or more parameters, but it can also be a non-linear equation consisting of one or more parameters. The parameter $fm_{stat,mod,i}$ is defined as the parameter for which the mean regression line with $fm_{stat,i}$ coincides with the line $y = x$. To achieve this the relationship between $fm_{stat,mod,i}$ and $IP_{fm,i}$ is described according to equation (3.33). Ideally, A is close to one and B is close to zero.

$$fm_{stat,mod,i} = A \cdot IP_{fm,i} + B \quad (3.33)$$

To be able to meet the assumptions for modeling, the residuals around the regression line between $fm_{stat,i}$ and $fm_{stat,mod,i}$ will have to follow a normal distribution around zero and have an equal variance s_e^2 over the entire range of $fm_{stat,mod,i}$ (and therefore also over the entire range of IP_{fm}). $fm_{stat,mod,i}$ are the predicted values on the regression line and $fm_{stat,i}$ are the actual observations. The residuals around the regression line can then be defined according to equation (3.34):

$$res_i = fm_{stat,mod,i} - fm_{stat,i} \quad (3.34)$$

Now, $p(i)$ is the probability that for a certain $IP_{fm,i}$ -value the $fm_{stat,i}$ -value is lower than the required fm_{stat} -value (the 5% fractile) for that grade. The $p(i)$ -values for a grade are therefore different for every $IP_{fm,i}$ -value and they also differ between grades for each $IP_{fm,i}$ -value. The derivation of $p(i)$ is formally written in equations (3.35) to (3.37):

$$p(i) = prob[X_i < fm_{0.05;req}], \text{ with } X_i \sim N(fm_{stat,mod,i}, s_e) \quad (3.35)$$

$$p(i) = prob\left[\left(\frac{X_i - fm_{stat,mod,i}}{s_e}\right) < \left(\frac{fm_{0.05;req} - fm_{stat,mod,i}}{s_e}\right)\right] \quad (3.36)$$

$$p(i) = prob\left[U < \left(\frac{fm_{0.05;req} - fm_{stat,mod,i}}{s_e}\right)\right] \quad (3.37)$$

U in equation (3.37) is distributed by $N(0,1)$.

X_i in equation (3.35) is the distribution of the residuals around the regression line for the value $fm_{stat,mod,i}$, which has an average value of $fm_{stat,mod,i}$ and a standard deviation of s_e . The principle is illustrated in figure 3.15. The line $y=x$ represents the case that in equation (3.33) $A=1$ and $B=0$. In that case $fm_{stat,mod,i} = IP_{fm,i}$.

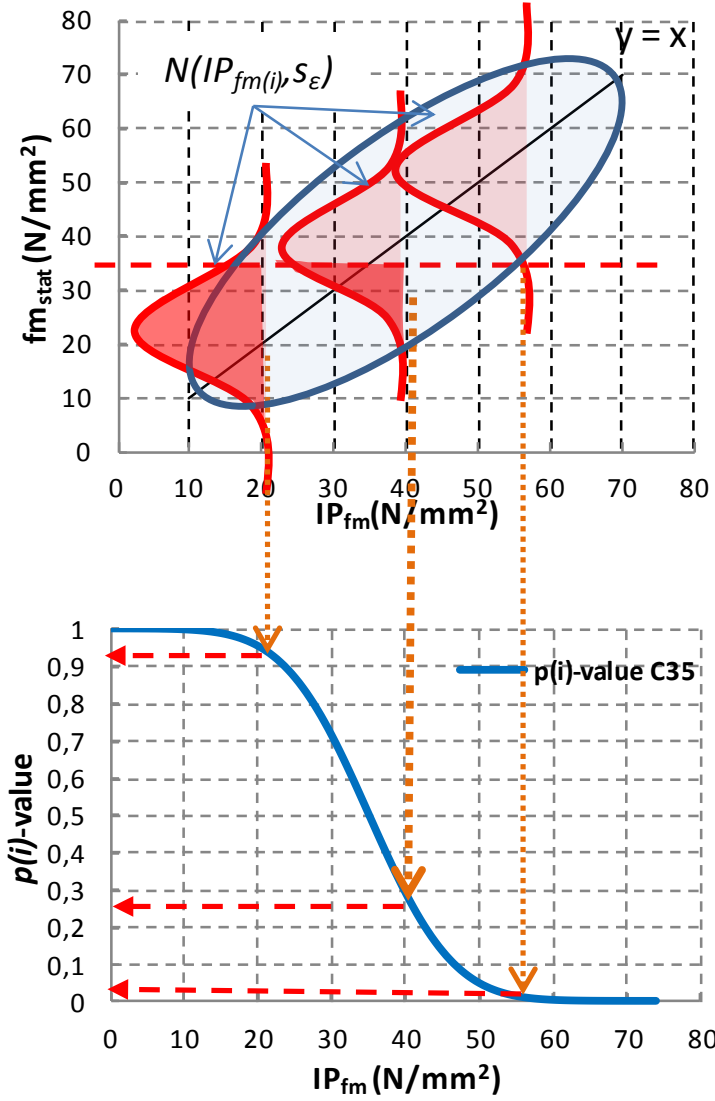


Figure 3.15. Principle of determination of $p(i)$ -values

In figure 3.15, the $p(i)$ -values are illustrated for a strength grade C35 with the following properties: $\mu_{IP} = 40.0 \text{ N/mm}^2$ and $\sigma_{IP} = 10.0 \text{ N/mm}^2$. $s_\epsilon = 9.0 \text{ N/mm}^2$.

For the $p(i)$ -value to be correct over the entire range of $IP_{fm(i)}$ -values implies an equal variance of the residuals over the entire range of IP_{fm} .

In figure 3.16 the $p(i)$ -function for the strength grades C35 from figure 1 is drawn together with the $pdf(IP_{fm})$ -distribution of the example. Finally the term p_{char} is introduced. The value of p_{char} is the average of the $p(i)$ -values for a certain IP_{fm} -interval. This is formally written in equation (3.38):

$$p_{char(l,h)} = \frac{\int_{i=IP_{fm(l)}}^{i=IP_{fm(h)}} (p(i) pdf(IP_{fm(i)})) di}{\int_{i=IP_{fm(l)}}^{i=IP_{fm(h)}} (pdf(IP_{fm(i)})) di} \quad (3.38)$$

p_{char} can be seen as the ratio of pieces below a characteristic value compared to the total amount of pieces for a certain IP_{fm} -interval .

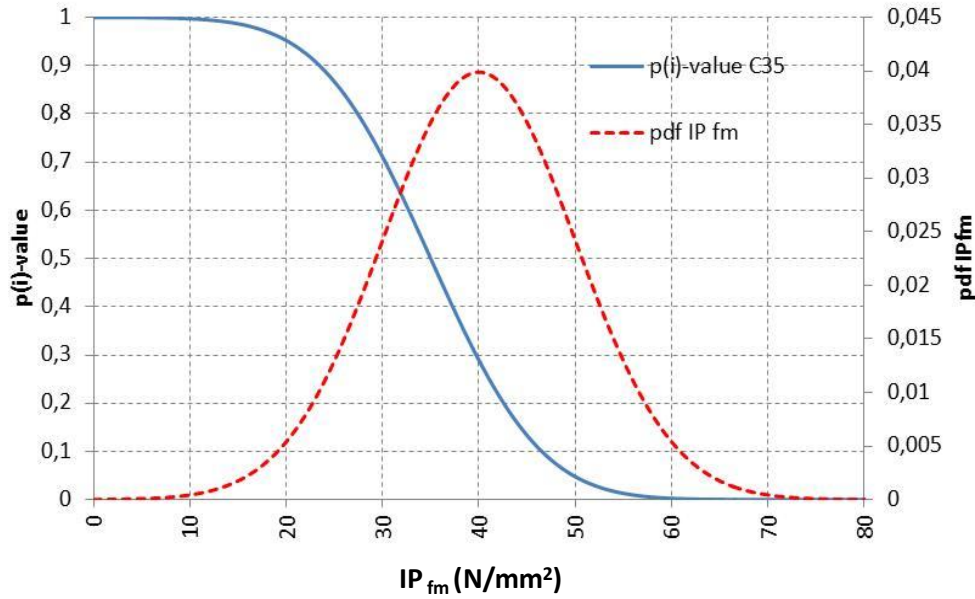


Figure 3.16. $p(i)$ -functions for strength grade C35 and pdf-function of IP_{fm} based on the properties used for figure 3.15.

The objective in deriving settings is to find the IP -limits where the 5% fractile coincides with the required value, that means for which $p_{char}=0.05$. This will be explained for the example of C35 in figure 3.16. The required value for the bending strength for C35 is 35 N/mm². The low(l) and high(h) IP_{fm} -values that will give $p_{char}=0.05$ have to be found. In principle, the high IP_{fm} -value is infinity. For the application in this example a value of 100 N/mm² for $IP_{fm}(high)$ is sufficient, because the value of $pdf(IP_{fm})$ is then practically zero. Equation (3.38) is valid for continuous functions, but for practical use a discretisation can be applied. In this thesis, a discretisation step of 0.1 N/mm² is applied. In figure 3.17 the function of $p_{char}(i,100)$ is plotted against $IP_{fm}(i)$. Where $p_{char}(i,100)=0.05$, the value of i is the value of $IP_{fm}(low)$. This value is then called the setting value for C35.

Basically, what is done in the method is the determination of the grading limit (the setting) with which a theoretical distribution for the test values can be constructed for which the 5% fractile coincides with the required value for the grade.

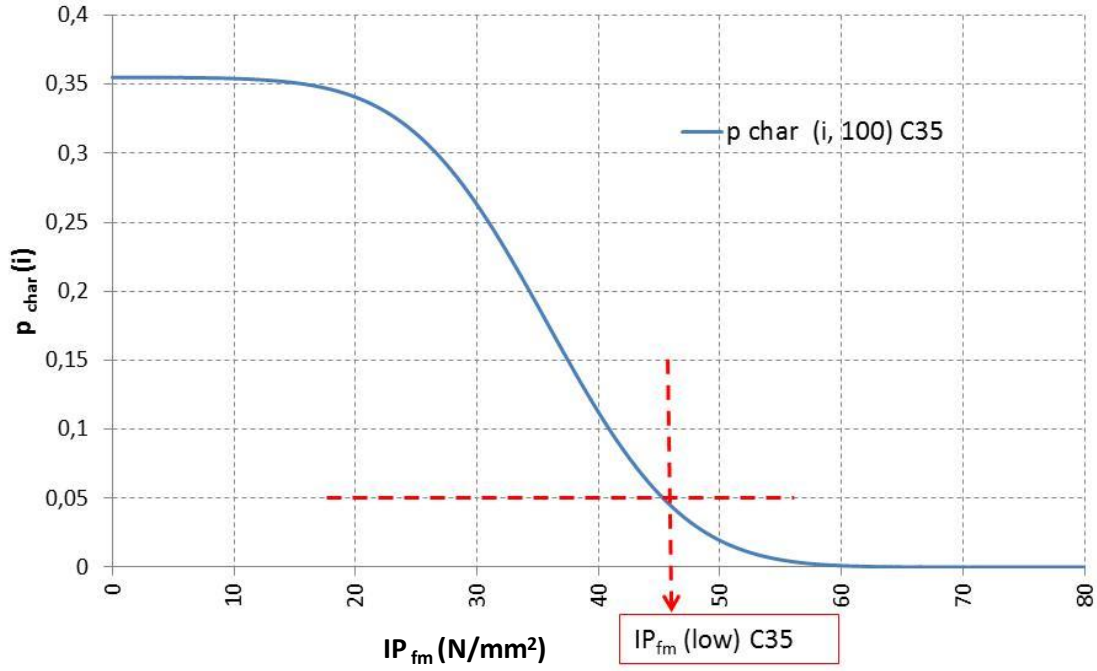


Figure 3.17. $p_{char}(i)$ -function for strength grade C35 based on the properties used in figure 3.15.

To get insight in the variation of the observations in the test data, they will be compared with this theoretical distribution of a grade .

When $IP_{fm(l)}$ and $IP_{fm(h)}$ are known, the theoretical conditional probability density function of f_{mstat} of a strength grade can be calculated. For a certain value of $f_{mstat,j}$ the theoretical conditional probability density function becomes:

$$Y_j = \frac{\int_{i=IP_{fm(l)}}^{i=IP_{fm(h)}} pdf(X_{i,j}) pdf(IP_{fm}(i)) di}{\int_{i=IP_{fm(l)}}^{i=IP_{fm(h)}} pdf(IP_{fm}(i)) di} \quad (3.39)$$

X_i is a random variable representing residuals around the regression line, for which a normal distribution is assumed with a standard deviation of s_e . By integrating the probability density function Y_j the theoretical cumulative distribution function Z can be constructed.

A discretisation procedure for steps of 0.1 N/mm^2 for di for calculating Y and dj for calculating Z gives sufficient accurate results. The cumulative distribution function Z_j will have a value of 0.05 for the required 5% fractile of that grade for $j = f_{mstat,j}$. From (3.39) it can be concluded that the shape of the distributions of the different grades will differ from each other and do not necessarily have to follow a specific (normal or lognormal) distribution, but simulations show that these will not deviate much from a normal distribution. Figure 3.18 gives the theoretical calculated cumulative distribution functions

of the total sample and strength grade C35 based on the model properties used in figures 3.15 and 3.16. The 90% confidence limit for the theoretical distribution of strength grade C35 will be explained in the next section.

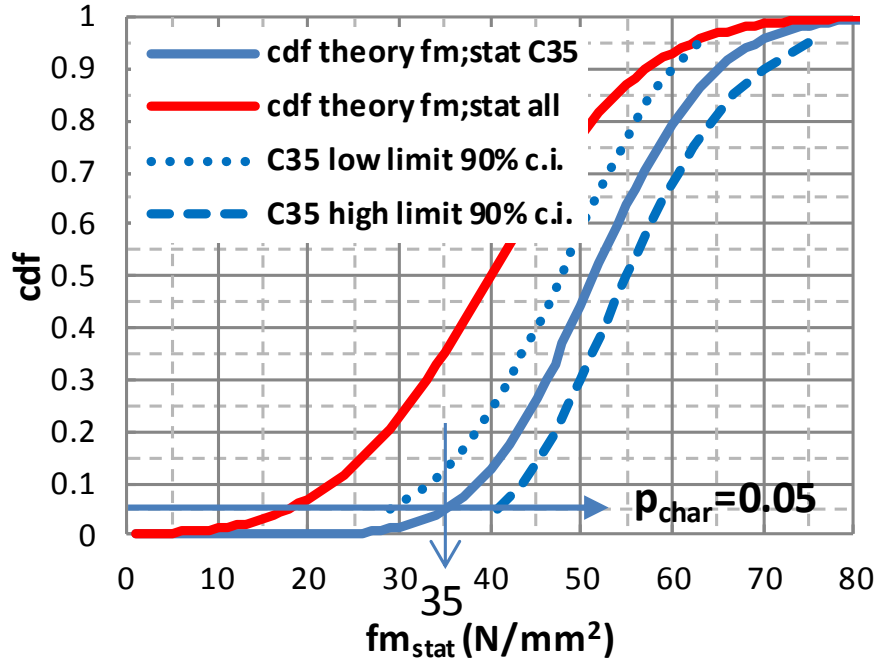


Figure 3.18. Theoretical cumulative distribution functions for the whole range and strength grade C35 based on the models properties used in figures 3.15, 3.16 and 3.17.

Although the theoretical cumulative distribution of the strength grade does not have to follow an exact normal distribution, it can be well estimated by it (see section 3.4.1). The mean (μ_g) and the variance (σ_g^2) of the theoretical cumulative distribution of the grades can be estimated with the following formulas:

$$\mu_g = \frac{\int_{i=IP_{fm(l)}}^{i=IP_{fm(h)}} fm_{stat,mod,i} pdf(IP_{fm(i)}) di}{\int_{i=IP_{fm(l)}}^{i=IP_{fm(h)}} pdf(IP_{fm(i)}) di} \quad (3.40)$$

$$\sigma_g^2 = \frac{\int_{i=IP_{fm(l)}}^{i=IP_{fm(h)}} [s_{\varepsilon}^2 + (fm_{stat,mod,i} - \mu_g)^2] pdf(IP_{fm(i)}) di}{\int_{i=IP_{fm(l)}}^{i=IP_{fm(h)}} pdf(IP_{fm(i)}) di} \quad (3.41)$$

Method for constructing of confidence intervals of the theoretical strength grade distributions.

The theoretical cumulative distributions of the grades will be used to compare them with the actual realizations of the cumulative distributions of test data. This is a more comprehensive verification of the proposed method than just comparing the 5% fractile calculated by standardized methods. To be able to compare the theoretical and actual cumulative distributions, there has to be insight in the deviations of the actual realized distribution from the theoretical distribution. There are different methods to do this. The procedure given by Pellicane and Bodig (1981) is presented below. By performing simulations, pieces are randomly drawn from the theoretical distribution of the assigned grades. Based on the simulations, the appropriate confidence interval can be constructed based on sample size. For further information about this method is referred to Pellicane and Bodig (1981).

The randomizations are done in the following way:

- The theoretically derived distribution of $f_{m,stat}$ for a strength grade is discretized into 1000 values, whereby the values were taken at each 0.001 increment of the cumulative distribution function.
- Samples with sizes 10, 20, 40 and 100 pieces are randomly taken from these 1000 values. This was repeated 100 times for each sample size.

Based on the randomizations, confidence intervals can be constructed. In this thesis 90% confidence intervals was chosen. These are intervals where 90% of the realizations are expected to be found, depending on the sample size. These confidence intervals are calculated over the range of the distribution, from the 5% to the 95% fractiles.

The objective of this procedure is to be able to determine a 90% confidence level for a specific sample size and to get insight where the distribution of test values from grades is expected. For determining the fractiles, the method of ranking and a central t-distribution are used. The interval of 90% that was chosen means that with 90% probability within these intervals the distribution of the data will occur. In figure 3.18 the low and high limits for the 90% confidence interval for C35 are drawn.

Although the theoretical distributions according to equation (3.39) do not have to be exactly normally distributed, there will be made only a slight mistake when this is assumed. This is also supported by the outcome of literature where normal distributions for strength grades are found. In that case, equation (3.19) can be used, presented here again as equation (3.42).

$$SE (PTL) = \sigma_g \sqrt{\frac{1}{n} + \frac{k^2}{2(n-1)}} \quad (3.42)$$

Where for σ_g the standard deviation of the theoretical distribution is used according to equation (3.41).

The factor k in this case is the value of the normal distribution associated with a certain probability level. So for different values for the cumulative distribution the standard error (SE) of the PTL can be calculated. Then the 90% confidence interval can be calculated for every probability level of the cumulative distribution, by using the t-distribution for $n-1$ degrees of freedom. n is the number of pieces in the grade.

3.4.3 Application of the proposed method on simulated data

To illustrate the proposed method and to evaluate the expected error in the cumulative distribution functions, it will be applied on simulated data. The properties of the population of which these data are drawn from are exactly known. In this way, the influence of the realization from a known population can be studied for the deviations from the cumulative distribution functions. The following data were simulated:

- 3 samples of 150 non-destructive IP_{fm} -values were randomly taken from a known normal population. Realistic values for the population of non-graded European spruce are IP_{fm} , : $\mu_{IP} = 40.0 \text{ N/mm}^2$ and $\sigma_{IP} = 10.0 \text{ N/mm}^2$.
- A linear regression line between the non-destructive IP-values IP_{fm} and the destructively tested bending strength $f_{m,stat}$ is assumed with the following function: $f_{m,stat,sim} = IP_{fm} + \varepsilon_{reg}$.
- ε_{reg} is the variation around the regression line with equal variance for all IP_{fm} -values and with the following distribution properties (assuming a normal distribution): $\mu_{\varepsilon,reg} = 0.0 \text{ N/mm}^2$ and $\sigma_{\varepsilon,reg} = 9.0 \text{ N/mm}^2$. To each IP_{fm} -value randomly a ε_{reg} -value is assigned.
- Independently of each other, the IP-values and ε_{reg} -values are randomly determined by the built-in random-generator of the spreadsheet program Excel. The $f_{m,stat,sim}$ values can be calculated out of these two values.

In figure 3.19 the regression results for 3 different simulations are given, to show the differences in outcome that can be expected from different randomizations drawn from the same population. Figure 4 shows that the coefficient of determination, the r^2 -value, varies between 0.45 and 0.54.

The settings and cumulative distributions are determined according to the developed method for the grade combination C35-C24-C18-reject. In table 4.13, the results are shown for the example dataset. The intervals for IP_{fm} for strength classes in the combination C35-C24-C18-reject are determined. The $IP_{fm,low}$ values are the settings values. Note that for deriving these settings the given descriptive values are necessary, but the number of pieces in the sample is not. This does not mean that this number is not important; it is important to have enough pieces to derive the correct distribution values (regression constants and error properties). The advantage of the proposed procedure is

that the occurrence of only a small amount of data in certain specific grades - which makes the current standardized method unstable - is not an issue.

As a maximum value for IP_{fm} 100 N/mm² is chosen. This is purely for numerical reasons. In principle IP_{fm} can go to infinity and $p(i)_{high}$ will then go to zero. Table 3.5 shows that for $IP_{fm}=100$ N/mm² $p(i)_{high}$ is already practically zero.

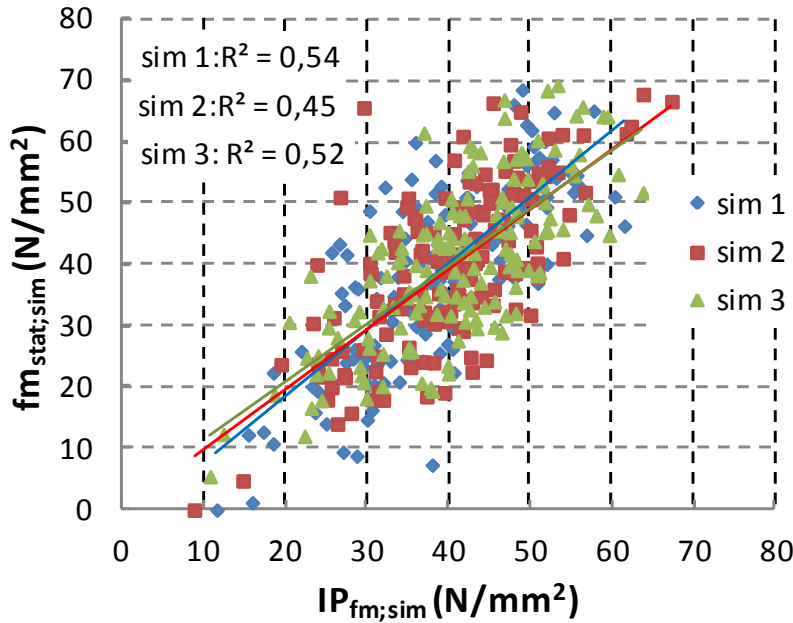


Figure 3.19. Regression plots for 3 simulations based on the modeling assumptions.

Table 3.5. Settings based on the example data, derived with discretization steps for IP_{fm} of 0.1 N/mm².

Strength class	$f_{m,stat}$ -required (N/mm ²)	$IP_{fm,low}$ (N/mm ²)	$IP_{fm,high}$ (N/mm ²)	$p(i)_{low}$	$p(i)_{high}$	p_{char}	Yield (%)
C35	35	45.4	100	0.1239	3.00E-13	0.0493	29.0
C24	21.4	29.6	45.3	0.1811	0.0041	0.0498	56.2
C18	16.1					Not possible	

Table 3.5 gives the low and high $p(i)$ -values for each grade and the connected IP_{fm} -values. The expected yields (= the part of the total amount of pieces that can be assigned to that specific grade) on the basis of the distributions can also be automatically calculated. The IP_{fm} limit values can be determined from the p_{char} functions. See figure 3.20. In figure 3.20 the p_{char} functions are drawn for C35, C24 and C18. The low IP_{fm} -value for C35 is 45.4 N/mm². A discretization step of 0.1 N/mm² is applied, so the high IP_{fm} -value for C24 is 45.3 N/mm². The figure shows that for C18 no settings are possible, because the value for the p_{char} function is already above 0.05 for the high IP_{fm} -value of 29.4 N/mm² for C18.

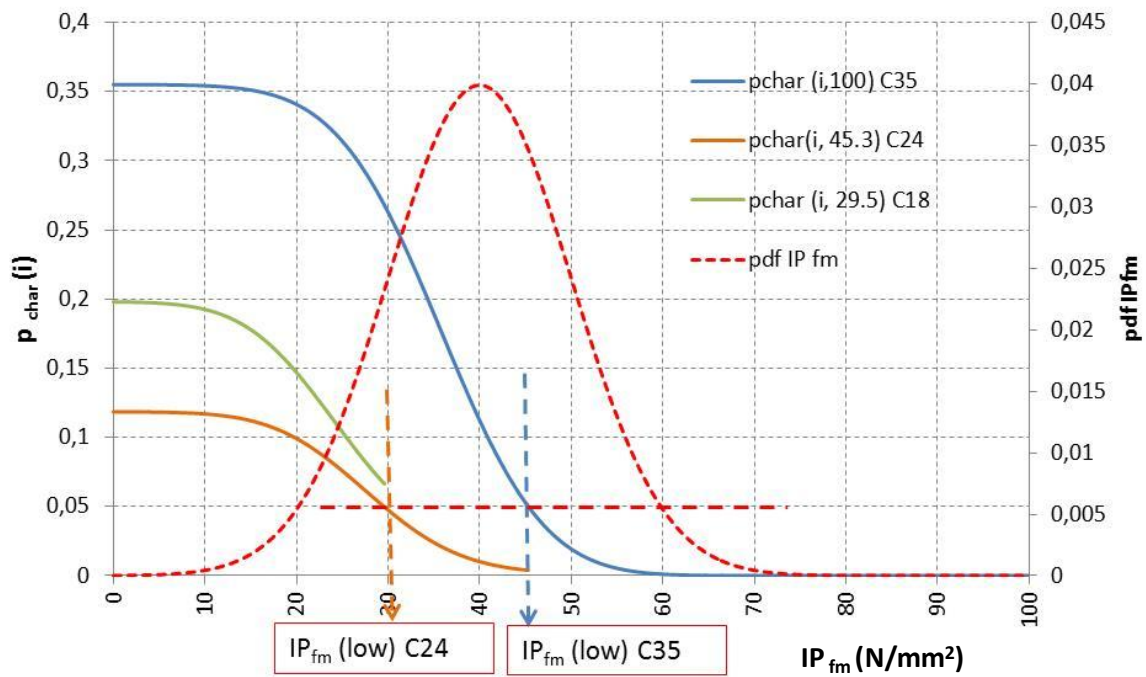


Figure 3.20. Determination of IP_{fm} limits by use of p_{char} functions.

This is also explained in figure 3.21. In figure 3.21 the $p(i)$ -values for C35 and C24 for the limits according to table 3.5 are plotted. For IP_{fm} - values below the lower limit value for C24 the $p(i)$ -values for grade C18 are plotted. The graph immediately shows that it is not possible to determine grade limits for C18, because the $p(i)$ -value for the highest possible IP_{fm} - value is above 0.05. Since the p_{char} value is an average value of $p(i)$ -values, it is clear that a value of 0.05 for p_{char} cannot be achieved. This means that everything below grade C24 has to be assigned as reject in this grade combination. The only way to establish pieces in a grade C18 is to set a higher limit level for C24, so that a considerable amount of $p(i)$ -values will get a value lower than 0.05 for the IP_{fm} -values of C18. This will result in a lower yield for C24 (or C35 when the lower limit for this grade will also be raised), and a lower value for p_{char} for C24 (or C35). This means a higher 5% fractile (more safe) for these higher grades.

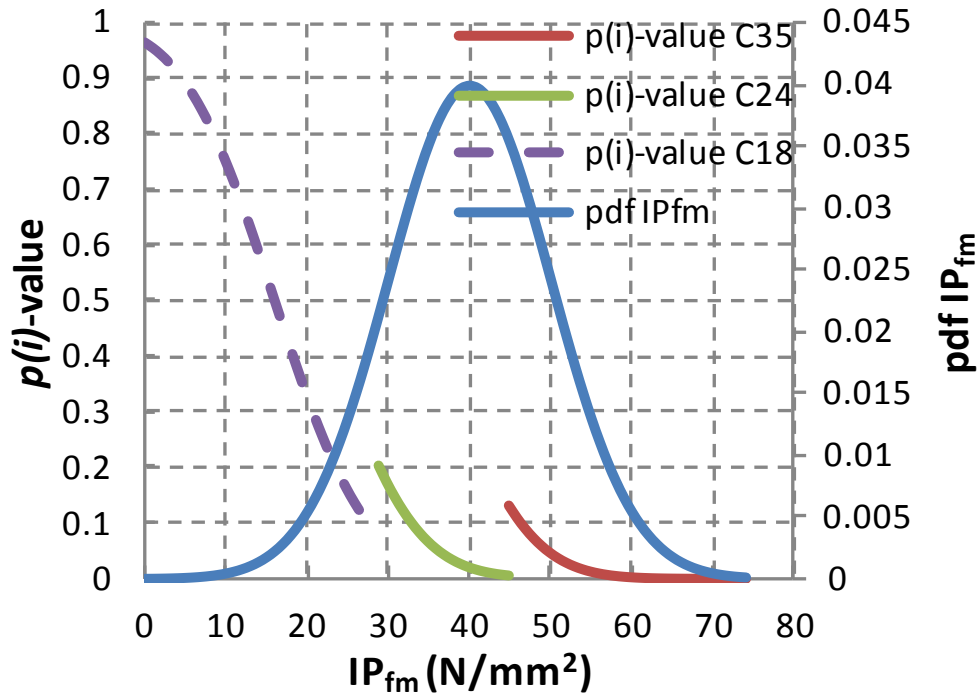


Figure 3.21. $p(i)$ -values for the settings according to table 4.13.

Figure 3.21 also gives an indication how the problems can be solved when settings are determined with prediction limits as proposed in Ziethen et al. (2010). The idea behind prediction limits is that the magnitude of the normally distributed standard deviation of the residuals of the regression line is determined and assumed constant (equal variances). The prediction limit is chosen (this can be the 5% lower limit line, but also the 10% lower limit line) in such a way that the required 5% fractile for the grade will meet the requirements.

However, when figures 3.20 and 3.21 are studied, it can be seen that a certain prediction limit can only give correct results for all grades when the distribution of IP_{fm} would be rectangular and the distances between the setting values are at equal distances. Since both preconditions practically never exists, the prediction limit method will always overestimate and/or underestimate certain grades in a grade combination, which also can be concluded from Ranta-Maunus (2012). The proposed method in this thesis can be seen as a further elaboration of the principle of the prediction limit method.

With the settings determined from table 3.5, the pieces are graded based on their simulated IP_{fm} . The basic grading results of the 3 randomizations of figure 3.19 are listed in table 3.6.

Table 3.6. Basic data for 3 randomizations

properties	Simulation number			theoretical
	1	2	3	
$IP_{fm;sim}$ total sample (N/mm ²)	39.4 (10.0)	40.1 (9.4)	40.8 (10.0)	40.0 (10.0)
$f_{mstat;sim}$ total sample (N/mm ²)	39.7 (14.0)	39.7 (12.9)	40.3 (13.1)	40.0 (13.4)
$f_{mstat;sim}$ C35 (N/mm ²)	52.0 (8.1)	50.2 (9.6)	50.1 (10.0)	51.7 (10.4)
Yield C35 (%)	29.3	26.0	35.3	29.9
$f_{mstat;sim}$ C24 (N/mm ²)	37.7 (11.2)	37.9 (10.3)	38.0 (10.0)	38.1 (10.0)
Yield C24(%)	54.7	60.7	51.3	56.2
$f_{mstat;sim}$ reject (N/mm ²)	22.5 (12.6)	25.1 (14.5)	23.3 (8.1)	24.2 (9.7)
Yield reject (%)	16.0	13.3	13.3	14.5

Note: Main numbers are mean values, standard deviations in parenthesis. Yields are exact values.

The mean and standard deviations for the pieces assigned to grades are calculated. The largest absolute value for the skewness was 0.34 and for the kurtosis 0.67, so the assumption of normality of IP_{fm} seems reasonable, when a limit of an absolute value of 1 is kept for both properties (De Vocht, 2000). In table 3.7, the 5% fractiles of the C35 and C24 grades are given, determined with different methods that would normally be used. The different methods are:

- Ranking according to EN 384
- Using a central student distribution, assuming that the data are normally distributed
- The method according of EN14358, using a non-central student distribution on the lognormal values, assuming that the data are logarithmically normally distributed. In this method, a confidence interval of 75% on the 5% fractile is applied.

Table 3.7. 5% fractiles of assigned C35 and C24 grades.

Simulation number	Strength grade	Ranking (EN 384) (N/mm ²)	Central t-distribution (N/mm ²)	Non-central lognormal t-distribution (EN 14358) (N/mm ²)	Mean $p(i)$	Required value (N/mm ²)
1	C35	37.4	38.4	38.1	0.046	35
1	C24	20.4	19.1	19.0	0.052	21.4
2	C35	32.7	34.1	34.3	0.052	35
2	C24	19.5	20.7	21.8	0.042	21.4
3	C35	32.0	33.4	33.7	0.052	35
3	C24	20.4	21.3	22.2	0.046	21.4

Also, the mean value of the calculated $p(i)$ -values of the simulations is given for every grade. A remarkable finding is that there is a big difference between the C35 –values for simulations 1, 2 and 3. The mean $p(i)$ - values are close to 0.05. Since we know that the parent distribution is the same for all three simulations, this is apparently the variation that can be expected for 5% fractiles of samples from the parent sample. According to the current EN 14081-2 procedure (for which the settings are found by trial and error), settings for C35 for simulations 2 and 3 would have to be raised, and for simulation 1 could be lowered. With the developed method in this thesis would not be done, all these 3 different observations would be accepted. In figure 3.22 the calculated absolute errors for a 90% -confidence interval for the 5% fractile of the theoretical distribution of C35 (a value of 35 N/mm²) are shown. They are plotted against the sample size.

As expected, figure 3.22 shows, that for smaller sample sizes the error interval increases. This graph is only valid for the error interval at the 5% fractile of theoretical distribution for the grade C35 derived in this example. For other distributions (between other IP_{fm} -limits) the error intervals may be different. Figure 3.22 shows that for samples of $n=300$ an absolute error of approximately 2 N/mm² can be expected for 90% of the samples. (So for 90% of the samples with $n=300$, a 5% fractile between 33 and 37 can be expected). For the number of specimens in the grade C35 in this example - around 45 - an error of at least 4.0 N/mm² can be expected. The 5% fractiles found in table 3.7 are then not illogical. Figure 3.22 shows that the approximation of the absolute error according to equation (3.42) does not deviate much from the absolute error derived from simulations from the theoretical distribution.

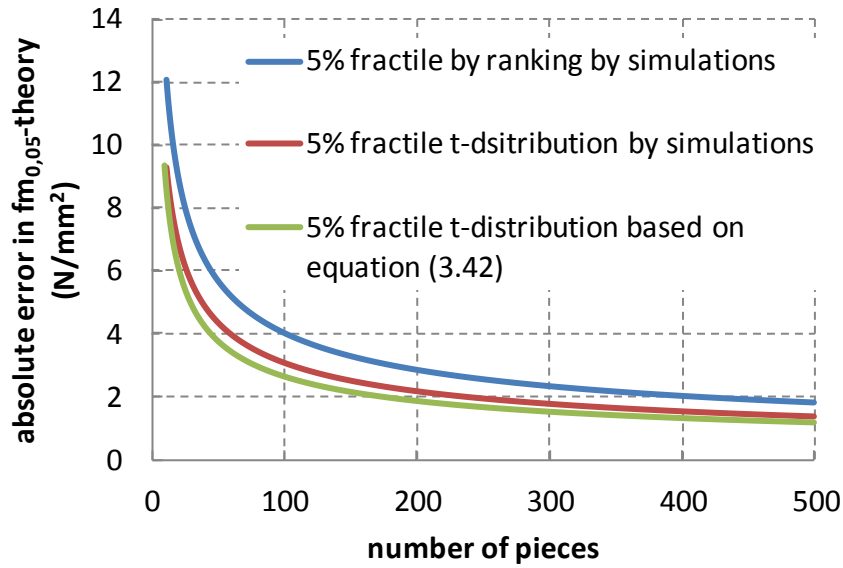


Figure 3.22. The absolute errors for a 90% -confidence interval for the 5% fractile of the theoretical distribution of C35 (35 N/mm^2) based on simulations and by applying equation (3.42).

To be on the safe side, results for the 5% fractiles of samples with $n=300$, found with the method of ranking, should be reduced by 2 N/mm^2 , if a 90% interval is chosen. Mostly, a confidence interval of 75% is used. This would give a slightly lower error. However, on the other hand, when the sample is not used to derive the 5% fractiles but to verify a sample, and a 5% fractile between 33 and 37 is found, then with 90% probability it cannot be stated that the sample does not come from a population with a 5% fractile of 35 N/mm^2 . This last observation will be used to verify the settings derived according to the example. Not only at the 5% fractile level but the entire distribution will be used to evaluate samples.

The C35-distribution of the 3 simulations is compared with the theoretical distribution of C35. Therefore the 3 simulations are plotted together with the calculated upper and lower limit to create a 90% interval for a sample size of 45 pieces. In this case the method of determining the 90% interval is the method of ranking. See figure 3.23. Sample sizes are 44 (simulation 1), 39 (simulation 2) and 54 (simulation 3). When there are more pieces in a grade, the 90% confidence limit will be smaller, see also figure 3.22.

Figure 3.23 shows that these 3 simulations of which we are certain that they are random samples from the theoretical population, stay within the 90% limit lines over the entire distribution, but locally the distribution can show considerable deviations from the theoretical one.

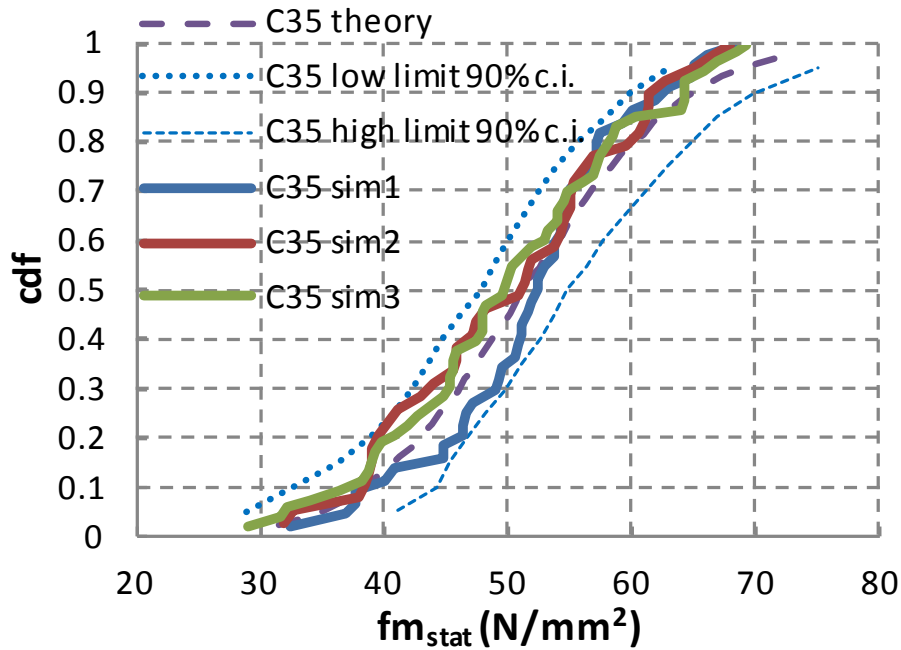


Figure 3.23. Distributions of C35 for simulations 1, 2 and 3 and the 90% confidence interval for the theoretical distribution.

For large numbers of specimens in a sample, the ranking method will eventually coincide with the theoretical distribution, but for the numbers that are dealt with in practice, all methods for determining the 5% fractiles of a grade are only an indication of the 5% of the distribution they are taken from.

3.4.4 Discussion and conclusions

A new method for the derivation of settings for grading machines is presented in section 3.4. 2. The determination of the settings is based on the distribution properties of the non-destructive Indicating Property and the residuals around the regression lines with the bending strength. The method avoids the problem of interpreting the 5% fractiles of graded samples with relatively small samples sizes.

The following can be concluded:

- Literature review indicates that the distribution of graded and ungraded timber can be well described by a statistical distribution.
- The accuracy of the 5% fractiles of grades very much depends on the number of pieces in the grade for the numbers that occur in machine grading. Simulations show that this therefore also has a great influence on the determination of settings with the current method.
- The proposed method is based on model properties and is less sensitive for small number of pieces in grades. To evaluate the method, the whole distribution of the grade can be compared with the theoretical distribution, for which an expected confidence level can be calculated.

- In this thesis, the proposed method will be used to calculate settings for machine grading in combination with the prediction models to be derived.

Examples of the method described in this section on real data can be found in Ravenshorst and Van de Kuilen (2014).

Experimental results

4.1 Basic test results

In this section, the basic test results are presented.

Basic test results for the dataset of tropical hardwoods.

In table 4.1, the properties for which quantified measured data for every specimen is available are given for every sample according to table 3.1. When the cell is denoted with “Y” (for Yes), then quantified data are available for every specimen. When no quantified data are present still all pieces are visually assessed according to the methods described in chapter 3. An overview of the pieces that did not pass this visual assessment is listed in appendix B. They are removed from the analysis and not included in the given property tables. Samples GR4 and MAS5 were split into two where one half was tested dry (around 12%-15% moisture content) and the other half was tested above fiber saturation point. Sample GR4 was also divided in pieces containing knots and pieces with no knots.

In table 4.2 the mean and standard deviations for the bending strength, MOE_{local} , MOE_{global} and MOE_{dyn} for the tropical hardwood samples are given. Table 4.3 shows the mean and standard deviation for the density, the moisture content, the knot ratio and the slope of grain for the tropical hardwood samples. In tropical hardwood timber, knots are more rare than in temperate hardwoods and softwoods, and when they occur, there is usually only one present in the beam. The mean and standard deviation of the knot ratios given in table 4.3 only take into account the pieces containing knots.

For all knot ratios listed in table 4.3, the single knot ratio (SKR) is given.

All MOE values were rounded to 100 N/mm^2 , all density values to 10 kg/m^3 . All bending strength values and moisture content values were rounded to 0.1 N/mm^2 . For the knot ratios and slopes of grain the values were rounded to two decimal numbers after the decimal point.

Table 4.1. Available test data per test specimen for tropical hardwood samples

Sample ID	Density (ρ)	m.c.	MOE_{dyn}	MOE_{loc}	MOE_{glob}	Knot ratio (SKR)	Slope of grain (SoG)
AV1	Y	Y	Y	Y			
AV2	Y	Y	Y		Y		
AV3	Y	Y	Y		Y		
AV4	Y	Y	Y	Y	Y		
AV5	Y	Y	Y	Y	Y		
CUM1	Y	Y	Y	Y			
CUM1	Y	Y	Y	Y	Y		
CUM3	Y	Y	Y	Y	Y		
CUM4	Y	Y	Y	Y	Y		
CUM5	Y	Y	Y	Y	Y		
MAS1	Y	Y	Y	Y	Y		
MAS2	Y	Y	Y	Y	Y		
MAS3	Y	Y	Y	Y	Y		
MAS4	Y	Y	Y	Y	Y	Y	Y
MAS5	Y	Y	Y		Y	Y	Y
AZ1	Y	Y			Y		
AZ2	Y	Y	Y	Y	Y		
AZ3	Y	Y	Y	Y	Y		
GR1	Y	Y	Y		Y		
GR2	Y	Y	Y	Y	Y		
GR3	Y	Y	Y	Y	Y		
GR4	Y	Y	Y	Y	Y	Y	Y
OK1	Y	Y	Y	Y	Y		
OK2	Y	Y	Y		Y	Y	Y
OK3	Y	Y	Y		Y	Y	Y
KA1	Y	Y	Y	Y			
NA1	Y	Y	Y	Y			
PI1	Y	Y	Y	Y			
VI1	Y	Y	Y	Y			
BAS1	Y	Y	Y	Y			
BAN1	Y	Y	Y	Y			
SV1	Y	Y	Y	Y	Y		
CR1	Y	Y	Y		Y		
LA1	Y	Y	Y		Y		
LF1	Y	Y	Y		Y		
PU1	Y	Y	Y	Y			
TV1	Y	Y	Y	Y	Y		
FA1	Y	Y	Y	Y	Y		
SA1	Y	Y	Y	Y	Y		
FP1	Y	Y	Y	Y	Y		
BIL1	Y	Y	Y		Y	Y	Y
BIL2	Y	Y	Y		Y	Y	Y
EV1	Y	Y	Y		Y	Y	Y
EV2	Y	Y	Y		Y	Y	Y
TA1	Y	Y	Y		Y	Y	Y
TA2	Y	Y	Y		Y	Y	Y

Table 4.2. Test data for the bending strength, MOE_{local} , MOE_{global} and MOE_{dyn} for the tropical hardwood samples. Mean ($\bar{x}$) and standard deviations (s) of every property.

Sample ID	f_m (N/mm ²)		MOE_{loc} (N/mm ²)		MOE_{glob} (N/mm ²)		MOE_{dyn} (N/mm ²)	
	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s
AV1	82.8	16.9	1600	1800	-	-	18900	1900
AV2	103.0	16.6	-	-	17000	1800	20400	1700
AV3	97.8	22.8	-	-	16600	3100	19700	3200
AV4	98.9	23.2	20000	3200	16400	2100	21400	2800
AV5	79.2	15.7	14800	3800	13100	2000	17000	2500
CUM1	102.8	21.2	18300	2900	-	-	21200	3100
CUM1	107.3	24.1	19500	4500	17400	3400	20900	4100
CUM3	117.7	15.9	20700	2600	17800	1900	21300	2300
CUM4	87.1	18.3	18400	3000	15600	2000	20400	3000
CUM5	79.8	18.4	17500	3300	14500	1700	19500	2500
MAS1	124.2	17.7	24700	8600	-	-	21600	2200
MAS2	96.5	16.6	19100	2500	20300	2600	20000	2300
MAS3	64.4	15.3	-	-	11200	2000	13600	2300
MAS4	119.1	23.2	-	-	19100	2500	25100	3500
MAS5	109.5	34.6	-	-	21700	3700	24600	3700
AZ1	93.8	14.7	-	-	17000	2200	20700	2700
AZ2	80.7	12.5	16900	2700	13700	1700	17700	2000
AZ3	111.6	20.1	21700	3100	18700	2400	21200	2700
GR1	102.1	12.2	-	-	19400	2300	23400	2700
GR2	91.4	21.0	23400	3900	19600	3000	23500	3700
GR3	80.7	18.0	27200	3300	19500	2300	26400	2900
GR4	92.3	26.7	24800	4500	21700	4500	24600	2900
OK1	75.7	11.8	17000	5100	-	-	18000	1700
OK2	58.2	17.6	-	-	13800	2700	18000	3800
OK3	81.6	18.7	-	-	16800	2600	19500	3200
KA1	65.6	9.2	15800	2300	-	-	18900	2200
NA1	75.3	13.6	18200	3900	-	-	17000	2000
PI1	63.3	14.2	18600	5700	-	-	16200	2400
VI1	58.4	9.7	13100	2100	-	-	13700	1600
BAS1	61.3	17.0	17700	5300	-	-	18100	2000
BAN1	91.4	18.2	20400	3500	-	-	21000	2500
SV1	79.9	16.7	17400	3900	16100	3300	18800	3700
CR1	87.6	10.3	-	-	15400	1300	18100	1600
LA1	81.8	15.6	-	-	15300	2600	17600	3200
LF1	81.7	13.7	-	-	18300	2500	21600	3000
PU1	84.3	17.0	18600	2200	-	-	19400	1700
TV1	60.0	12.1	13500	4800	14800	3600	15100	2200
FA1	76.7	19.0	17300	2100	15400	1600	18900	2400
FP1	89.1	16.1	19300	2300	16900	1800	20900	2500
SA1	71.9	14.3	15700	3100	14100	2400	17200	2500
BIL1	36.6	9.3	-	-	10500	1400	12900	1100
BIL2	47.6	17.8	-	-	11800	2800	13200	3300
EV1	128.2	28.5	-	-	24700	1900	28000	1500
EV2	100.6	16.8	-	-	18500	1800	23000	2300
TA1	71.4	14.8	-	-	14100	1200	17800	1400
TA2	63.3	15.9	-	-	14300	1700	16800	1800

Table 4.3. Test data for the density, the moisture content, the knot ratio and the slope of grain for the tropical hardwood samples.

Sample	Density ρ (kg/m ³)		m.c. (%)		Knot ratio (SKR)		Slope of grain (SoG)	
	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s
AV1	1070	50	15.0	1.6	-	-	-	-
AV2	1260	40	35.2	2.6	-	-	-	-
AV3	1240	40	30.2	4.3	-	-	-	-
AV4	1220	50	30.0	0.0	-	-	-	-
AV5	1200	60	30.0	0.0	-	-	-	-
CUM1	1080	60	20.3	3.6	-	-	-	-
CUM1	1010	80	15.9	1.1	-	-	-	-
CUM3	1060	80	16.8	2.7	-	-	-	-
CUM4	950	70	15.6	1.1	-	-	-	-
CUM5	970	60	16.5	1.1	-	-	-	-
MAS1	1100	50	20.7	2.5	-	-	-	-
MAS2	1070	50	18.8	2.3	-	-	-	-
MAS3	1030	40	16.9	0.0	-	-	-	-
MAS4	1110	60	19.8	4.5	0.05	0.04	0.07	0.05
MAS5	1130	110	23.9	12.2	0.11	0.10	0.05	0.03
AZ1	1240	50	42.6	4.3	-	-	-	-
AZ2	1190	40	41.9	5.5	-	-	-	-
AZ3	1130	30	30.9	2.1	-	-	-	-
GR1	1160	60	27.9	2.9	-	-	-	-
GR2	1070	70	25.0	5.3	-	-	-	-
GR3	1040	50	25.5	5.5	-	-	-	-
GR4	1030	60	19.6	6.9	0.25	0.13	0.08	0.03
OK1	990	80	18.4	0.9	-	-	-	-
OK2	1060	60	15.0	0.0	0.08	0.04	0.34	0.23
OK3	940	150	16.3	2.3	0.15	0.13	0.21	0.11
KA1	930	60	50.3	12.9	-	-	-	-
NA1	740	70	15.7	6.5	-	-	-	-
PI1	940	50	36.9	15.3	-	-	-	-
VI1	910	80	41.6	7.7	-	-	-	-
BAS1	940	110	47.9	12.4	-	-	-	-
BAN1	940	50	16.2	5.0	-	-	-	-
SV1	930	60	19.5	3.7	-	-	-	-
CR1	850	50	17.1	0.0	-	-	-	-
LA1	810	100	19.3	0.0	-	-	-	-
LF1	970	50	22.5	0.0	-	-	-	-
PU1	820	100	16.9	0.8	-	-	-	-
TV1	740	30	21.0	2.8	-	-	-	-
FA1	860	70	32.0	5.6	-	-	-	-
FP1	920	90	19.8	2.8	-	-	-	-
SA1	720	50	17.6	3.2	-	-	-	-
BIL1	870	40	33.4	10.2	0.12	0.04	0.38	0.24
BIL2	740	50	13.9	1.4	0.05	0.03	0.32	0.18
EV1	1030	40	10.4	1.1	0.14	0.09	0.07	0.04
EV2	1020	50	16.8	4.6	0.12	0.08	0.13	0.07
TA1	910	60	15.0	0.0	0.10	0.04	0.24	0.20
TA2	990	60	23.6	7.4	0.15	0.07	0.23	0.11

Basic test results for the dataset of temperate European hardwoods.

In table 4.4. the properties that quantified data for every specimen is available for, are given for every temperate hardwood sample, as listed in table 3.2.

In table 4.5. the mean and standard deviations for the bending strength, MOE_{local} , MOE_{global} and MOE_{dyn} for the temperate hardwood samples are given. In table 4.6. the mean and standard deviation for the density, the moisture content and the knot ratio for the temperate hardwood samples are listed. For all knot ratios *listed* the single knot ratio (SKR) is given.

Table 4.4. Available test data per test specimen for temperate hardwood samples.

Sample ID	Density (ρ)	m.c.	MOE_{dyn}	MOE_{loc}	MOE_{glob}	Knot ratio (SKR)
O1	Y	Y	Y	Y		Y
O2	Y	Y	Y	Y		Y
R1	Y	Y	Y	Y		Y
C1	Y	Y	Y	Y	Y	

Table 4.5. Test data for the bending strength, MOE_{local} , MOE_{global} and MOE_{dyn} for the temperate hardwoods samples. Mean ($\bar{x}$) and standard deviations (s) of every property.

Sample	f_m (N/mm ²)		MOE_{loc} (N/mm ²)		MOE_{glob} (N/mm ²)		MOE_{dyn} (N/mm ²)	
	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s
O1	42.0	6.8	9300	1600	-	-	11200	1800
O2	38.8	10.4	8600	3500	-	-	9000	2400
R1	66.1	14.8	15400	1900	-	-	15800	1700
C1	55.1	14.1	12800	2200	11600	1600	14100	2000

Table 4.6. Test data for the density, the moisture content and the knot ratio for the temperate hardwood samples

Sample	Density ρ (kg/m ³)		m.c. (%)		Knot ratio (SKR)	
	$\bar{x}$	s	$\bar{x}$	$\bar{x}$	s	$\bar{x}$
O1	930	60	69.6	8.3	0.14	0.15
O2	790	70	32.5	9.9	0.37	0.29
R1	750	50	22.1	4.1	0.10	0.07
C1	590	50	13.4	1.3	-	-

Basic test results for the dataset of European softwoods.

In table 4.7, the properties for which quantified data for every specimen is available, are given for every European softwood sample.

In table 4.8, the mean and standard deviations for the bending strength, MOE_{local} , MOE_{global} and MOE_{dyn} for the European softwood samples are given. In table 4.9, the mean and standard deviation for the density, the moisture content and the knot ratio the European softwood samples are given. For all knot ratios the single knot ratio (SKR) is given, except for sample D1 where the group knot ratio (GKR) is given.

Table 4.7. Available test data per test specimen for softwood samples

Sample ID	Density (ρ)	m.c.	MOE_{dyn}	MOE_{loc}	MOE_{glob}	Knot ratio (SKR or (GKR))
S1	Y	Y	Y	Y		Y (SKR)
S2	Y	Y	Y	Y		
S3	Y	Y	Y		Y	
S4	Y	Y	Y	Y		
S5	Y	Y	Y	Y	Y	
S6	Y	Y	Y	Y	Y	
S7	Y	Y	Y	Y		
S8	Y	Y	Y	Y		
D1	Y	Y	Y	Y		Y (GKR)
L1	Y	Y	Y	Y		Y (SKR)
L2	Y	Y	Y	Y		Y (GKR)

Table 4.8. Test data for the bending strength, MOE_{local} , MOE_{global} and MOE_{dyn} for the European softwoods samples. Mean ($\bar{x}$) and standard deviations (s) of every property

Sample	f_m (N/mm ²)		MOE_{loc} (N/mm ²)		MOE_{glob} (N/mm ²)		MOE_{dyn} (N/mm ²)	
	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s
S1	37.0	10.4	10600	2400	-	-	11600	2000
S2	41.8	14.4	11900	2900	-	-	13100	2500
S3	38.2	12.4	-	-	10000	2300	12500	2400
S4	42.9	13.1	11000	2500	-	-	11800	2300
S5	41.9	8.2	13500	2700	10600	1800	13000	2100
S6	42.1	10.5	12900	2800	10800	1800	12800	2300
S7	62.5	9.6	16100	2000	-	-	16400	2200
S8	46.3	10.4	8600	2500	-	-	11000	2100
D1	48.6	17.0	13300	3200	-	-	14100	3100
L1	48.4	10.5	10400	2400	-	-	10400	1900
L2	55.7	14.2	14700	4500	-	-	14300	2200

Table 4.9. Test data for the density, the moisture content and the knot ratio for the European softwood samples

Sample	Density ρ (kg/m ³)		m.c. (%)		Knot ratio (SKR)	
	s	$\bar{x}$	$\bar{x}$	s	$\bar{x}$	s
S1	440	40	12.1	2.7	0.29	0.14
S2	440	50	12.0	1.3	-	-
S3	460	40	15.1	0.9	-	-
S4	430	40	12.0	0.3	-	-
S5	480	50	17.1	0.8	-	-
S6	450	50	14.2	1.6	-	-
S7	520	50	17.0	1.0	-	-
S8	440	50	12.6	1.2	-	-
D1	580	60	13.6	1.8	0.24	0.12
L1	530	60	13.7	1.6	0.09	0.06
L2	620	60	14.4	3.7	0.28	0.23

4.2 Relationships between unadjusted properties

In this section, scatterplots with the relationships between the measured material properties are given. In general, two plots are given for the relationship between two properties. One plot with datapoints from all three datasets (tropical hardwoods, European temperate hardwoods and European softwoods) together with the linear regression line derived by a least squares regression. In the plots the equation of the linear regression line and the coefficient of determination r^2 are given. The second plot gives the same data but now with the three datasets indicated separately.

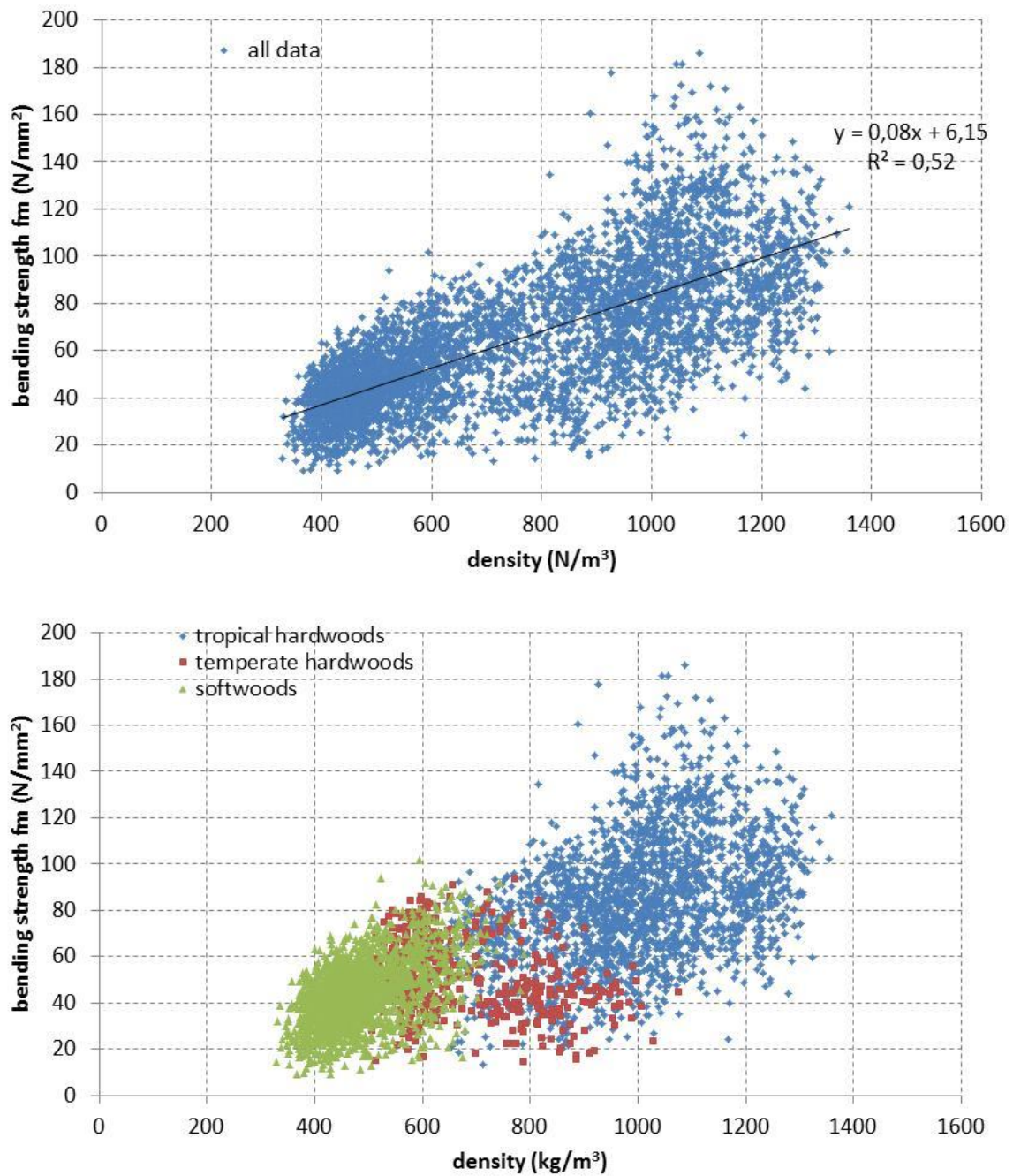


Figure 4.1. Scatterplots of the bending strength f_m against the density ρ .

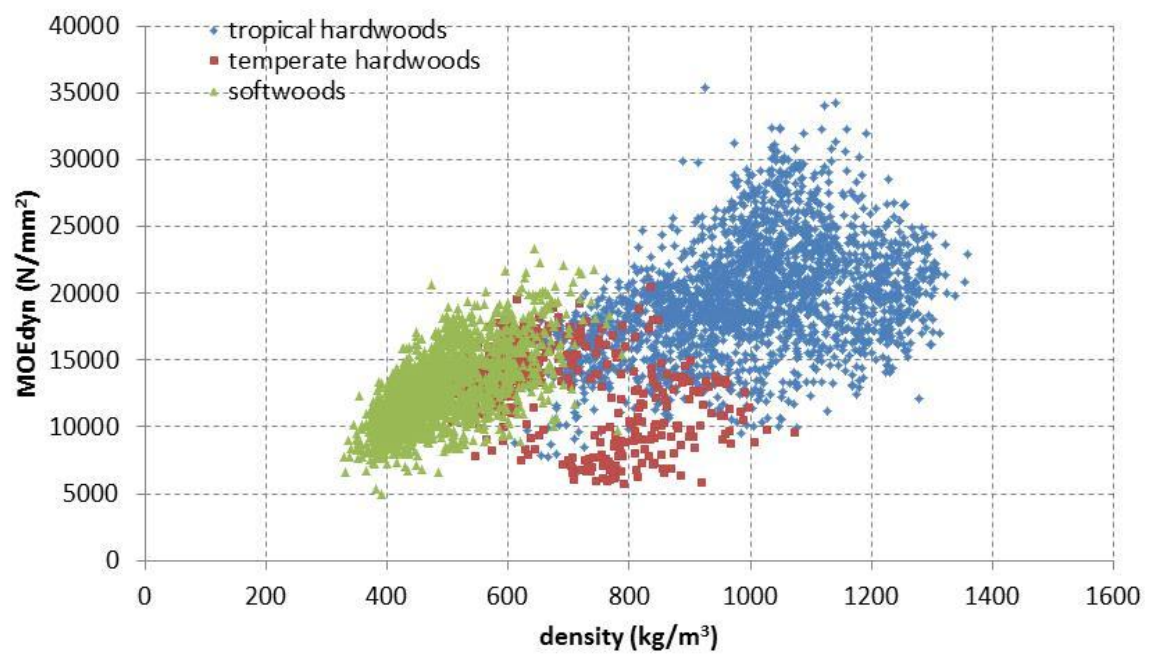
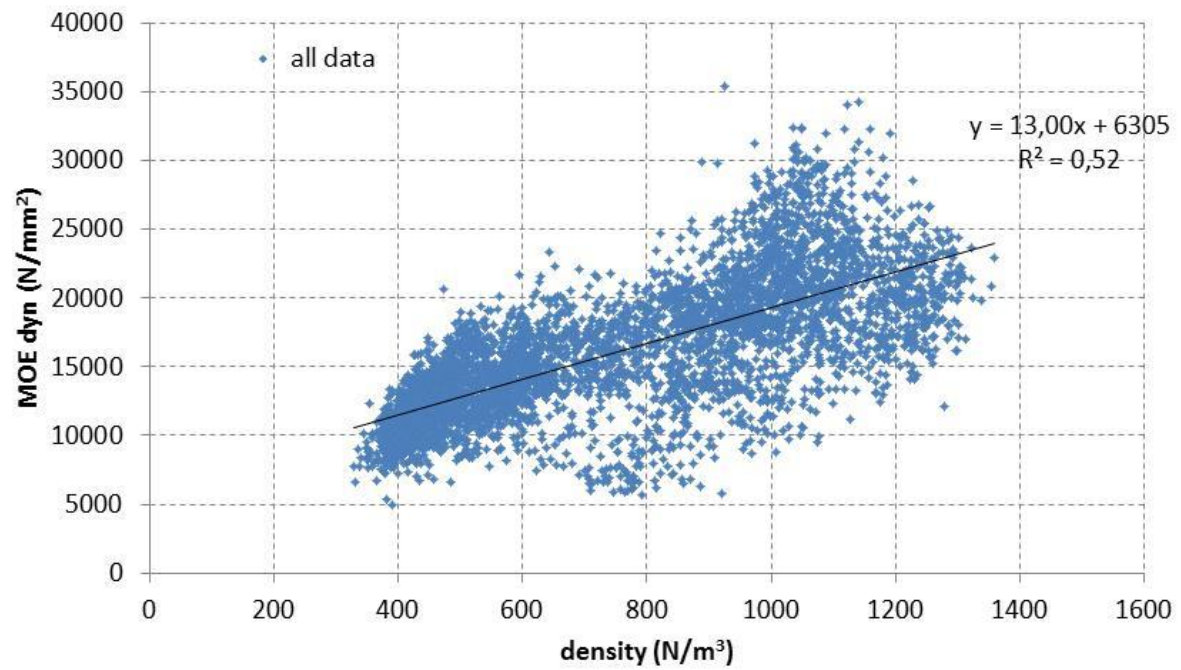


Figure 4.2. Scatterplots of the MOE_{dyn} against the density ρ .

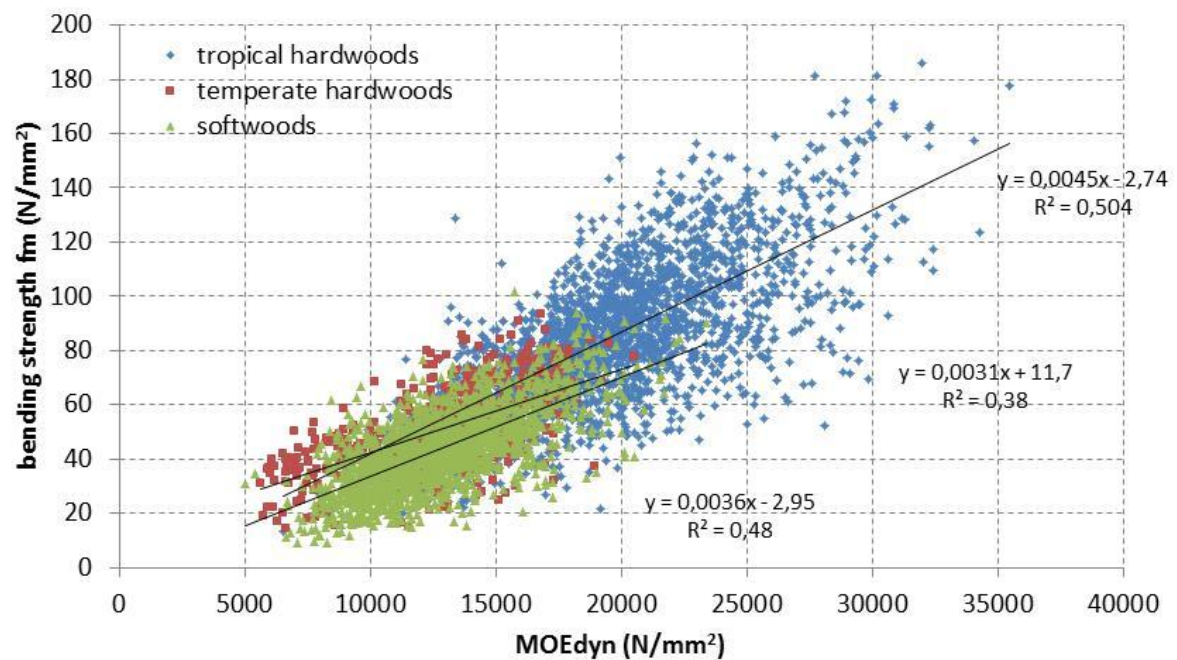
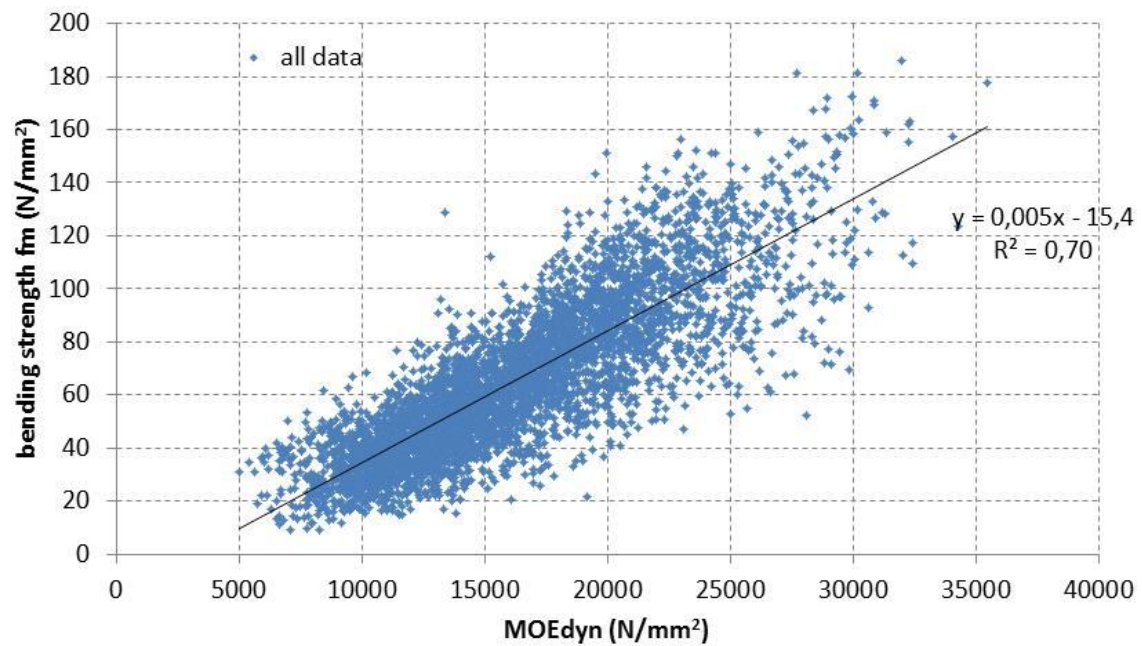


Figure 4.3. Scatterplots of the bending strength against the MOE_{dyn}

The regression lines and coefficients of determination r^2 that are connected with the datasets are:

- Tropical hardwoods: $y = 0.0045x - 2.74$, $r^2 = 0.50$.
- European temperate hardwoods: $y = 0.0031x + 11.7$, $r^2 = 0.38$.
- European softwoods: $y = 0.0036x - 2.95$, $r^2 = 0.48$.

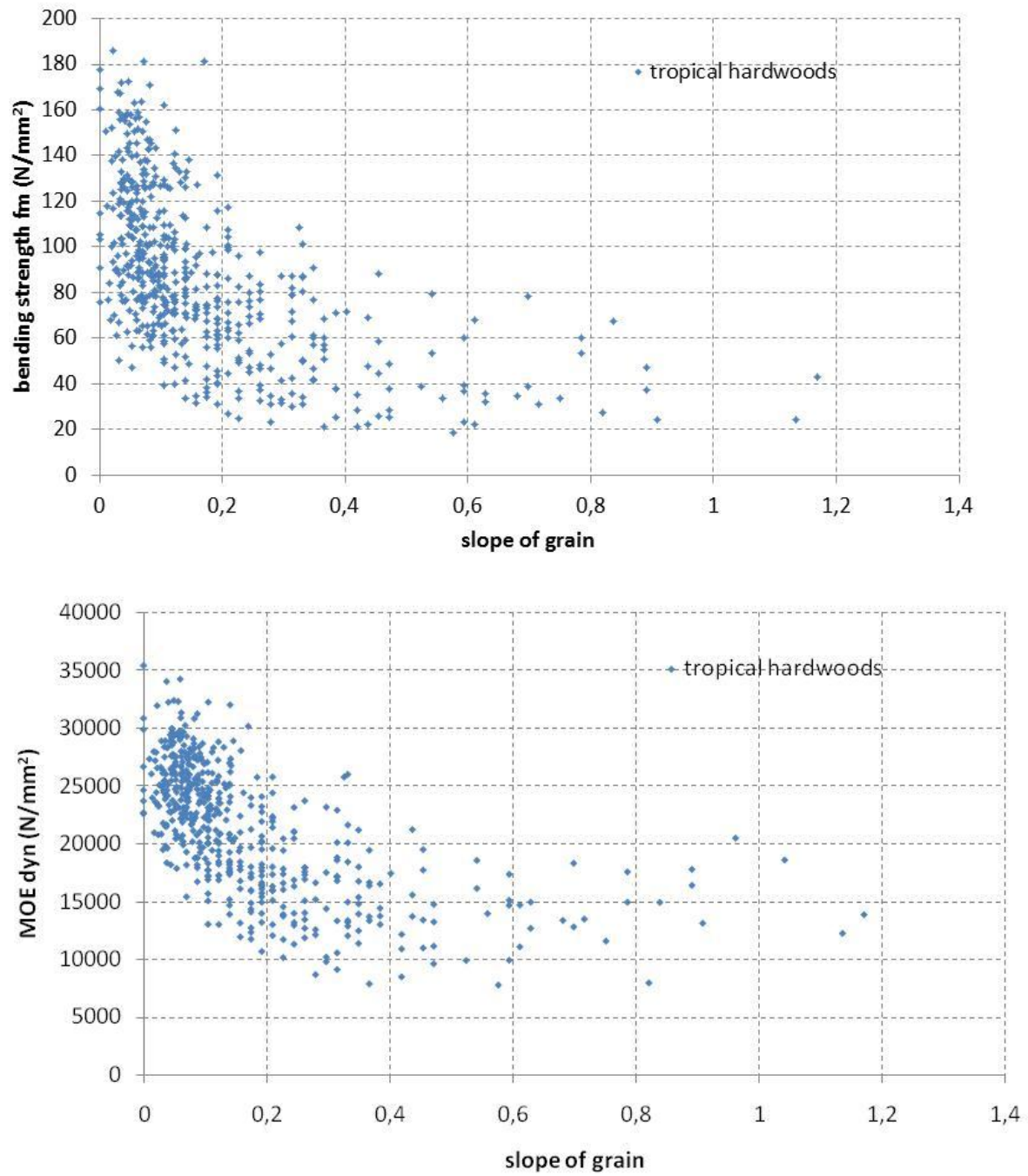


Figure 4.4. Scatterplots of the bending strength (above) and the MOE_{dyn} (below) against the slope of grain (SoG) for tropical hardwoods only.

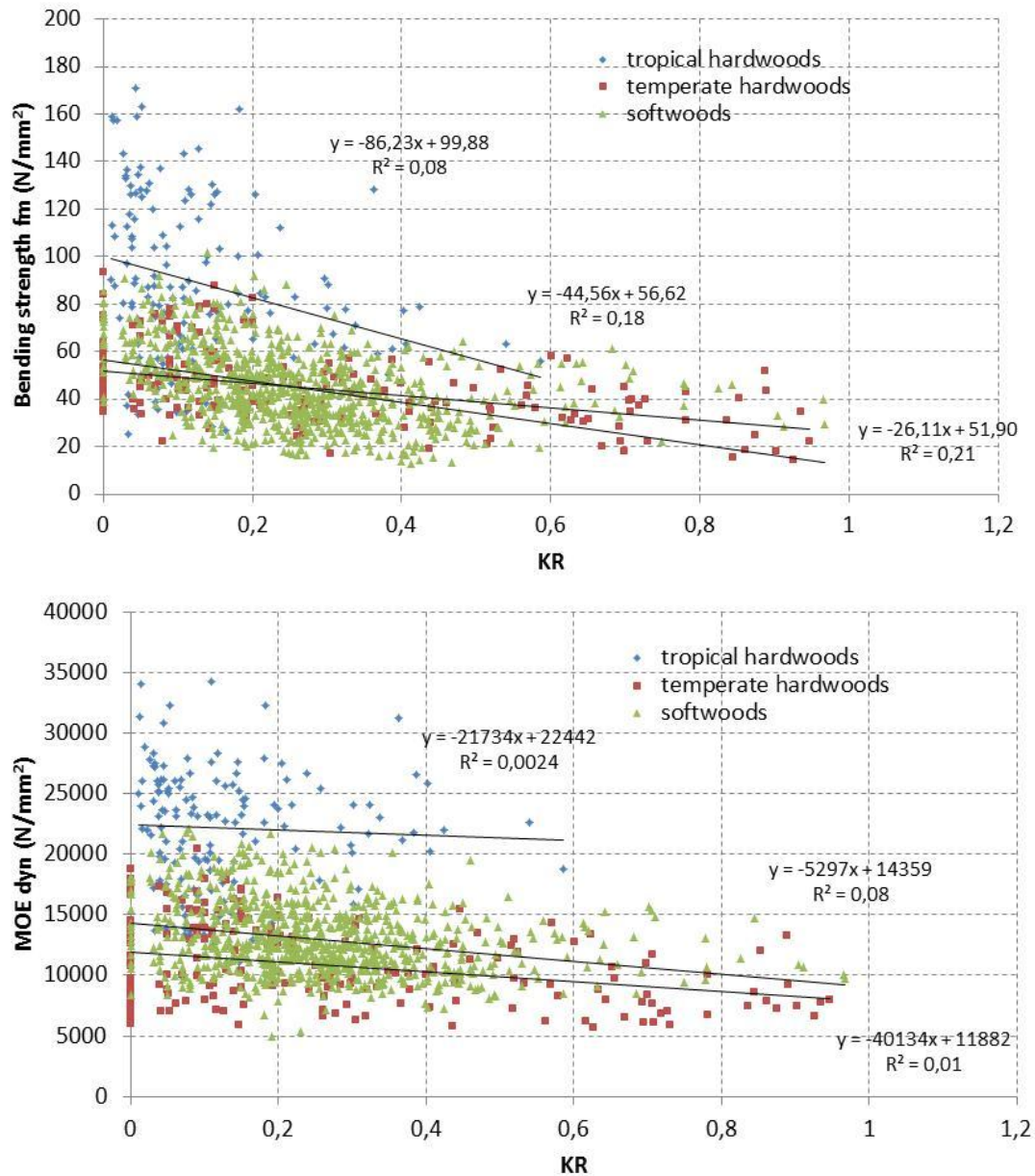


Figure 4.5. Scatterplots of the bending strength (above) and the MOE_{dyn}(below) against the knot ratio for all datasets.

The regression lines and coefficients of determination r^2 that are connected with the datasets are:

Figure 4.5 (above): bending strength against KR.

- Tropical hardwoods: $y = -86,2x + 99,9$ $r^2 = 0,08$.
- European temperate hardwoods: $y = -44,6x + 56,6$ $r^2 = 0,18$.
- European softwoods: $y = -26,1x + 51,9$ $r^2 = 0,21$.

Figure 4.5 (below): MOE_{dyn} against KR.

- Tropical hardwoods: $y = -21734x + 22442$ $r^2 = 0,0024$.
- European temperate hardwoods: $y = -40134x + 11882$ $r^2 = 0,01$.
- European softwoods: $y = -5297x + 14359$ $r^2 = 0,08$.

The regression values are in N/mm².

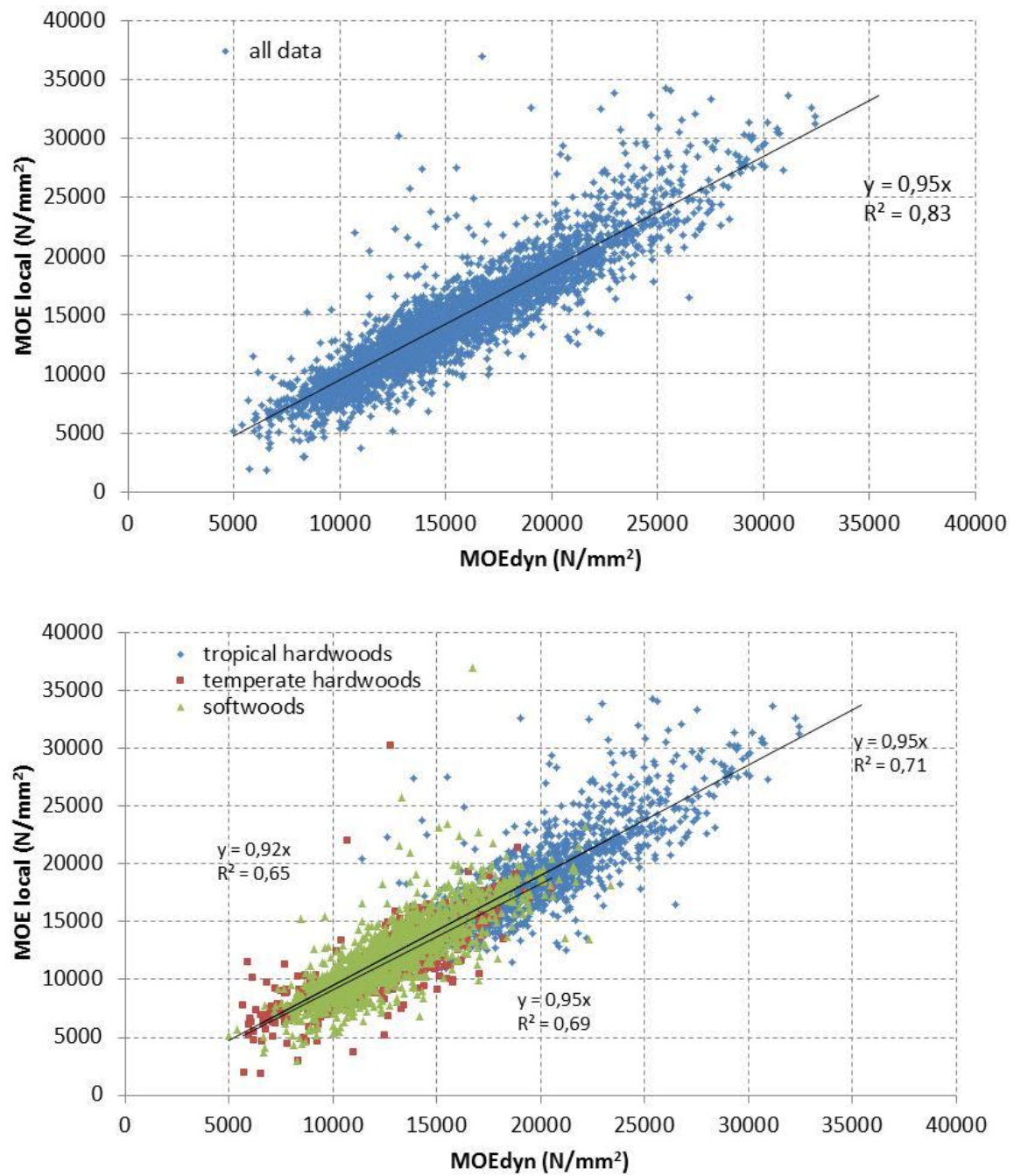


Figure 4.6. Scatterplots of the MOE_{local} against the MOE_{dyn} .

The regression lines and coefficients of determination r^2 that are connected with the datasets are (the regression lines are forced through the origin):

- Tropical hardwoods: $y = 0.95x$. $r^2 = 0.71$.
- European temperate hardwoods: $y = 0.92x$. $r^2 = 0.65$.
- European softwoods: $y = 0.95x$. $r^2 = 0.69$.

The regression values are in N/mm².

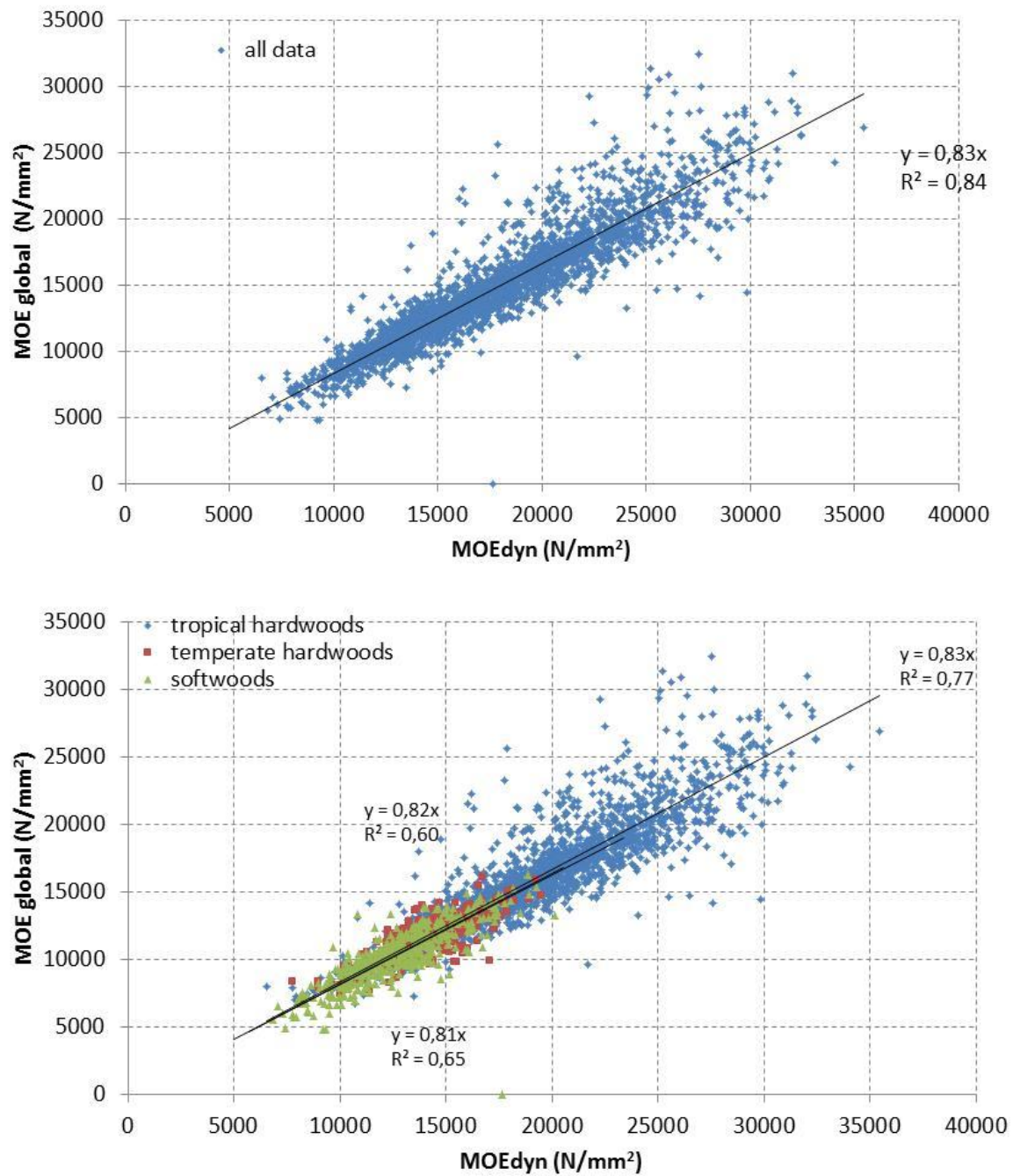


Figure 4.7. Scatterplots of the MOE_{global} against the MOE_{dyn} .

The regression lines and coefficients of determination r^2 that are connected with the datasets are (the regression lines are forced through the origin):

- Tropical hardwoods: $y = 0.83x$. $r^2 = 0.77$.
- European temperate hardwoods: $y = 0.82x$. $r^2 = 0.60$.
- European softwoods: $y = 0.81x$. $r^2 = 0.65$.

The regression values are in N/mm².

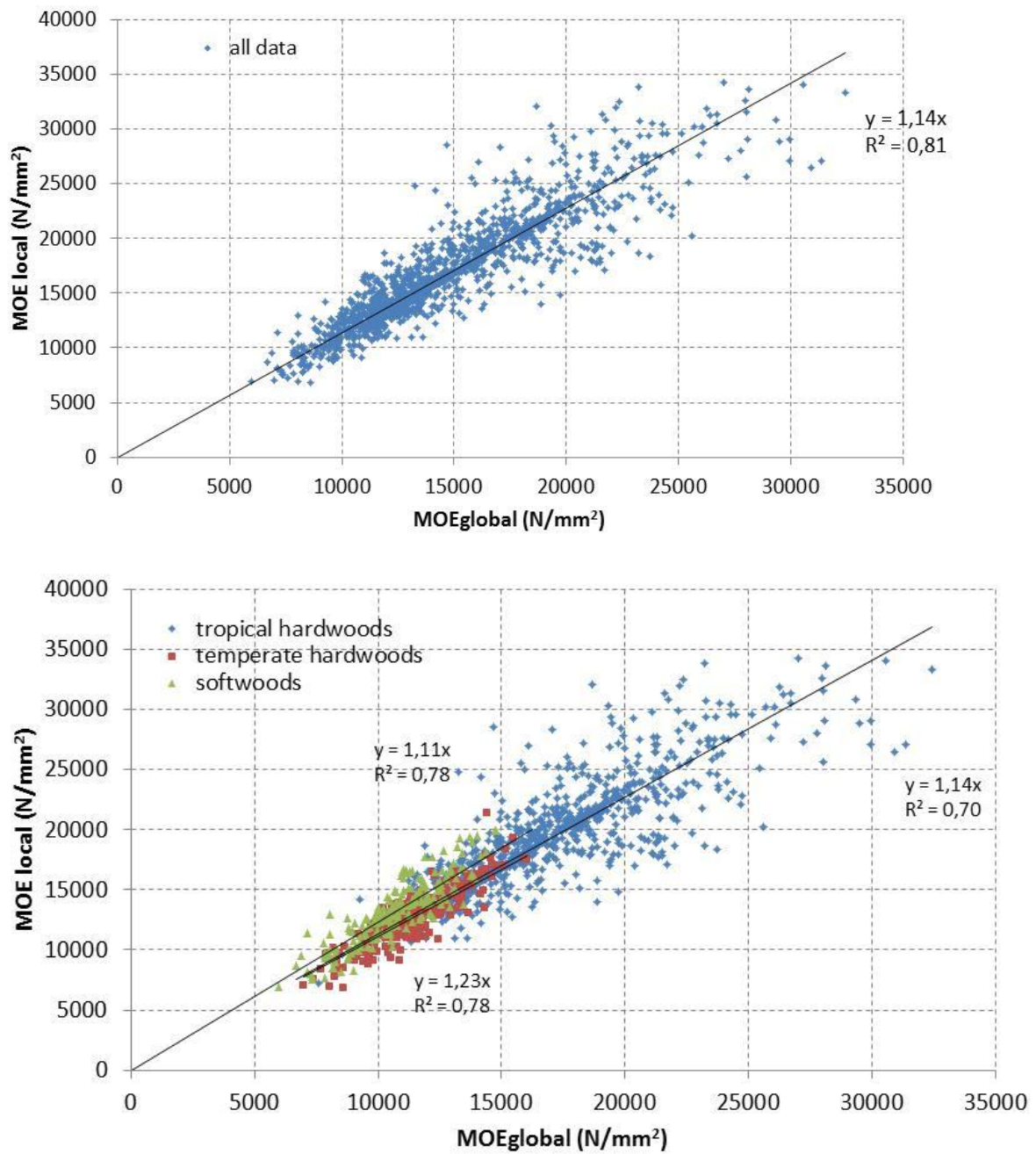


Figure 4.8. Scatterplots of the MOE_{local} against the MOE_{global} .

The regression lines and coefficients of determination r^2 that are connected with the datasets are (the regression lines are forced through the origin):

- Tropical hardwoods: $y = 1.14x$. $r^2 = 0.70$.
- European temperate hardwoods: $y = 1.23x$. $r^2 = 0.78$.
- European softwoods: $y = 1.11x$. $r^2 = 0.78$.

The regression values are in N/mm^2 .

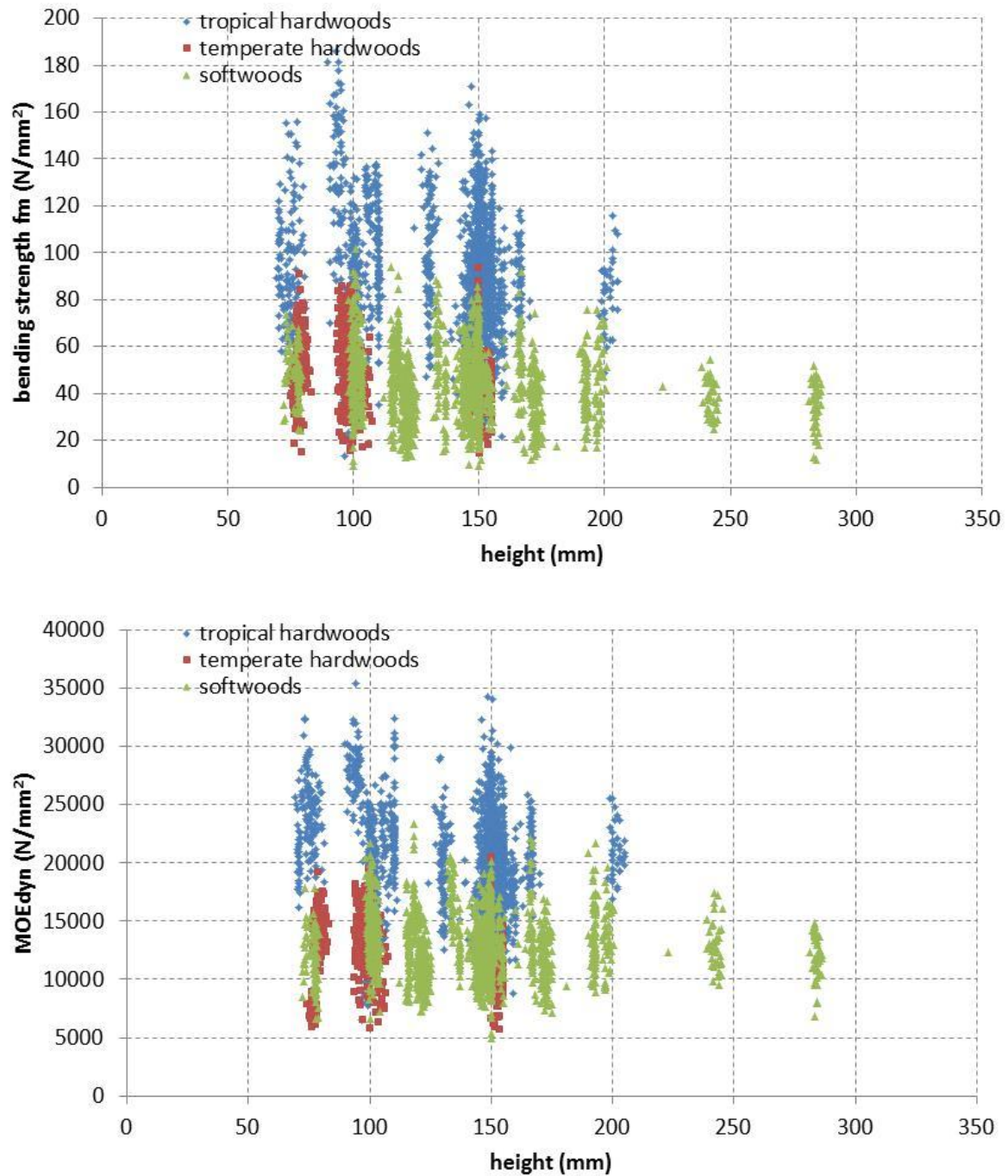


Figure 4.9. Scatterplots of the bending strength f_m (above) and the MOE_{dyn} (below) against the height of the test specimens for all datasets.

The scatterplots show that the relationship of the visually measured properties knot ratio and slope of grain with the bending strength and MOE give low correlations with a large spread. From figure 4.4 (bending strength and MOE_{dyn} against the slope of grain) it is clear that increasing slope of grain for tropical hardwoods has a reducing effect on the bending

strength and MOE_{dyn} , but that the scatter is too large to be used in (species independent) grading. The main reason for this is that the measurement of the exact slope of grain (even after the destructive tests) is very difficult.

This is explained in figures 4.10 and 4.11. It must be emphasized that the measurements shown in figures 4.10 and 4.11 were made in the laboratory, and are not an assessment in practice where it is only decided whether the slope of grain of a beam is within accepted limits or not. For the grading in practice, all pieces were assessed to meet the requirements for the visual grade.

The slope of grain is measured before (denoted as α_2 in figure 3.7.) and after the bending test (denoted as α_1 in figure 3.7.) Since the fracture line follows the grain direction, the slope of grain measured after the bending test is the most accurate one and regarded as the real slope of grain. Figure 4.10 shows two things. Firstly that the slope of grain measured after the bending test cannot be accurately predicted from measurements before the bending test. Secondly that the slope of grain after the bending test can be very large compared to the accepted values (maximum measured slope of grain after the bending test is 0.45, where 0.1 is the limit). When this was noticed in practice, the pieces with a slope of grain higher than 0.1 before testing should be removed from the analysis. However, because of the low predictability, this would not have had affected the outcome of the analysis of the strength properties at all.

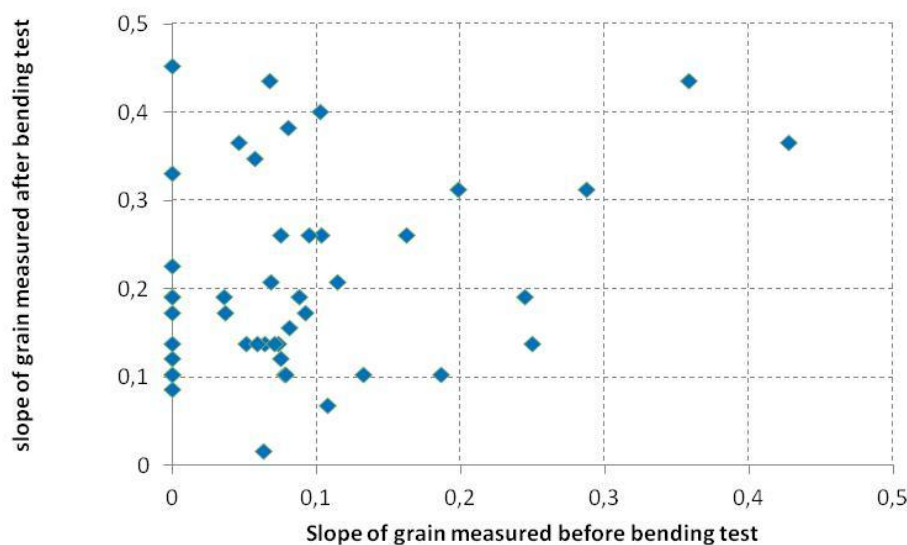


Figure 4.10 Slope of grain measured before and after the bending test for sample OK3 (okan)

Figure 4.11 shows pieces of greenheart, sample GR4. These pieces were cut from pieces from sample GR3, so already a better indication of the slope of grain could be obtained before testing the pieces of sample GR4. This results in a somewhat better prediction capability, but still there is a large scatter.

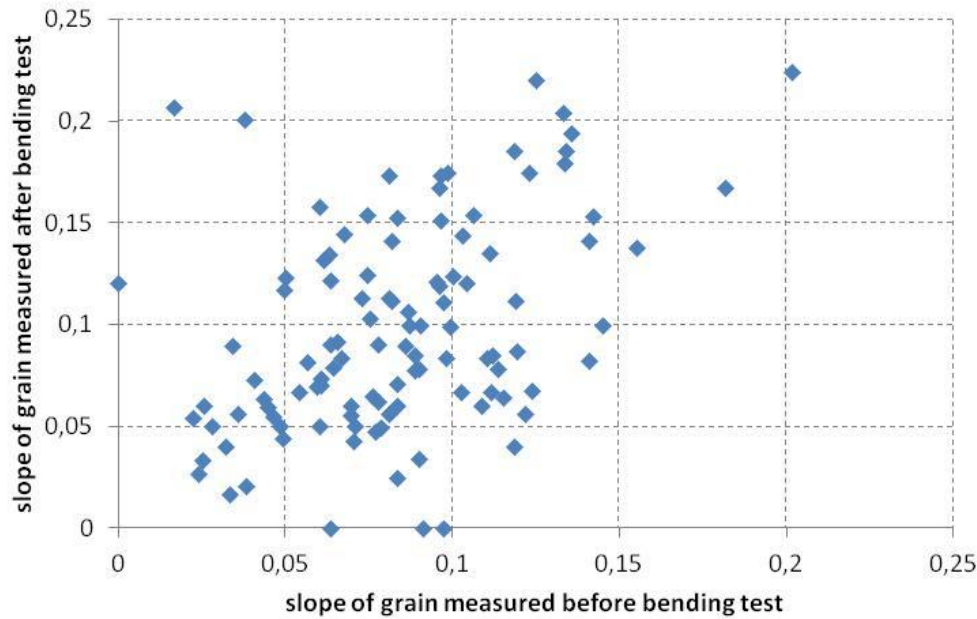


Figure 4.11 Slope of grain measured before and after the bending test for sample GR4 (greenheart)

In figure 4.12, an example of the difficulty of visual grading of tropical hardwoods concerning slope of grain in practice is given. Three beams with the commercial name eyoum (test results are not included in this thesis) are shown. Assessment of the slope of grain is very difficult; all beams would probably pass the visual grading. By measuring the density and the MOE_{dyn} , the slope of grain can be recalculated with the Hankinson equation. Although the density-values of the 3 beams are very similar, the MOE_{dyn} varied between 16600 N/mm² and 22700 N/mm². When the slopes of grains are recalculated with realistic constants in the Hankinson equations this leads to values between 0.08 and 0.17. These differences will have a significant influence on the bending strength values from a destructive test. For realistic constants in the Hankinson equation for the bending strength, for a slope of grain of 0.17, a bending strength value of approximately 65% of the bending strength value for a slope of grain of 0.08 is expected. For the tests performed on the presented beams, this percentage was $100 \cdot (93.5/133.1) = 70\%$. This shows that a wrong assessment of the slope of grain can have a significant effect on the actual strength compared with the expected strength.

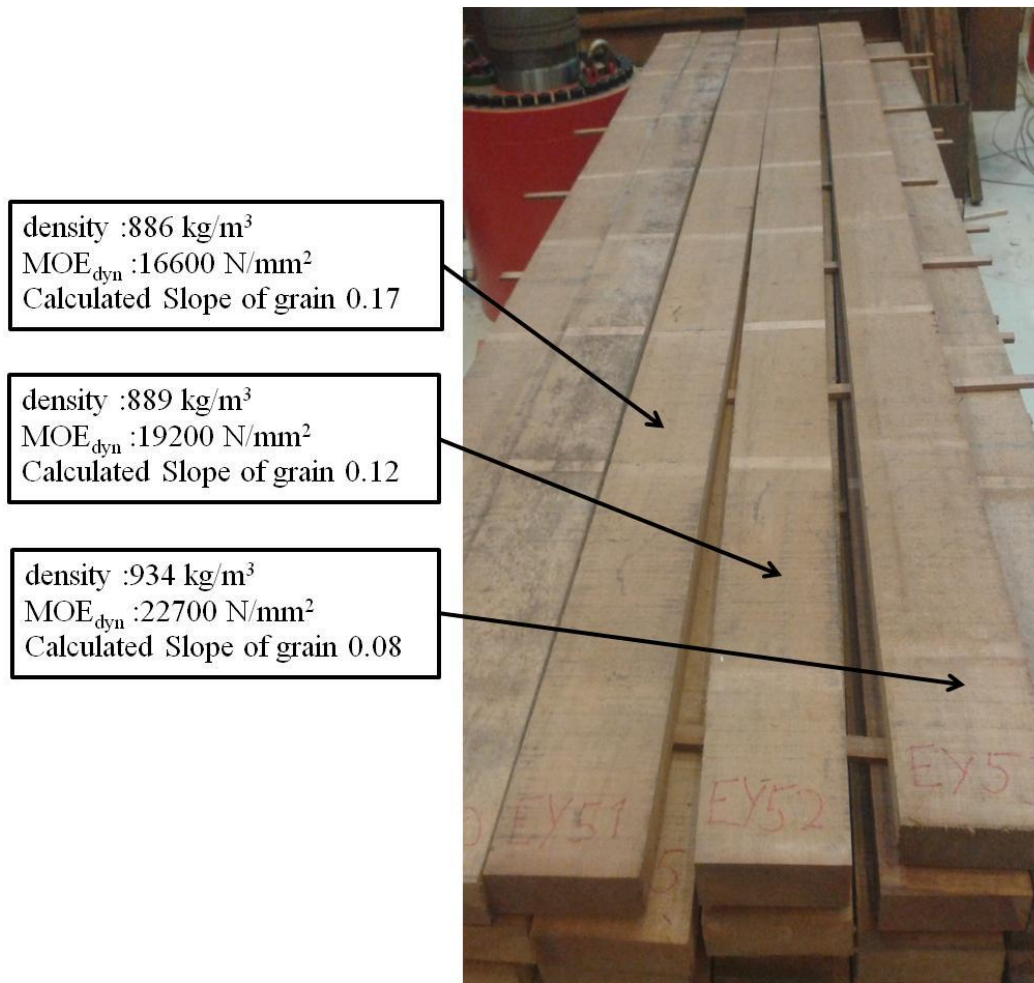


Figure 4.12. Measured density, MOE_{dyn} and estimated slope of grain for three Eyoum beams

From evaluation the slope of grain measurements it can be concluded that it is very difficult to determine the slope of grain values from visual assessment. This does not mean that the slope of grain is not the cause of the reduction in strength and stiffness, only that it is not measurable in a way that it can be used in grading. This will be further investigated in chapter 5.

The slope of grains addressed in figures 4.10, 4.11 and 4.12 are measured on the large size of the specimen (parallel to the load direction in the bending test). So, these are 2D measurements. For the sample of GR 4 (figure 4.11) the slope of grain of each piece was measured on all sides, so parallel and perpendicular to the loading direction. With these measurements the 3D slope of grain could be calculated. However, this calculated 3D slope of grain gave worse correlations with the bending strength than the 2D slope of grain. The reason for this is the difficulty of performing accurate slope of grain measurements, even after failure of the specimen in the test.

Figure 4.3 shows that the MOE_{dyn} has a good potential to be used as a species independent grading parameter for structural timber, although it must be investigated whether the prediction capability and scatter around the regression line is the same for all datasets. The relationships and scatter, however, are different than for small size clear wood (compare with figure 2.7). These differences must be investigated.

Comparing figures 4.1 and 4.2 with figures 2.5 and 2.6 for small size clear wood it becomes clear that for structural timber the density as a single parameters is not a good predictor for strength and stiffness. There is certainly a trend that the strength and stiffness increase with increasing density, but because there is a lot of scatter, density on its own is not suitable for (species independent) grading for structural timber.

Figures 4.6 and 4.7 show that there is a good correlation of the MOE_{dyn} with both the MOE_{local} and MOE_{global} , and that these relations can be assumed to be species independent. In practice, MOE_{dyn} will be used as a grading parameter. MOE_{local} is the parameter the value of which has to be evaluated against the strength class values. Where no MOE_{dyn} – values are available, they will be calculated from the values of MOE_{local} or MOE_{global} . When there are no MOE_{local} values, they will be calculated from MOE_{global} . The following equations will be applied. These equations can be used in both directions.

$$MOE_{local} = 0.95 MOE_{dyn} \quad (4.1)$$

$$MOE_{global} = 0.83 MOE_{dyn} \quad (4.2)$$

$$MOE_{local} = 1.14 MOE_{global} \quad (4.3)$$

Because the influence of moisture content is expected to be the same for all three of these MOE measurements, equations (4.1), (4.2) and (4.3) are applicable at every moisture content.

4.3 Adjustments of basic test data to reference moisture content and size.

For the strength classes defined in EN 338, a reference moisture content of 12% is used and a reference height of 150 mm. The test results therefore have to be adjusted to these reference values.

Density

The equations for the adjustment of the density are given in section 3.2.1

MOE

For the MOE, no size effect is expected according to the Weibull theory, because the MOE is determined in the elastic range of the test, and not at failure.

To determine the influence of moisture content on the MOE, in principle two similar samples with different moisture contents could be compared. In practice, however, this is not possible, because it is difficult to find two samples that are similar in slope of grain and in density. Therefore, from 54 pieces of sample GR4 and 25 pieces of sample MAS5 the MOE_{dyn} was measured at high moisture content, and also after the pieces were dried. The results are given in table 4.10. All values given are mean values.

Table 4.10. Mean values for MOE_{dyn} at two moisture content levels

Sample ID	m.c.dry (%)	MOE_{dyn} dry (N/mm ²)	m.c.wet (%)	MOE_{dyn} wet (N/mm ²)	$MOE_{dyn,dry}/$ $MOE_{dyn,wet}$	n=
GR4A	15.0	26800	25.0	23900	1.12	36
GR4B	15.0	24800	25.0	22300	1.11	18
MR5	12.0	25800	35.9	22000	1.17	25

Sample GR4 contained specimens without knots, and specimens with knots. They were divided in GR4A for the pieces with knots and GR 4B for the pieces without knots. Table 4.10 shows that the mean MOE_{dyn} is lower for the pieces with knots, but that the ratio between the $MOE_{dyn,dry}$ and $MOE_{dyn,wet}$ is practically the same.

The adjustment equation to be used is:

$$MOE_{dyn,12\%} = MOE_{dyn,mc} / (1 - k_{mc} \frac{\min(m.c.; 25.0) - 12}{13}) \quad (4.4)$$

For $k_{mc} = 0.13$ for samples GR4A and GR4B a ratio of 1.11 is found between the calculated $MOE_{dyn,12\%}$ and $MOE_{dyn,wet}$ and for sample MR5 a ratio of 1.15. The value of 0.13 is therefore on the safe side and will be used in equation (4.4) to adjust the MOE_{dyn} values to 12%. This equation will also be applied to the MOE_{local} and MOE_{global} .

Bending strength.

Evaluating the top plot of figure 4.9 there seems to be no clear size effect for structural timber, at least not for size as a parameter on its own. To investigate the size effect on structural timber, similar groups have to be formed and compared. Based on visual characteristics, this is very difficult as explained before. Similar groups can be made by creating a model with the properties that predict the bending strength well, like the MOE . This will be investigated in chapter 5 (section 5.3) where for specific wood species more data with different dimensions and moisture contents are prepared, tested and analysed. Anticipating on chapter 5, the outcomes will already be presented here, to make it possible that these adjustments can already be used to analyse results of visual grading.

The outcome of the analysis is that there is no significant size effect for structural timber of sizes larger than 75 mm. This is valid for both softwoods and hardwoods.

As to the moisture content correction, there is not enough data on structural timber with high moisture content for softwoods and temperate hardwoods to be included in the modelling.

In Green and Evans (2001) an overview is given of the adjustments for changing moisture content for several properties for clear wood and structural softwood timber. Where for small sized clear wood the bending strength increases with decreasing moisture content at the 5% fractiles of the distribution, for structural timber it is different. For structural timber it is observed that for low values of the 5% fractiles there is almost no difference with changing moisture content, whereas for higher values of the 5% fractiles an increase is observed for decreasing moisture content.

An explanation for the fact that the 5% fractiles almost does not change for low values is that these pieces contain larger sized knots and the influence of these knots on the failure strength is much larger than the influence of the moisture content.

For that reason in EN 384 no adjustment factor for moisture content is allowed on the bending strength. Also in this thesis for the softwood and temperate hardwood species, which contain knots, no adjustment is made for the bending strength values.

For tropical hardwoods, where knots are very rare and the slope of grain is the main failure mechanism, the influence of moisture content might have an influence. The outcome of the modelling in chapter 5 confirms this. The outcome is the following adjustment equation, which is valid for the pieces of tropical hardwoods:

$$f_{m_{12\%}} = f_{m_{mc}} / (1 - k_{b,mc} \frac{\min(m.c.; 25.0) - 12}{13}) \quad (4.5)$$

For $k_{b,mc}$ a value of 0.15 is found. This value will be used for adjustments for tropical hardwoods.

4.4 Adjustments test data to a reference moisture content of 12% and a height of 150 mm.

In this section, the basic test data is adjusted to a reference moisture content of 12% and a height of 150 mm. The adjustment factors presented in section 4.3. have been used.

The mean and standard deviations for the bending strength, the MOE_{loc} , the MOE_{dyn} and the density are presented for the datasets of European softwoods, European temperate hardwoods and tropical hardwoods.

Table 4.11. Properties of tropical hardwoods adjusted to 12% m.c. Mean ($\bar{x}$) and standard deviations (s) of every property.

Sample	f_m (N/mm ²)		MOE_{loc} (N/mm ²)		MOE_{dyn} (N/mm ²)		Density ρ (kg/m ³)	
	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s
AV1	85.8	17.7	16500	1800	19500	1900	1050	50
AV2	121.2	19.5	22200	1800	23400	1900	1120	40
AV3	114.7	26.9	21500	3500	22600	3700	1140	50
AV4	116.4	27.3	23000	3600	24600	3200	1130	50
AV5	93.2	18.4	17000	4400	19500	2900	1110	50
CUM1	113.6	24.2	20000	3100	23100	3300	1040	60
CUM1	112.4	25.3	20300	4600	21700	4200	990	80
CUM3	124.4	16.2	21800	2800	22300	2500	1040	80
CUM4	90.9	19.1	19000	3200	21200	3200	930	70
CUM5	84.2	19.7	18400	3500	20400	2700	940	60
MAS1	138.0	20.8	26900	9300	23600	2600	1060	40
MAS2	104.9	18.5	20500	2700	21500	2500	1040	50
MAS3	68.2	16.2	13500	2300	14300	2500	1000	40
MAS4	130.6	24.0	25800	3500	27200	3700	1070	60
MAS5	117.1	30.6	24900	2900	26200	3100	1050	60
AZ1	110.3	17.3	22300	2900	23500	3100	1030	20
AZ2	94.9	14.9	19400	3100	20300	2300	1000	60
AZ3	131.3	23.7	24900	3500	24400	3100	1030	30
GR1	119.8	14.4	25500	3000	26800	3100	1080	50
GR2	104.3	22.6	26200	3900	26300	3800	1000	60
GR3	92.5	20.3	30600	3700	29700	3300	980	50
GR4	100.2	27.8	26600	4900	26400	2800	1000	50
OK1	81.7	12.8	18200	5500	19200	1900	960	80
OK2	60.3	18.3	17600	3700	18600	3900	1040	60
OK3	85.9	19.5	19300	3100	20300	3300	920	150
KA1	77.2	10.9	18100	2700	21700	2600	740	60
NA1	77.9	14.1	18700	4100	17500	2000	730	70
PI1	74.4	16.4	21400	6500	18600	2700	830	80
VI1	68.6	11.4	15100	2400	15700	1800	770	60
BAS1	72.0	19.9	20300	6100	20800	2300	750	50
BAN1	95.9	20.3	21200	3800	21800	2700	920	60
SV1	87.5	18.7	18700	5000	19900	4900	890	60
CR1	93.1	10.9	18100	1600	19100	1700	830	50
LA1	89.3	17.0	18000	3300	18900	3500	780	90
LF1	93.0	15.6	22900	3200	24100	3400	920	50
PU1	89.3	17.7	19600	2300	20400	1800	800	100
TV1	66.9	13.4	15900	3600	16600	2500	710	30
FA1	89.8	22.5	19800	2400	21600	2800	770	60
FP1	98.0	18.5	20900	2600	22700	2800	890	80
SA1	76.9	16.0	16600	3400	18200	2800	700	50
BIL1	42.8	10.8	14000	1200	14700	1300	770	30
BIL2	48.7	18.1	12800	3200	13500	3400	730	50
EV1	128.3	28.5	26600	1500	28000	1500	1040	40
EV2	105.8	16.4	22800	2100	24000	2300	990	50
TA1	73.9	15.3	17500	1400	18400	1500	890	60
TA2	70.8	18.4	17600	2000	18500	2100	930	50

Table 4.12. Properties of temperate European hardwoods adjusted to 12% m.c.

batch	f_m (N/mm ²)		MOE_{loc} (N/mm ²)		MOE_{glob} (N/mm ²)		Density ρ (kg/m ³)	
	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s
O1	42.0	6.8	10700	1900	12900	2000	620	30
O2	38.8	10.4	9800	4000	10300	2800	680	70
R1	66.1	14.8	17000	2100	17400	2000	700	40
C1	55.1	14.1	13000	2300	14300	2000	580	50

Table 4.13. Properties of European softwoods adjusted to 12% m.c.

batch	f_m (N/mm ²)		MOE_{loc} (N/mm ²)		MOE_{dyn} (N/mm ²)		Density ρ (kg/m ³)	
	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s	$\bar{x}$	s
S1	37.0	10.4	10600	2400	11600	2000	440	40
S2	41.8	14.4	12000	2900	13200	2500	440	50
S3	38.2	12.4	12300	2400	12900	2500	450	40
S4	42.9	13.1	11000	2500	11800	2300	430	40
S5	41.9	8.2	14200	2900	13700	2300	470	50
S6	42.1	10.5	13200	2800	13100	2300	440	50
S7	62.5	9.6	16900	2100	17200	2300	500	50
S8	46.3	10.4	8700	2500	11100	2100	440	40
D1	48.6	17.0	13500	3300	14300	3100	570	60
L1	48.4	10.5	10600	2500	10600	2000	520	60
L2	55.7	14.2	15100	4600	14700	2200	610	60

In figure 4.13, bending strength values adjusted to 12% m.c. are plotted against the density values adjusted to 12% m.c.

In figure 4.14, MOE_{dyn} values adjusted to 12% m.c. are plotted against the density values adjusted to 12% m.c.

In figure 4.15, MOE_{dyn} values adjusted to 12% m.c. are plotted against the bending strength values adjusted to 12% m.c.

The prediction capability of the MOE_{dyn} for the bending strength at 12% is comparable with that of the basic test results. It is clear that the MOE_{dyn} has the greatest potential as a strength predicting parameter. Whether this can be totally species independent will be investigated in chapter 5.

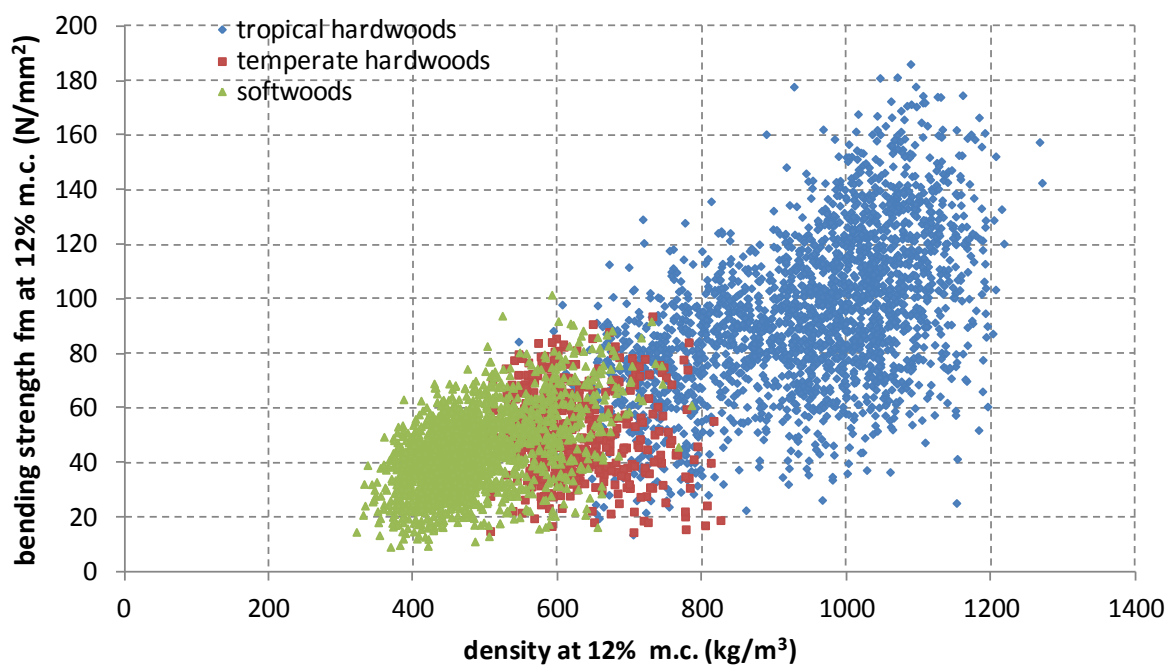
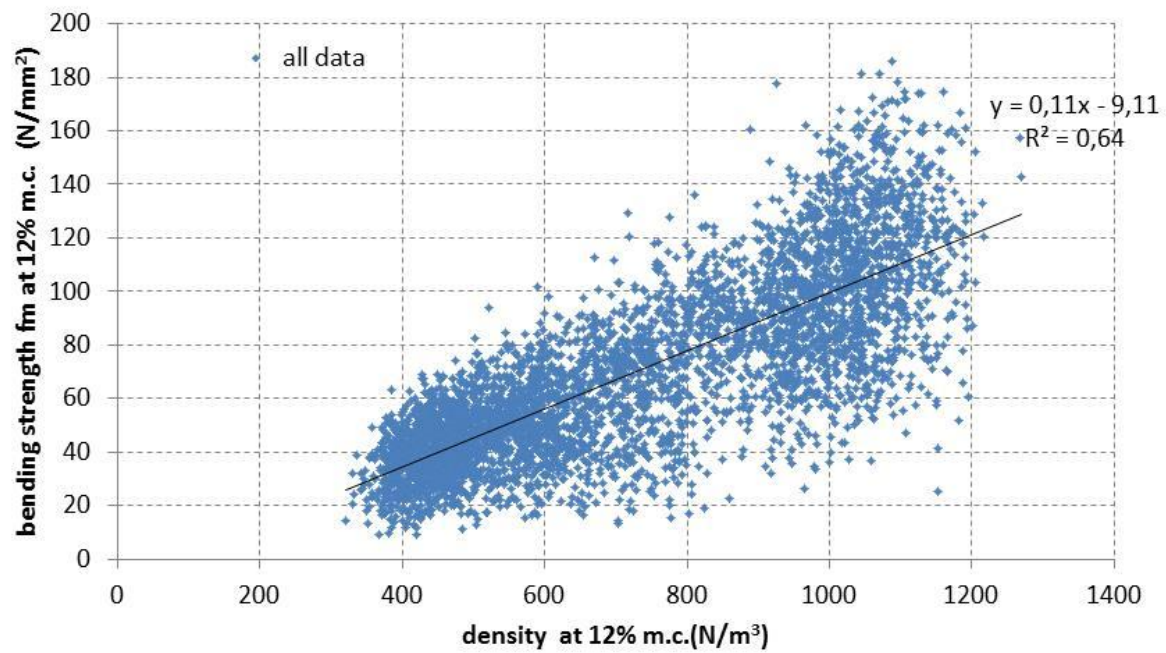


Figure 4.13. Scatterplots of the bending strength at 12% m.c. against the density at 12% m.c.

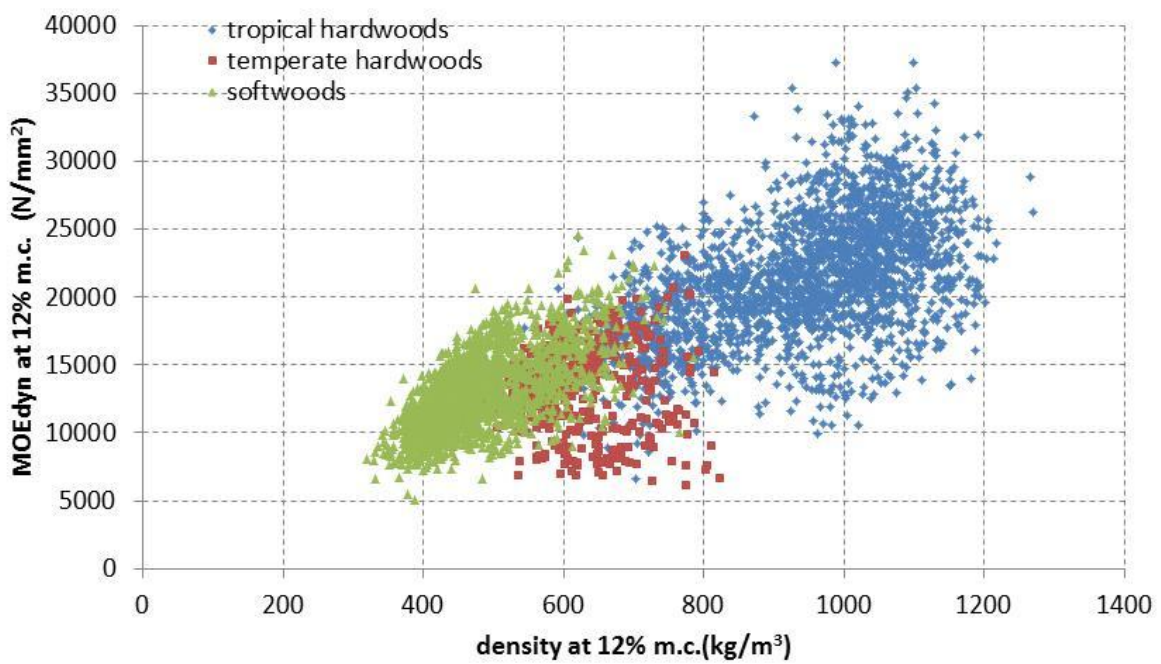
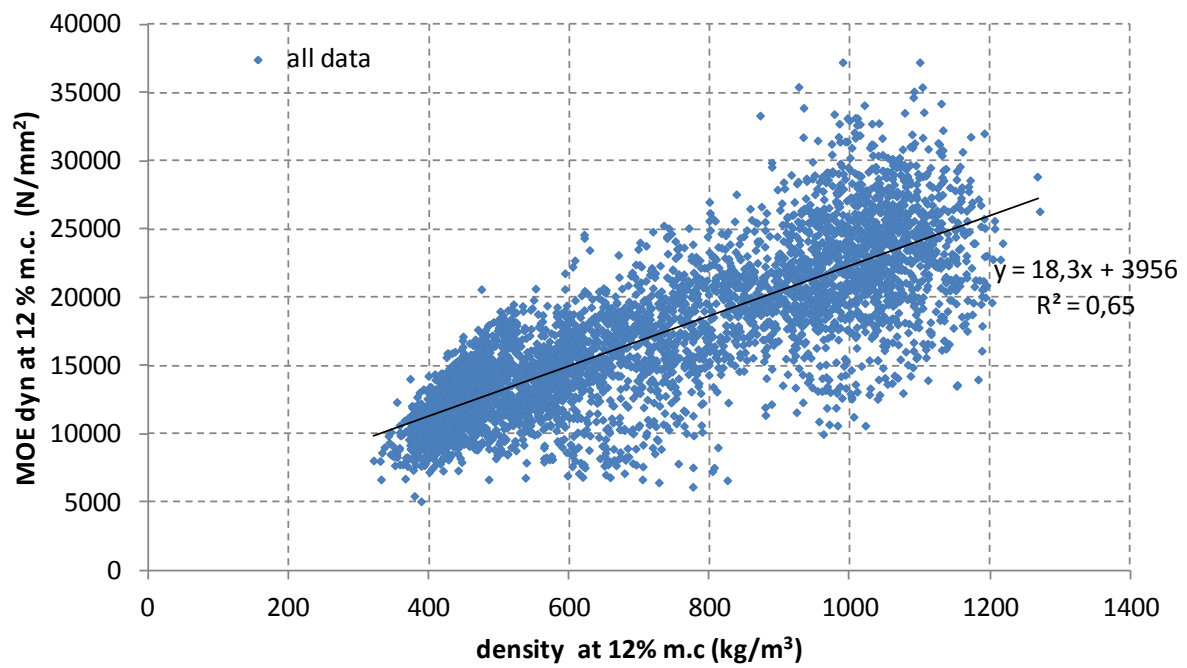


Figure 4.14. Scatterplots of the MOE_{dyn} at 12% m.c. against the density at 12% m.c.

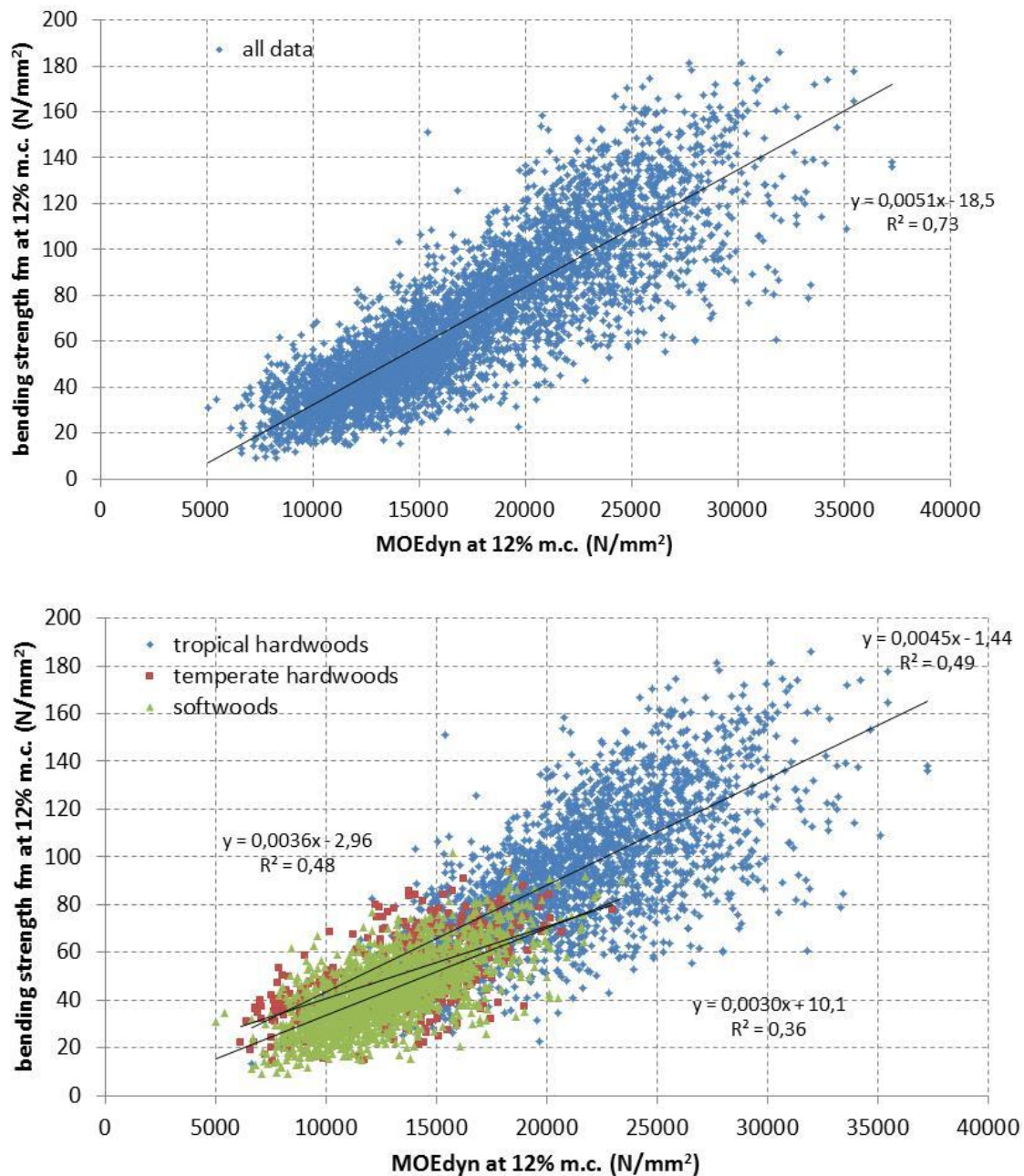


Figure 4.15. Scatterplots of the bending strength at 12% m.c. against the MOE_{dyn} at 12% m.c.

The regression lines and coefficients of determination r^2 that are connected with the datasets are:

- Tropical hardwoods: $y = 0.0045x - 1.44$. $r^2 = 0.49$.
- European temperate hardwoods: $y = 0.0030x + 10.1$. $r^2 = 0.36$.
- European softwoods: $y = 0.0036x - 2.96$. $r^2 = 0.48$.

4.5 Calculation of characteristic values for visual grading of tropical hardwood timber species

The objective of this thesis is to investigate species independent grading of structural timber. One aspect to be investigated is the method to perform this species independent grading. However, in section 4.2 it was concluded that visual grading is not suited to perform species independent grading on pieces where grain angle deviation is the governing strength reducing characteristic. This is caused by the fact that the grain angle deviation cannot be measured unambiguously by means of a visual assessment (for knots as governing strength reducing characteristic this is possible). This observation is not only valid for species independent visual grading of tropical hardwoods, but for visual grading of tropical hardwoods in general (also for a single species). To take into account that the grain angle deviation cannot be measured exactly, but also that batches with the same trade name might in fact come from trees of different species, a reduction factor $k_{s,tn}$ was introduced in section 3.3, which has to be applied on the characteristic value (the 5% fractile) of the bending strength. The idea is that the 5% fractile of the test data available is determined and that by applying $k_{s,tn}$ the characteristic value of the timber with that trade name can be determined. In section 3.3 it was concluded that with the derived factor $k_{s,tn}$ it would not be possible to classify tropical hardwoods in strength classes in an economic way to be used by visual grading. It is shown in chapter 5 that with machine grading this is possible.

However, since in practice visual grading of tropical hardwoods is still used, it is of interest to compare the output in terms of yields of the two methods for grading, visual and machine grading. Therefore, it is decided to calculate the 5% fractile of the bending strength with the statistical methods of section 3.3, based on the available samples for each trade name without applying the $k_{s,tn}$ -factor. This is only to be able to make a comparison between the yield of the material that is visually graded and machine graded.

The principle of both the statistical methods of section 3.3 based on visual grading and section 3.4 for machine grading is that every batch graded according to either method has the strength properties that are assigned to these batches. This seems obvious, but according to the current standard EN 384 for determining characteristic values based on visual grading this is not the case. There the assigned strength properties are a mean value of the strength properties of the batches.

In this section the characteristic values of visual grades for tropical hardwood timber species are calculated. For tropical hardwoods, only one visual grade is defined (in this thesis Dutch visual grade C3 STH from NEN 5493 is used), with as most important characteristics that have to be limited:

1. single knot ratio < 0.2
2. slope of grain < 1:10

Furthermore, all pieces were assessed on the visual override (see appendix B)

All the data that was used in the analysis met the visual grade C3 STH from a visual inspection before testing, except for a part of the pieces in sample GR4 which were deliberately prepared for research purposes. These pieces are shown in figure 4.5 (top) having a knot ratio larger than 0.2. These pieces were left out of the analysis for determining the characteristic values of the visual grade.

In figure 4.4, the measured slope of grain after the destructive tests is shown. This plot shows that for a large amount of pieces the slope of grain was much larger than 1:10. However, this was for pieces included in the analysis not detected in the visual grading process before destructive testing. The conclusion is that in practice it is not possible to apply this requirement on the slope of grain limitation in the grading process. Therefore in the analysis, all pieces were included, since the slope of grain of the pieces as shown in figure 4.4 was measured after destructive testing.

The characteristic density will be calculated from the weakest homogeneous samples based on the bending strength, where it is assumed that in this way the variation in the density is addressed

For the modulus of elasticity, the characteristic value is the mean value of the weakest homogeneous samples.

The characteristic values are defined as:

- for the bending strength the 5% fractile at 12% m.c.
- for the *MOE* the mean value at 12 % m.c.
- for the density the 5% fractile at 12 % m.c.

The 5% fractile of the bending strength of a timber species is calculated according to the method described in section 3.3. Normal distributions of the samples are assumed.

Determination of characteristic values for samples with trade name cumaru.

To explain the principle of the calculation procedures, this is elaborated for the timber with the trade name cumaru. The assignments always relate to a source area. When all 5 samples are used in the analysis, the source area is Brazil, Peru and Bolivia.

In table 4.14, the descriptive statistics of the samples and the 5% fractile of the samples based on 75% confidence interval are given.

Table 4.14. Descriptive statistics for cumaru samples CUM1 to CUM5

sample ID	$\bar{x}$	s	n=	k=	$f_m 0.05$
CUM1	113.6	24.2	40	1.83	69.2
CUM2	112.4	25.3	44	1.83	66.3
CUM3	124.4	16.2	47	1.82	94.9
CUM4	90.9	19.1	40	1.83	55.8
CUM5	84.2	19.7	49	1.81	48.5

The weakest homogenous samples of which the characteristic values will be calculated have to be determined.

In tables 4.15 to 4.18 the results of the intermediate steps are given. In column O the percentage of observed pieces with a value below the overall mean value $f_{m_{mean}}$ is listed for every sample included in this analysis step. The expected values in column E are the summed observed percentages from column O , divided by the number of samples. In the last row, the significance values of the chi-square distribution of value Z with the number of samples-1 degrees of freedom is given. In every step, the strongest remaining sample (with the highest 5% fractile according to table 4.15) is removed.

This is repeated until the significance is above 0.01.

Table 4.15. Step 1. $f_{m_{mean}}$ of the samples is 105.0 N/mm².

sample ID	O (%)	E (%)	$(O-E)^2/E$
CUM1	40.0	51.7	2.6
CUM2	34.1	51.7	6.0
CUM3	12.8	51.7	29.3
CUM4	80.0	51.7	15.5
CUM5	91.7	51.7	30.9
Z			84.3
sig			2.09E-17

Table 4.16. Step 2. $f_{m_{mean}}$ of the remaining samples is 99.8 N/mm².

sample ID	O (%)	E (%)	$(O-E)^2/E$
CUM1	25.0	51.6	13.7
CUM2	31.8	51.6	7.6
CUM4	72.5	51.6	8.5
CUM5	77.1	51.6	12.6
Z			42.3
sig			3.39E-09

Table 4.17. Step 3. $f_{m_{mean}}$ of the remaining samples is 95.6 N/mm².

sample ID	O (%)	E (%)	(O-E) ² /E
CUM2	29.5	53.6	10.8
CUM4	62.5	53.6	1.5
CUM5	68.8	53.6	4.3
Z			16.6
sig			2.5410E-04

Table 4.18. Step 4. $f_{m_{mean}}$ of the remaining samples is 87.2 N/mm².

sample ID	O (%)	E (%)	(O-E) ² /E
CUM4	37.5	47.9	2.3
CUM5	58.3	47.9	2.3
Z			4.5
sig			0.033

Now samples CUM4 and CUM 5 are assumed to be the weakest samples that can be regarded as homogeneous for the determination of the strength and stiffness properties.

The minimum number of pieces in a sample is CUM4, for which $k=1.83$.

Then $k_{N,n} = z_p + (k - z_p)/\sqrt{(2)} = 1.65 + (1.83 - 1.65)/\sqrt{(2)} = 1.78$

The average of the means of the bending strength of CUM 4 and CUM 5 is:

$$(90.9 + 84.2)/2 = 87.6 \text{ N/mm}^2.$$

The average of the standard deviations of the bending strength of CUM 4 and CUM 5 is:

$$(19.1 + 19.7)/2 = 19.4 \text{ N/mm}^2.$$

The 5% fractile of the dataset of cumaru then becomes: $87.6 - 1.78 \cdot 19.4 = 52.9 \text{ N/mm}^2$.

The average of the means of the MOE_{local} of CUM 4 and CUM 5 is:

$$(19000 + 18500)/2 = 18750 \text{ N/mm}^2.$$

The average of the means of the density of CUM 4 and CUM 5 is:

$$(933 + 944)/2 = 939 \text{ kg/m}^3.$$

The average of the standard deviations of the density of CUM 4 and CUM 5 is:

$$(70 + 59)/2 = 64.5 \text{ kg/m}^3.$$

The 5% fractile of the density of cumaru then becomes: $939 - 1.78 \cdot 64.5 = 823 \text{ kg/m}^3$.

When only the samples CUM4 and CUM 5 are evaluated, connected to the source area Peru and Bolivia, then table 4.18 can immediately be evaluated to conclude that these two samples can be assumed to be homogeneous, so the calculated characteristic values remain the same.

Another option is to evaluate cumaru with source area Brazil. The 3 samples CUM1, CUM 2 and CUM 3 are available for the analysis. In table 4.19 the result for the homogeneity test is given.

Table 4.19. Step 1. $f_{m_{mean}}$ of the samples is 117.1 N/mm².

sample ID	O (%)	E (%)	(O-E) ² /E
CUM1	52.5	41.1	3.2
CUM2	40.9	41.1	0.0
CUM3	29.8	41.1	3.1
Z			6.3
sig			0.043

The significance level is 0.043, which is higher than 0.01, so the three samples are assumed to be homogeneous.

The minimum number of pieces in a sample is CUM1 for which $k = 1.83$.

Then $k_{N,n} = z_p + (k - z_p) / \sqrt{(3)} = 1.65 + (1.83 - 1.65) / \sqrt{(3)} = 1.75$

The average of the means of the bending strength of CUM1, CUM2 and CUM3 is:

$$(113.6 + 112.4 + 124.4) / 3 = 116.8 \text{ N/mm}^2.$$

The average of the standard deviations of the bending strength of CUM 4 and CUM 5 is:

$$(24.2 + 25.3 + 16.2) / 3 = 21.9 \text{ N/mm}^2.$$

The 5% fractile of the dataset of cumaru then becomes: $116.8 - 1.75 \cdot 21.9 = 78.5 \text{ N/mm}^2$.

The average of the means of the MOE_{local} of CUM1, CUM2 and CUM3 is:

$$(19971 + 20314 + 21758) / 3 = 20681 \text{ N/mm}^2.$$

The average of the means of the density of CUM1, CUM2 and CUM3 is:

$$(1038 + 992 + 1038) / 3 = 1023 \text{ kg/m}^3.$$

The average of the standard deviations of the density of CUM1, CUM2 and CUM3 is:

$$(60 + 82 + 80) / 3 = 74 \text{ kg/m}^3.$$

The 5% fractile of the density of cumaru of CUM1, CUM2 and CUM3 then becomes:

$$1032 - 1.75 \cdot 74 = 903 \text{ kg/m}^3.$$

Determination of characteristic values for samples of all trade names.

In table 4.20, the calculated characteristic values for the trade names listed in table 3.1, based on the samples available as listed in table 3.1 are given. The connected source areas are listed in table 3.1.

Once again, it is mentioned that the characteristic values as given in table 4.20 only apply to the tested samples of the trade name and not to the timber from the trade name in general, because then a reduction factor $k_{s,tn}$ according to equation (3.32) has to be applied. The table can be interpreted as the characteristic values of the tested samples of each trade name with 100% yield of the visually graded pieces assigned to the visual grade.

Table 4.20. Characteristic values for visual grading for the tested samples of trade names of tropical hardwood timber

Trade name	homogeneous samples	$k_{N,n}$	$fm_{0.05 \text{ hom. samples}}$ (N/mm ²)	$MOE_{loc, mean, \text{ hom. samples}}$ (N/mm ²)	$density_{0.05, \text{ hom. samples}}$ (kg/m ³)
angelim vermelho	AV1,AV5	1.78	57.3	16700	990
cumaru	CUM4,CUM5	1.78	53.0	18700	820
massranduba	MAS3	1.81	38.9	13500	930
azobe	AZ2	1.78	68.4	19400	900
greenheart	GR2,GR3,GR4	1.79	58.3	27800	900
okan	OK2	1.83	26.8	17600	940
Karri	KA1	1.85	57.1	18100	630
Nargusta	NA1	1.84	52.0	18700	600
piquia	PI1	1.84	44.2	21400	690
vitex	VI1	1.84	47.6	15100	660
basralocus	BAS1	1.85	35.2	20300	660
bangkirai	BAN1	1.77	60.1	21200	820
Sucupira vermelho	SV1	1.81	53.7	18700	790
castanhorosa	CR1	1.81	73.4	18100	740
louro amarela	LA1	1.81	58.5	18000	610
louro faia	LF1	1.82	64.7	22900	830
purpleheart	PU1	1.83	57.0	19600	620
tauari vermelho	TV1	1.82	42.5	15900	660
favinha	FA1	1.81	49.1	19800	660
sapupira	FP1	1.82	64.4	20900	740
favinha prunelha	SA1	1.81	47.9	16600	610
bilinga	BIL2,BIL2	1.78	20.1	13400	680
evuess	EV2	1.81	76.1	22800	910
tali	TA1,TA2	1.77	42.5	17500	820

Strength modelling of structural timber

5.1 Failure mechanisms and failure criterions

To be able to define strength predicting models, it is essential to know the mechanisms that cause failures in timber. When these failure mechanisms are known, strength predicting equations can be formulated. In this thesis, timber is considered as a Bernoulli-Euler beam: plane sections remain plane and normal to the deflected neutral axis. For the relationship between stresses and strains Hooke's Law is considered in the elastic range. For tension, only an elastic range and brittle failure is considered. For compression, a bi-linear stress-strain relation is assumed, whereby the elastic part follows Hooke's Law and for the plastic part a *MOE* of zero is assumed.

The bending strength calculated from a four point bending test according to figure 3.1 is given in equation 3.1 and repeated here as equation 5.1

$$f_m = \frac{3 Fa}{bh^2} = \frac{M}{W} \quad (5.1)$$

As can be seen from equation (5.1), the full section modulus of the timber is used, despite the existence of knots or grain angle deviation. In reality, the stress distribution is much more complicated. A closer look at the real stress distribution of timber in clear wood and in structural timber can help to formulate strength prediction models on a physical basis. The objective is that strength calculated according to equation 5.1. has to be predicted, irrespective of the stress distribution at failure.

In this section, the failure modes for clear wood, timber with grain angle deviations and timber with knots will be studied. Based on investigation of the failure modes, prediction models can be formulated. Other failure modes due to compression failures, fissures etc. should be avoided for structural timber by removing the material that contains these characteristics above a certain level by visual inspection.

5.1.1 Clear wood

As was explained in chapter 2, there is a great difference in strength properties of (small) clear wood and of timber of structural size. Clear wood is defined as wooden elements for which the grain angle is parallel to the longitudinal axis, and where disturbing features as knots, compression failures, etc. are not present. Clear wood is usually tested in small

sizes (cross section 20 mm x 20 mm, span 360 mm in a four-point bending test, or 50 mm x 50 mm, span 700 mm in a three-point bending test) because it is difficult to obtain it in larger sizes.

Clear wood is the most homogeneous form of timber that is dealt with in this thesis. This is the macroscopic level where no strength reducing characteristics are present that cause failure on the gross level. In this thesis, the scatter in bending strength (and also in *MOE*) that is expected for similar pieces of clear wood is regarded as the natural variability of timber that cannot be explained by measurements on the wood (either visual or with a machine). In chapter 2 it was defined that similar pieces of clear wood are pieces with the same density, regardless of the species. This will be further elaborated in section 5.2.

Test results have shown that for clear wood the tension strength is higher than the compression strength. This can be explained from the fact that compression failure is a stability problem of the cells the fibers are made up, whereby the pure compression strength of the cell walls cannot be reached.

This is illustrated by the following tests.

Ten pieces of clear wood of wood species greenheart with a cross section of 50 x 50 mm² were cut into specimens that were tested under tension, compression, bending and shear. The test set-ups of ASTM D143-09 were followed:

- For bending tests, specimens with a cross section of 50 x 50 mm² were tested in a three-point bending test with a span of 710 mm
- For the compression tests, specimens with a cross section of 50 x 50 mm² and a length of 200 mm were tested
- For the tension test, specimens were tested with a cross section of 35 x 35 mm² at the grips and a length of 600 mm. The specimens were machined to a ‘dogbone’ shape, in such a way that in the middle of the piece over a length of 63 mm a cross section of 4.8 by 9.5 mm² remained to ensure that failure occurs in this section.
- For shear tests, a cross section of 50 x 50 mm² was used with a length of partly 50 mm and partly 65 mm. The part of 50 mm length is sheared off parallel with the fiber direction.

In table 5.1 the test results are given. The bending strength can be calculated with equation (5.1), with the bending moment at midspan, where a is half of the span.

Table 5.1. Test results on clear wood of greenheart

Property	Mean value	Coefficient of Variation CoV(%)
Compression strength $f_{c,0}$ (N/mm ²)	75.8	10.4
Tension strength $f_{t,0}$ (N/mm ²)	219.0	23.8
Bending strength $f_{m,0}$ (N/mm ²)	164.0	11.3
Shear strength $f_{v,0}$ (N/mm ²)	7.5	15.8

Table 5.1 shows the difference in compression and tension strength. In figures 5.1 to 5.3 the failure patterns and the associated load-displacements diagrams are displayed for a compression, tension and bending test. The load-displacement diagram of the compression test in figure 5.1 shows a bi-linear relation. The associated plastic failure pattern is shown in the photograph. Figure 5.2 shows brittle behaviour in the failure pattern and in the load-displacement diagram for the tension tests.

Figure 5.3 shows the failure mechanism for a clear wood specimen under a three point bending test. There is a linear behaviour until the maximum compression stress is reached at the compression side of the specimen. After that, the stress distribution will change. The overall stiffness reduces and an elasto-plastic behaviour can be seen. When the maximum tension strength is reached at the tension side, the piece fails. The ultimate failure cause is tension failure, which is brittle.

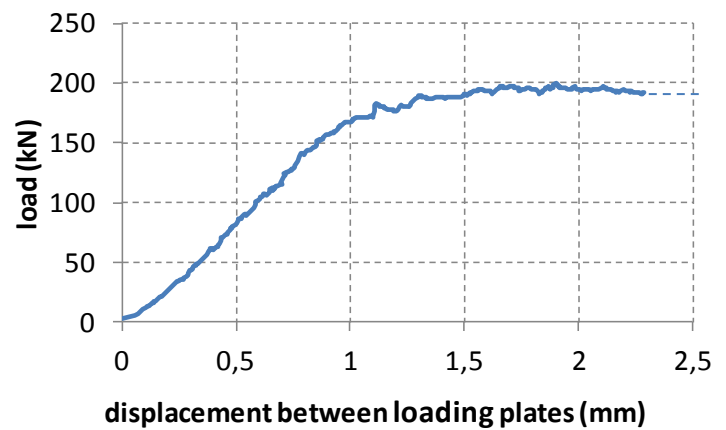


Figure 5.1. Failure pattern for a clear wood specimen of greenheart and associated load-displacement diagram under a compression test

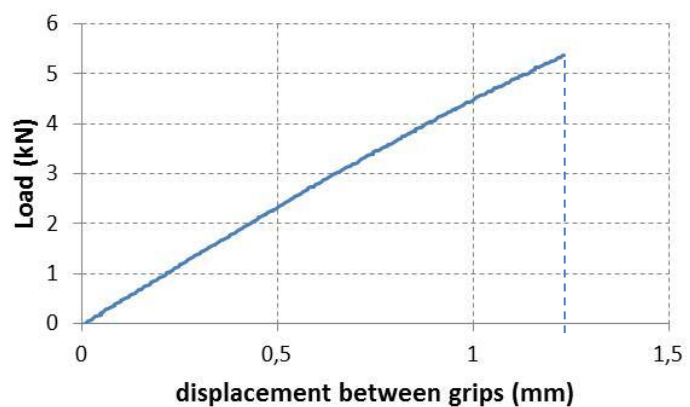


Figure 5.2. Failure pattern for a clear wood specimen of greenheart and associated load-displacement diagram under a tension test.

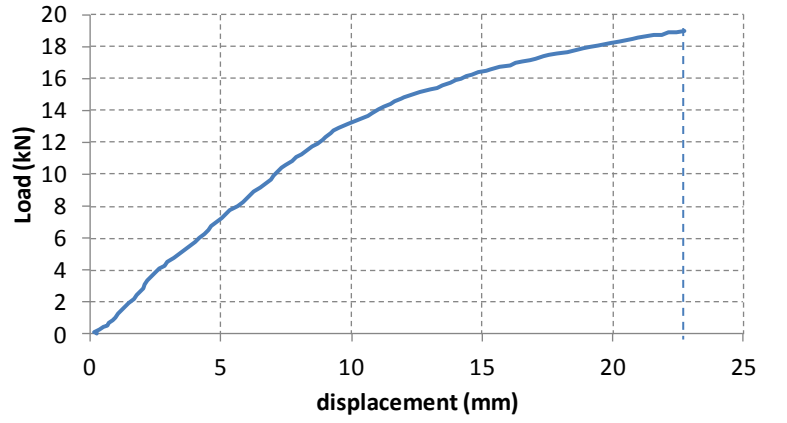


Figure 5.3. Failure pattern for a clear wood specimen of greenheart and associated load-displacement diagram under a three point bending test.

In figure 5.4, the stress distribution at failure for the specimen in figure 5.3 is shown.

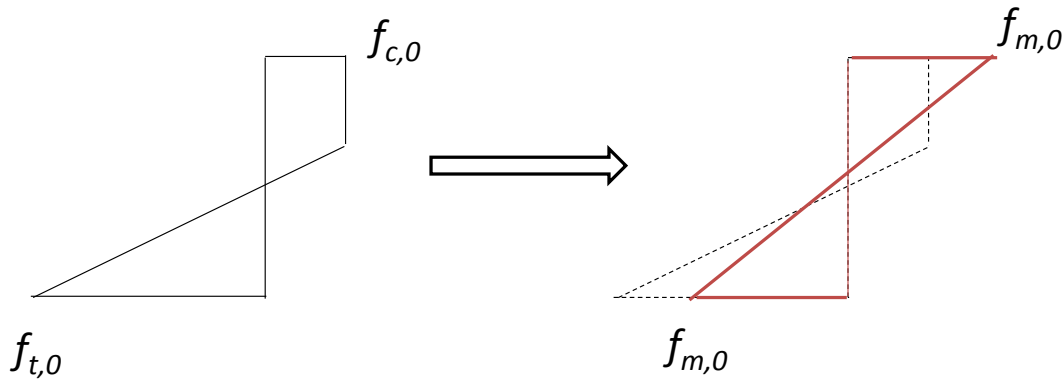


Figure 5.4. Stress distribution at failure for the piece shown in figure 5.3. On the left the real stress distribution and on the right the calculated bending strength according to equation (5.1).

The stress distribution at failure can be schematized according to the left picture in figure 5.4 with plastic behaviour on the compression side and brittle behaviour on the tension side. Based on this figure, the outcome of the bending strength $f_{m,0}$, assuming a linear stress distribution, as in the right picture of figure 5.4 can be calculated from the equilibrium of forces in the cross section with equation (5.2)

$$f_{m,0,cw} = \frac{f_{c,0} \cdot (3f_{t,0} - f_{c,0})}{f_{t,0} + f_{c,0}} \quad (5.2)$$

When the average tension and compression strengths according to table 5.1 are used as input in equation (5.2), an equivalent bending strength of 149,4 N/mm² is found. The difference with the mean bending strength value of 164 N/mm², calculated from tests according to equation 5.1 can be explained by size effects, especially on the tension side.

Starting with the failure mechanisms of clear wood, the failure mechanisms of structural timber will be explained. In section 5.2, models to predict the strength of timber will be developed based on non-destructive measurements. From these non-destructive measurements the density and the *MOE* (the MOE_{dyn}) will be used as strength predicting parameters. We can see that in equation (5.2) neither of these two parameters are present. This means that at macroscopic level (clear wood) the basic relationship between density-bending strength and density-*MOE* will be based on experimental results. These relationships will be the basis for further physical modelling of gross strength reducing characteristics.

5.1.2 Structural timber with grain angle deviations

The anisotropic behaviour of timber has a major influence on the strength of timber, also when the pieces are loaded axially. In most cases, the fiber direction (for tropical hardwoods in practical all cases) will not run parallel to the longitudinal axis of the beam. The result is that normal stresses will occur, not only parallel to the fiber direction, but also perpendicular to the fiber direction and shear stresses. This will have an effect on the failure strength, as well on the Modulus of Elasticity.

Hoffmeyer (1987) states that grain angle deviation is not the governing cause of failure for structural timber. However, this observation is based on failure patterns for softwoods, and it is certainly not the case for tropical hardwood timber. Structural beams from tropical timber are often delivered with almost no knots, and then the grain angle deviation will be the governing cause for failure. In figure 5.5, pictures are shown for cumaru beams with a height of 150 mm with failure due to grain angle deviations from the longitudinal axis. These tests were performed in a four point bending test according to figure 3.1.

The load-displacement diagrams of figure 5.5 show that at the gross level, elastic-plastic behavior and pure elastic behavior can occur. This will depend on the magnitude of the slope of grain.

Since the compression strength of clear wood is lower than the tension strength of clear wood, elastic-plastic behavior may be visible in the load displacement diagram. It depends on the grain angle deviation which behavior occurs.

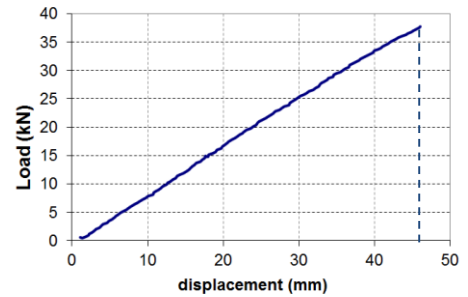
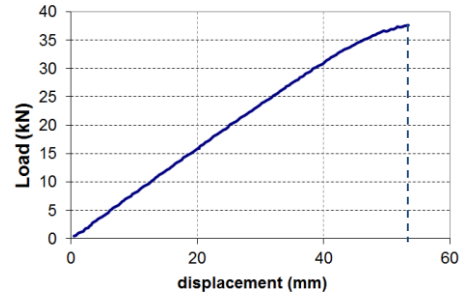
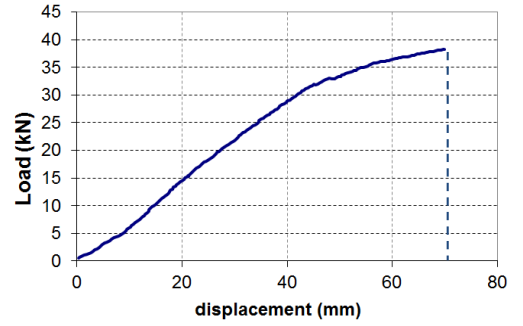


Figure 5.5. Failure patterns for specimens with different grain angle deviations of structural cumaru timber and associated load-displacement diagram from a four point bending test.

In section 2.2 it was explained that the strength under an angle with the grain can be described by the equations defined by Hankinson or Norris.

Using the test results for clear wood from table 5.1, the tension strength under an angle with the grain can be described by equations (2.4) to (2.6), assuming that failure is only induced by either tension strength parallel to the grain, the tension strength perpendicular to the grain or the shear strength. In this case, a mean value of $f_{t,90} = 3 \text{ N/mm}^2$ is assumed.

The tension strength under an angle with the grain described by equations (2.4) to (2.6), the Hankinson equation described by equation (2.9) and the Norris equation (with $\beta = 1$) described by equation (2.15) are plotted in figure 5.6.

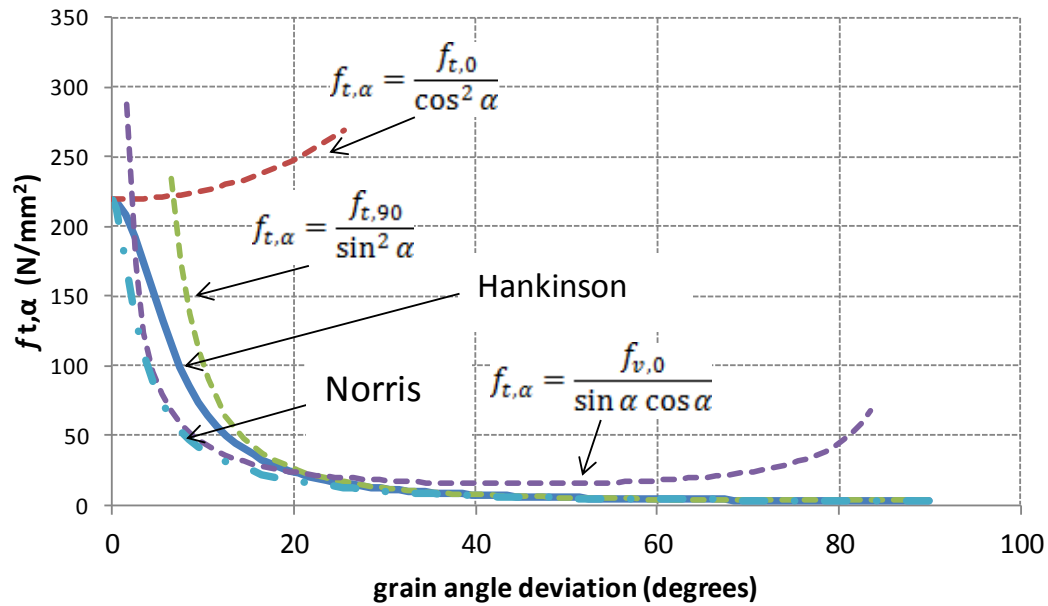


Figure 5.6. Tension strength of clear wood of greenheart plotted against the grain angle deviation, based on different failure prediction equations.

From figure 5.6 it can be seen that the Norris equation seems to be more suitable than the Hankinson equation when the shear strength is governing in the interaction of stresses. Figure 5.6 suggests that this could be the case when the shear strength, determined by the ASTM test, is used in the interaction equation of Norris. However, it can be questioned whether this value is the correct shear value in this situation, for instance because size effects might occur. Therefore, when the shear strength is not governing in the interaction of stresses, the Hankinson equation can be used in the modeling.

To evaluate the suitability of the Hankinson equation for bending, a test program with specimens under different grain angles was set-up for softwood species spruce and tropical hardwood species massaranduba. There were no knots in the test pieces. The pieces had a cross section of 50 x 50 mm² and were tested in a four-point bending test according to EN 408 as shown in figure 3.1. Both the bending strength and the MOE_{local} were determined.

For structural timber the normal range for the grain angle deviation is between 0 and 12 degrees (at 12 degrees the slope of grain is 1:5). Therefore, test pieces were prepared for a four-point bending test with grain angle deviations of 0, 10 and 18 degrees with the beam axis in span direction. The given grain angle deviations are in the plane of the loading direction. In the plane perpendicular to the loading direction the grain angle deviation was practically zero for all pieces. For each grain angle deviation degree, 5 or 6 pieces were made for each wood species. See figure 5.7 for the preparation of specimens with a grain angle deviation of 18 degrees. The pieces with grain angles 10 and 18 degrees were elongated to be able to perform a four point bending test with the same span for all pieces.

All failures occurred in the parts with the investigated grain angle deviation. The MOE_{local} was also measured in this part.

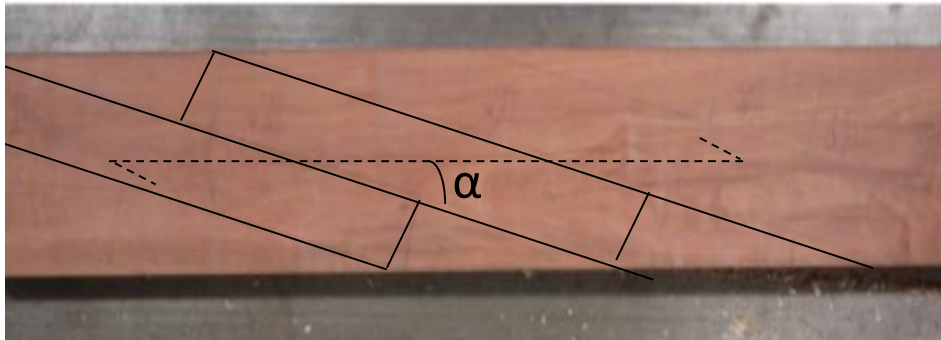


Figure 5.7. Cutting pattern for massaranduba pieces with grain angle deviation of 18 degrees.

Figure 5.8. shows the failure patterns and associated load-displacement diagrams for pieces of massaranduba with a grain angle deviation of 0, 10 and 18 degrees.

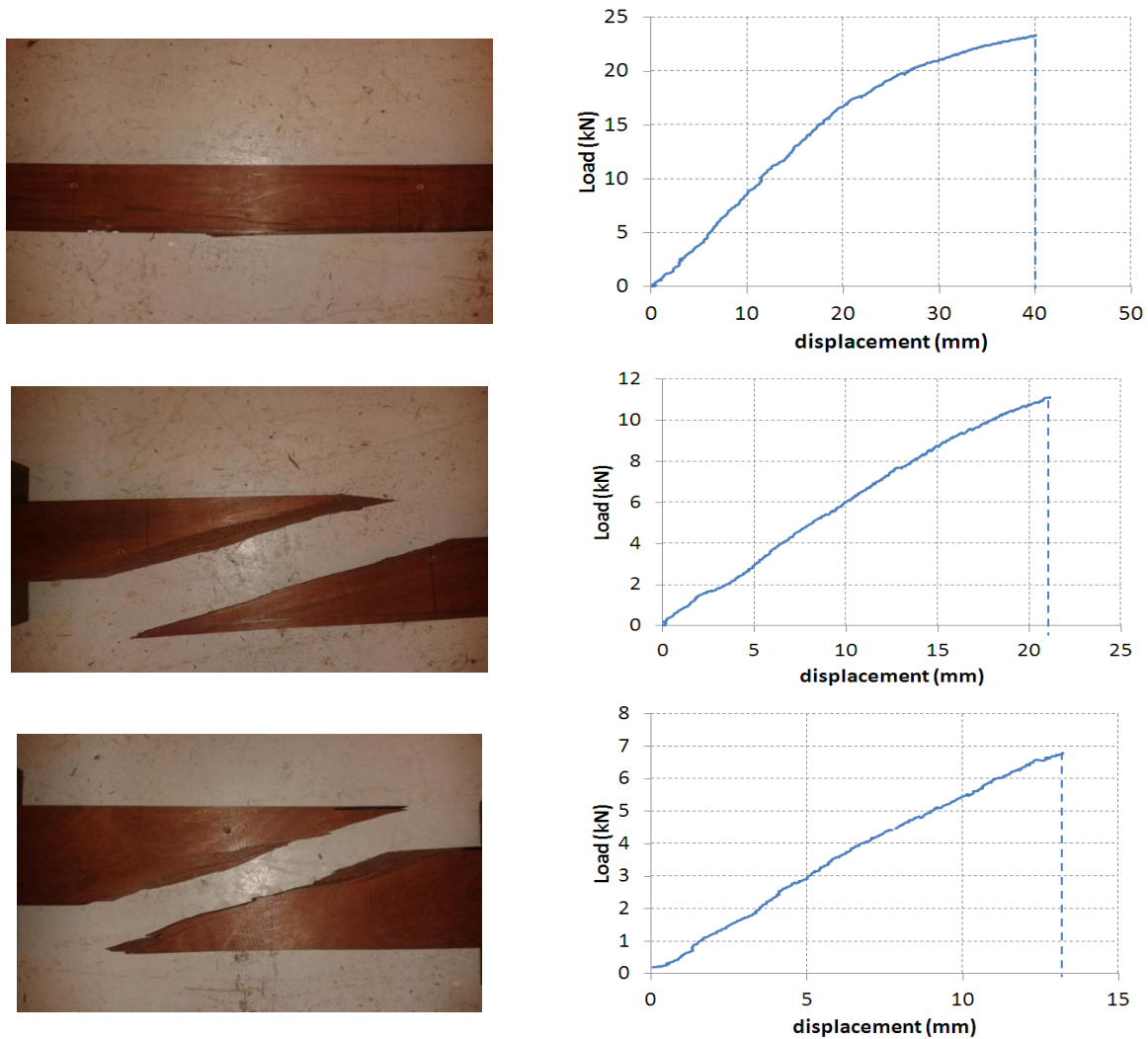


Figure 5.8. Failure patterns and associated load-displacement diagrams for pieces of massaranduba with a grain angle deviation of 0 (top), 10 (middle) and 18 (below) degrees.

Figure 5.8 shows that at 0 degrees grain angle deviation, elasto-plastic behavior occurs and at 10 and 18 degrees grain angle deviation elastic brittle behavior occurs. For the spruce specimen equivalent load-displacement diagrams were found at the same grain angle deviations.

The values of $f_{m,0}$ and k_f were fitted to the Hankinson equation (5.3) for bending by a non-linear regression analysis.

$$f_{m,\alpha} = \frac{f_{m,0}}{k_f \sin^2(\alpha) + \cos^2(\alpha)} \quad (5.3)$$

With

$$k_f = \frac{f_{m,0}}{f_{m,90}} \quad (5.4)$$

The exact grain angles were measured after the test and used in the analysis.

The results are given in table 5.2. $f_{m,90}$ is calculated from $f_{m,0}$ and k_f and therefore given in italics.

Table 5.2. Properties derived by a non-linear regression analysis fitted to equation (5.3)

property	spruce	massaranduba
$f_{m,0}$ (N/mm ²)	68.5	166.4
$f_{m,90}$ (N/mm ²)	2.0	5.0
k_f	33.4	33.1

In figures 5.9 and 5.10 the Hankinson equation with $f_{m,0}$ and $f_{m,90}$ from table 5.2 is shown together with the test data. The Hankinson equation is fitted over the range from 0 to 20 degrees. It can be seen that over this range a linear regression line could be constructed. This is the reason that for structural timber with the normal range of grain angle deviations, a linear regression fit between the grain angle deviation and the strength can be observed.

The values of $f_{m,0}$, $f_{m,90}$ and f_v , determined from a non-linear regression using the Norris equation (2.15) with $\beta = 1$, are given in table 5.3. Two analyses were performed. In the first analysis all three values were determined with a non-linear regression. In the second analysis $f_{m,0}$ and $f_{m,90}$ were taken from table 5.2. To investigate which shear strength would be required for these values to fit with the Norris equation, the values of $f_{m,0}$ and $f_{m,90}$ of table 5.2 are put into the Norris equation (2.15) with $\beta = 1$. Then only f_v was determined through a the non-linear regression. The results are given in table 5.3.

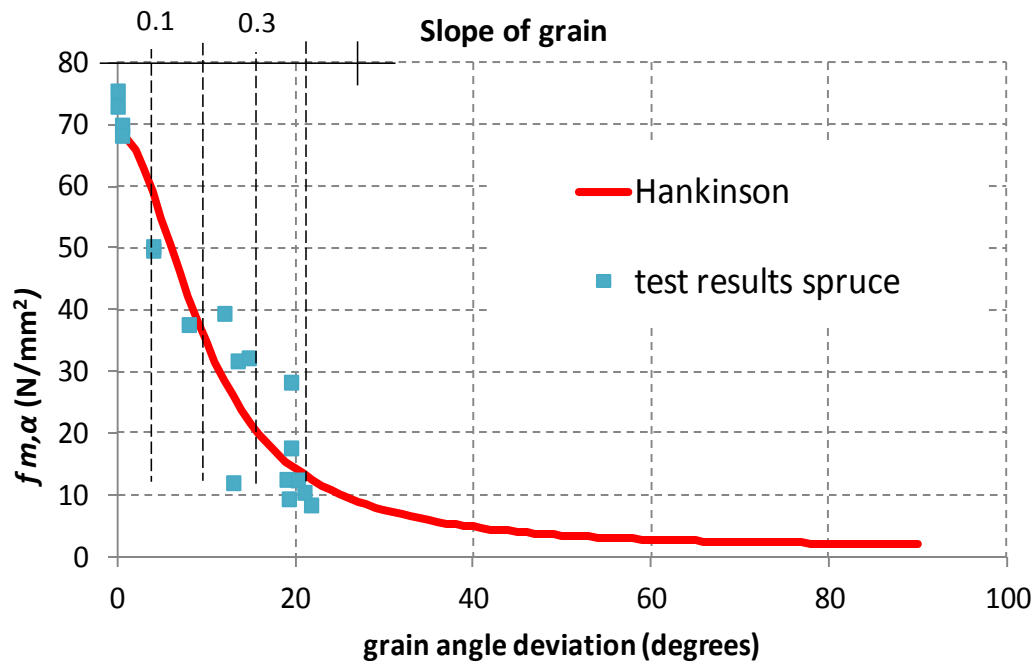


Figure 5.9. The Hankinson equation according to equation (5.3) with strength properties according to table 5.2 for spruce together with the test results for spruce.

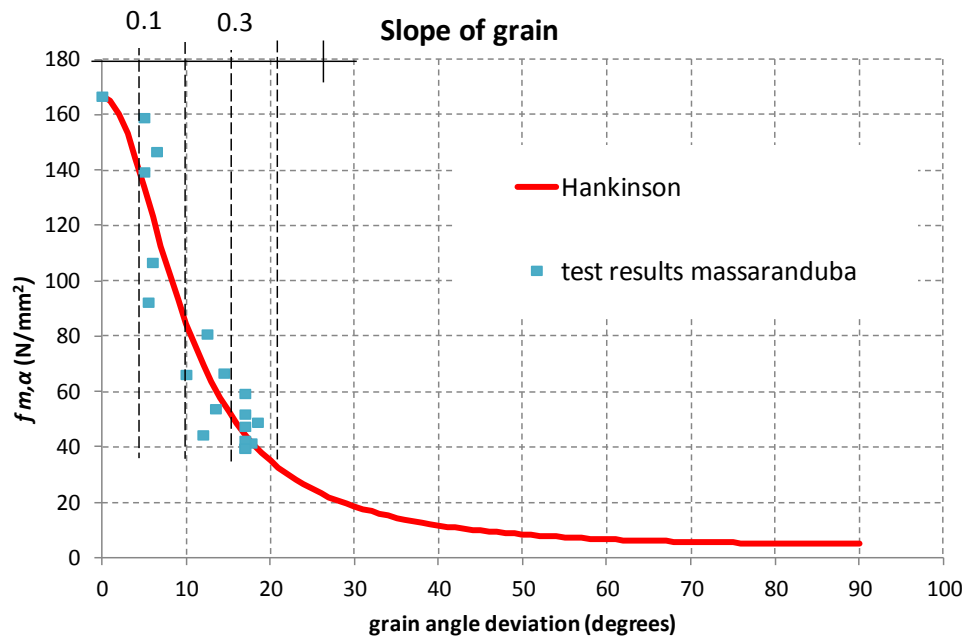


Figure 5.10. The Hankinson equation according to equation (5.3) with strength properties according to table 5.2 for massaranduba together with the test results for massaranduba.

Table 5.3. Properties derived by a non-linear regression analysis fitted to equation (2.15)

properties	spruce		massaranduba	
fitted properties	$f_{m,0}$, $f_{m,90}$ and f_v	f_v	$f_{m,0}$, $f_{m,90}$ and f_v	f_v
$f_{m,0}$ (N/mm ²)	70.8	68.5	171.4	166.4
$f_{m,90}$ (N/mm ²)	3.6	2.0	5.8	5.0
f_v (N/mm ²)	6.1	7.5	18.5	20.1
k_f	19.5	33.4	29.4	33.1

Equation (2.15) with inserted the results from table 5.3 will give a prediction line between bending strength and grain angle deviation that practically coincides with the Hankinson line according to equation (5.3). Table 5.3 shows that, when fitted to the Norris equation, the shear strength is much higher than retrieved from pure shear tests. The strength profile of massaranduba can be compared with that of greenheart (the species are in the same density range). Where in table 5.1 a shear strength of 7.5 N/mm² for greenheart is given, Hek (2014) found a mean shear strength value (with the test set-up of ASTM D143-09) of 11.0 N/mm² for massaranduba, based on 3 specimens. In the Norris interaction from the bending test data a shear strength of 18.5 N/mm² for massaranduba was found. This is probably due to size effects, and a cause could be that for pure shear the strength values for timber are much lower than for shear in interaction with tension parallel and perpendicular to the grain in bending. For the shear in interaction with tension parallel and perpendicular to the grain the maximum stresses only occur in the outer fibers, wherein a pure shear test the entire cross section undergoes the same shear stress.

That the shear strength values found are much higher than the values in EN 338 is in agreement with test results in a 5-point bending test performed by Van de Kuilen en Leijten (2002). They found much higher shear values than those listed in EN 338. The shear strength values for tropical hardwoods currently listed in EN 338 are however not based on shear tests according to EN 408, but based on expert judgment. The difference between pure shear test values and values found from fitting on the basic Norris equation can be taken into account by applying the factor β in equation (2.15).

Figures 5.9 and 5.10 show that in the range of interest of structural timber the influence of grain angle deviation can be well described by the Hankinson equation for both softwood and hardwood. This is because when the shear strength is above a certain level, the influence on the shape of the Norris equation is very low. When the shear strength is very low, it cannot be neglected and the Norris equation has to be used. Based on the regression results listed in tables 5.2 and 5.3 it is concluded that for structural timber the shear strength in interaction with stresses parallel and perpendicular to the grain is above the level that would require the application of the Norris equation. Therefore, in this thesis the Hankinson equation will be used for the modeling of the influence of grain angle

deviation on the bending strength, with only the bending strength parallel and perpendicular to the grain as input properties.

At zero degrees grain angle deviation elasto-plastic behavior will occur at failure and above approximately 12 degrees grain angle deviation (Van de Have, 2013) the failure is caused exclusively by the tension strength reduction and pure elastic brittle behavior occurs. Between zero and 12 degrees grain angle deviation some elasto-plastic behavior will occur. Theoretically, a slight deviation of the Hankinson curve according to equation (5.3) could be expected below 12 degrees grain angle deviation. This will be in the order of 1 or 2 percent and disappears in the scatter in test results. Therefore this will not be taken into account.

The Wood Handbook (Ross et al., 2010) gives different power values for tension, compression and bending. Based on the theoretical considerations (equations 2.4, 2.5 and 2.6) one would expect a power value of 2. In the paper introducing the Hankinson formula, a factor 2 was found for the compression strength. In the regression results of table 5.2 a power value of 2 fitted well and this will be used in this thesis.

The influence of grain angle deviation on the MOE can be calculated by applying geometrical transformations on the compliance tensor S for orthotropic materials.

The result is given in for instance Leknitskij (1981):

$$\frac{1}{MOE_{\alpha}} = \frac{\cos^4 \alpha}{MOE_0} + \left(\frac{1}{G_{12}} - \left(\frac{2\nu_{12}}{MOE_0} \right) \right) \sin^2 \alpha \cos^2 \alpha + \frac{\sin^4 \alpha}{MOE_{90}} \quad (5.5)$$

Again, the Modulus of Elasticity could also be approximated with the Hankinson formula. See equations (5.6) and (5.7)

$$MOE_{\alpha} = \frac{MOE_0}{k_m \sin^2(\alpha) + \cos^2(\alpha)} \quad (5.6)$$

With

$$k_m = \frac{MOE_0}{MOE_{90}} \quad (5.7)$$

For equation (5.6), the shear modulus and the Poisson coefficient do not have to be known. Equation (5.5) coincides with equation (5.6) when G_{12} is estimated by equation (5.8), based on MOE_0 , MOE_{90} and ν_{12} :

$$\frac{1}{G_{12}} = \frac{1}{MOE_0} + \frac{1}{MOE_{90}} + \frac{2\nu_{12}}{MOE_0} \quad (5.8)$$

It is interesting to see the difference with EN 338 in which a ratio for G_{12} of 1/16 with MOE_0 is given for both softwoods and hardwoods for all strength classes. The shear modulus G_{12} is not an easy property to determine and is under constant discussion (Ravenshorst and De Vries, 2014).

With equation (5.8) the shear modulus G_{12} depends on the ratio between MOE_0 and MOE_{90} . EN 338 gives a ratio of 30 between MOE_0 and MOE_{90} for softwoods and 15 for hardwoods. With a value of $\nu_{12} = 0.3$ this gives a ratio between G_{12} and MOE_0 of 1/32 for softwoods 1/17 for hardwoods.

Leknitskij (1981) emphasizes that the property of the shear Modulus from the theory of anisotropic materials is independent of the Moduli of Elasticity parallel and perpendicular to the grain. He states that relations between these properties (he gives examples for crystalline materials) are purely experimental.

By using the Hankinson relation, an experimental relation can be found with relations for the shear modulus implicitly included. However, as with the equations for strength, this approach might give better results when fitted on experimental data than by assuming values for the shear modulus and using equation (5.5).

Fitting the test data for the MOE_{local} by a non-linear regression to equation (5.6) gives values for the MOE as listed in table 5.4.

Table 5.4. Properties derived by a non-linear regression analysis fitted to equation (5.6)

property	spruce	massaranduba
MOE_0 (N/mm ²)	13700	25000
MOE_{90} (N/mm ²)	690	2200
k_m	19.8	12.4

Table 5.4 shows that the ratio between MOE_0 and MOE_{90} is greater for the softwood species than the hardwood species, but different from the ratio of EN 338 (EN 338 gives a ratio of 30 for softwoods and 15 for hardwoods).

In figures 5.11 and 5.12, the Hankinson equation (5.6) with MOE_0 and MOE_{90} according to table 5.4 is shown, together with the test data.

Figures 5.11 and 5.12 indicate that the Hankinson formula is also suited to predict the MOE under an angle with the grain based on MOE_0 and MOE_{90} . Therefore, in this thesis, the Hankinson equation will also be used for the prediction of the MOE under an angle with the grain.

Based on tables 5.3 and 5.4, for every grain angle value the bending strength and MOE can be calculated with the Hankinson equations. Using the values from tables 5.3 and 5.4, the bending strength values calculated with equation (5.3) are plotted against the MOE -values calculated with equation (5.6) in figure 5.13. For both spruce and massaranduba also the test results are plotted.

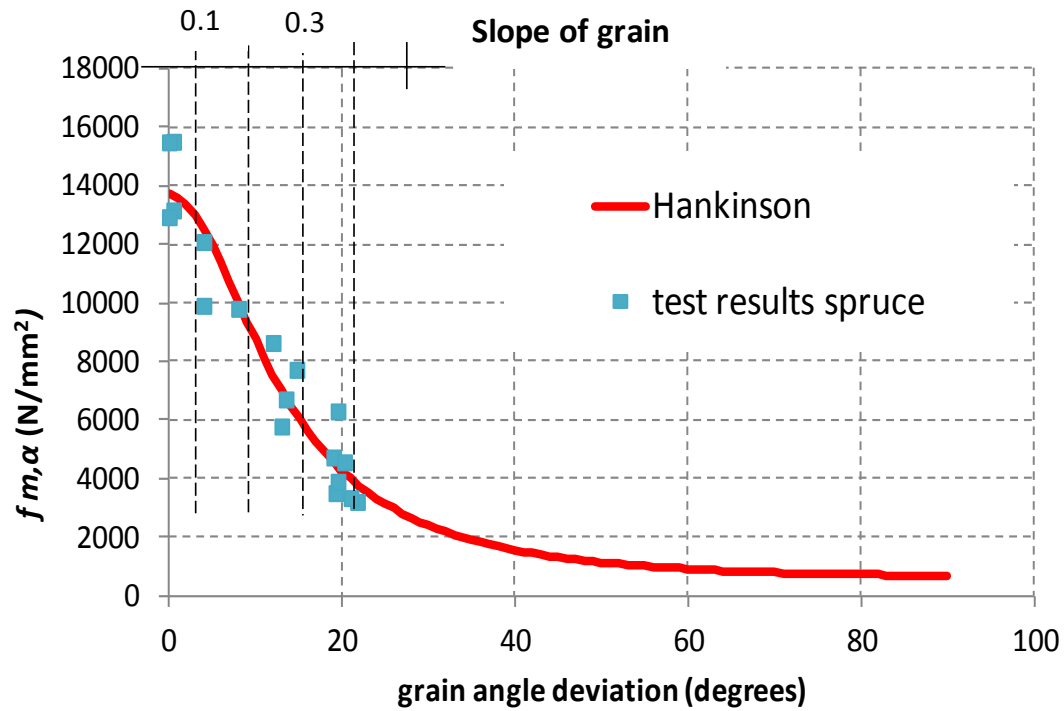


Figure 5.11. The Hankinson equation according to equation (5.6) with stiffness properties according to table 5.4 for spruce together with the test results for spruce.

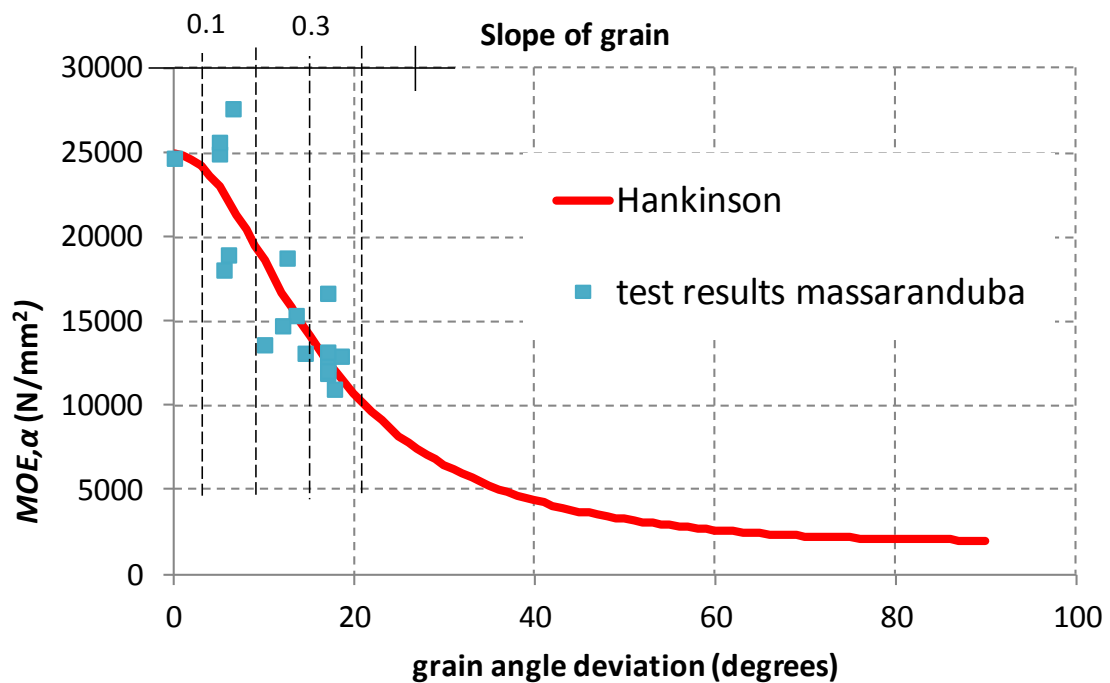


Figure 5.12. The Hankinson equation according to equation (5.6) with stiffness properties according to table 5.4 for spruce together with the test results for massaranduba.

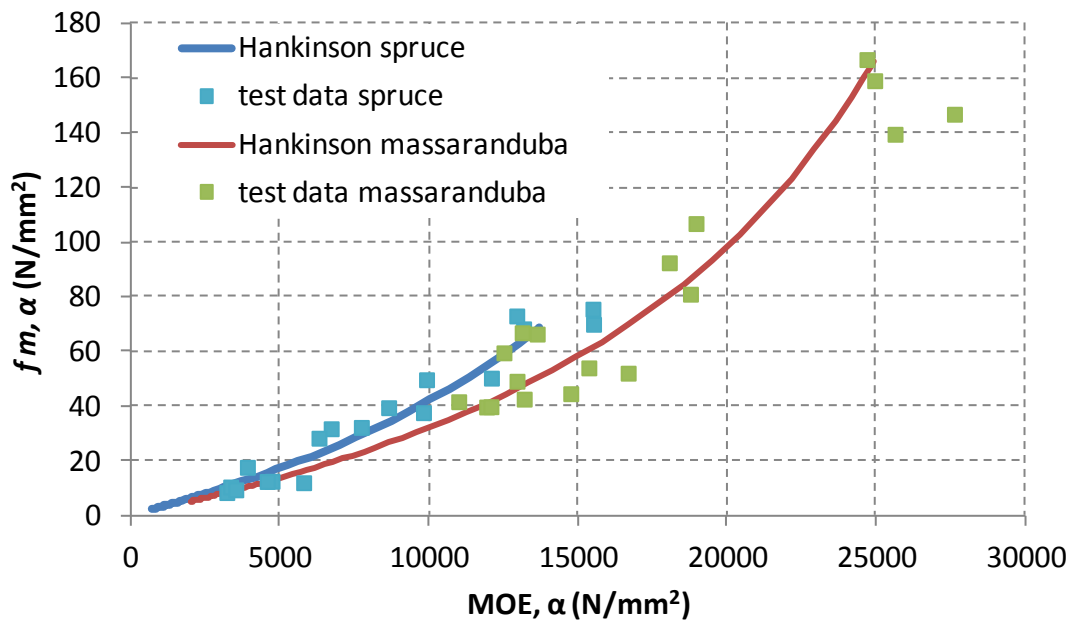


Figure 5.13. *MOE and bending strength values from test results of spruce and massaranduba plotted together with the calculated values with equations (5.3) and (5.6) based on property values of tables 5.2. and 5.4.*

Figure 5.13 shows that the relationship between the *MOE* and the bending strength due to the occurrence of grain angle deviation is non-linear. The test data is positioned around the Hankinsons relationships for both species which confirms the applicability of these relationships. Figure 5.13 also shows that for the test data (with grain angle deviations between 0 and 20 degrees) a linear regression line might give good results, although the “real” behaviour is non-linear. The difference between the data of the two species is caused by the starting values for grain angle 0 and the ratios between the properties parallel with the grain and perpendicular to the grain. A linear regression between strength and *MOE* independent of the species might give some trends that have no direct physical background. In section 5.2, the Hankinson relations will be adapted in such a way that a prediction model with a physical background can be constructed.

This section shows that the reduction in bending strength and bending stiffness caused by grain angle deviation can be described by applying the Hankinson equations. This was verified on test pieces where the exact grain angle was known. In structural timber, measurement of the grain angle deviation is much more difficult, as was shown in chapter 4, and then only the grain angle deviation at the surface can be measured. However, because grain angle deviation has an influence on both strength and stiffness, the assumption in this thesis is that when the *MOE* is measured, the overall grain angle deviation will affect the *MOE* in such a way that it can be used to predict the bending strength.

5.1.3 Structural timber with knots

Besides grain angle deviation, knots are the most important strength reducing characteristic. For softwoods, knots are the main characteristic causing failure (Hoffmeyer (1987)). The presence of knots has a global and local effect on the stress distribution of a timber beam in bending.

The global effect is caused by the fact that the knot can be regarded as a hole in the cross section. The axial stresses around the hole will become larger as a result of the reduced section modulus of the cross section. The local effect is caused by the fact that around the knot locally large grain deviations can occur. In these regions, the forces have to be transferred from the full cross section to the reduced cross section. Around the knot, stress concentrations with tension stresses perpendicular to the grain and shear stresses may occur.

Due to the presence of knots, the tension zone is normally weaker than the compression zones. Therefore, brittle behaviour in the tension zone before plastic behaviour in the compression zone has started is the most common failure due to the knots. This is illustrated in figure 5.14a for a spruce beam (softwood) and in figure 5.14b for a greenheart beam (tropical hardwood). Both were tested in a four-point bending test according to figure 3.1.

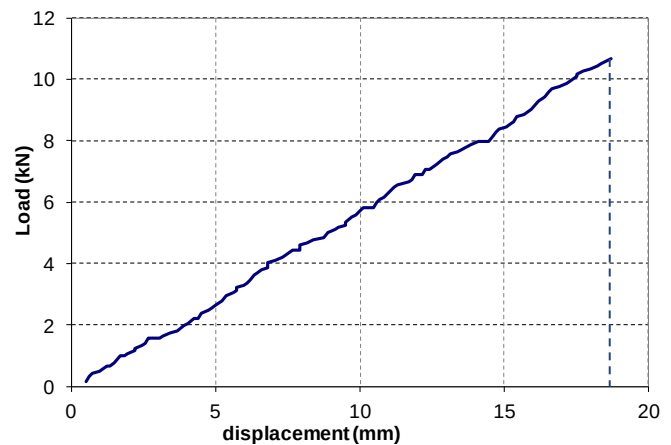


Figure 5.14a. Failure pattern for a spruce beam with knots and associated load-displacement diagram under a bending test.



Figure 5.14b. Failure pattern for a greenheart beam with a knot and associated load-displacement diagram under a bending test.

Figure 5.15 illustrates the stress concentrations for timber with a knot in the compression zone and a knot in the tension zone. These figures are based on finite element calculations as reported by Bano et al. (2010). These figures are meant as illustrations and show the effect of a knot in the 2D-plane. Just like the influence of grain angle deviations is a 3D - effect, also the influence of knots is a 3D-effect. This is because knots will change in size within the cross section of the piece, depending on the position of the pith in our outside the cross section of the beam.

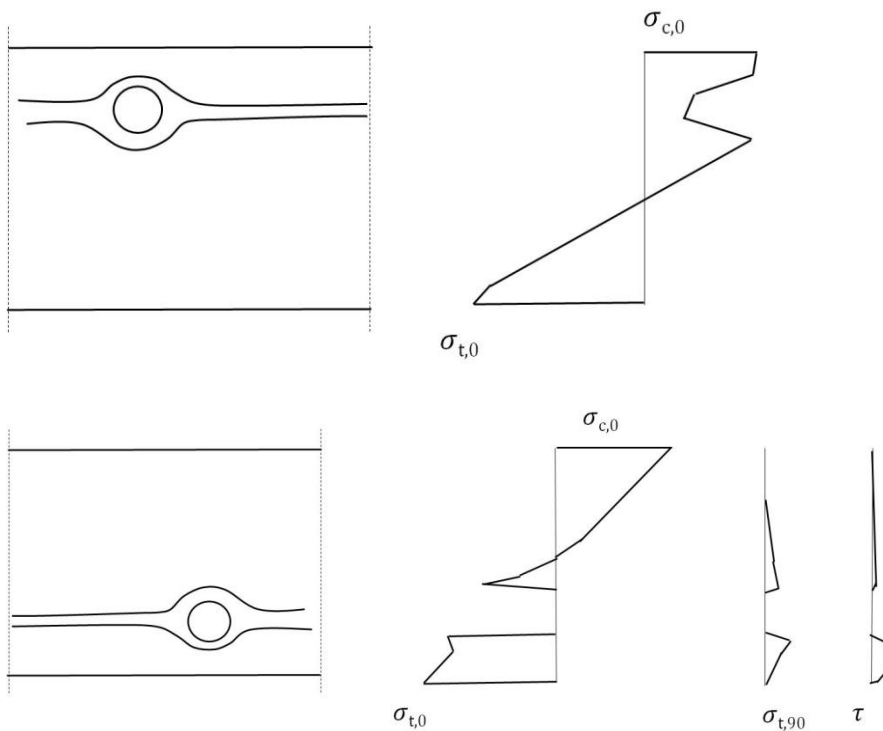


Figure 5.15. Stress distribution due to a knot in the compression zone (top) and in the tension zone (bottom). Resketched after Bano et al. (2010).

Figure 5.15 illustrates that the stress distribution around knots can be very complicated. However, the bending strength of a beam with knots under a bending test is also in this case calculated according to equation (5.1), assuming a linear stress distribution. To investigate the effect of the knot ratio KR on the bending strength and MOE, only the global effect whereby knots are seen as holes is adopted in this thesis.

In figure 5.16, for the case that a knot is positioned in the tensions side of the beam of the concept of knots as holes is illustrated.

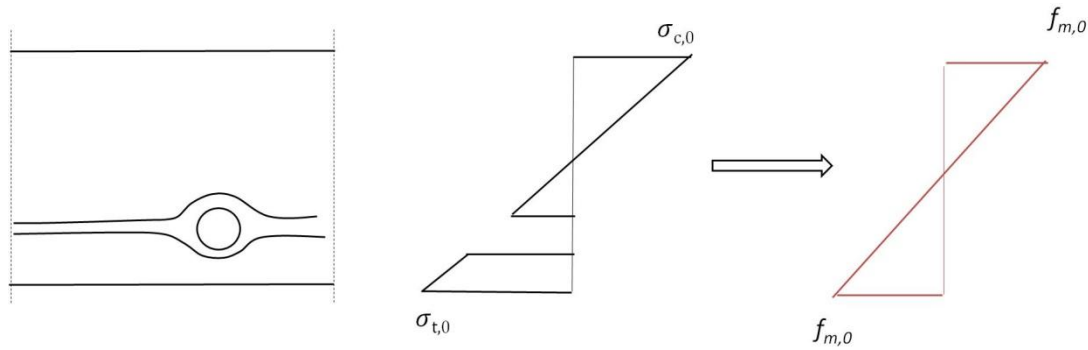


Figure 5.16. Global stress distribution due to a knot in the tension zone and the bending strength calculated with equation (5.1).

The effect of the global stress distribution due to the presence of knots on the bending strength calculated according to equation (5.1) and the MOE_{local} according to equation (3.2) will be investigated.

For this purpose, the weak zone model concept which was introduced by Riberholt et al. (1979), is used. See figure 5.17. The timber beam is regarded as clear wood connected by weak zones. The weak zones have an effect on both the strength and stiffness.

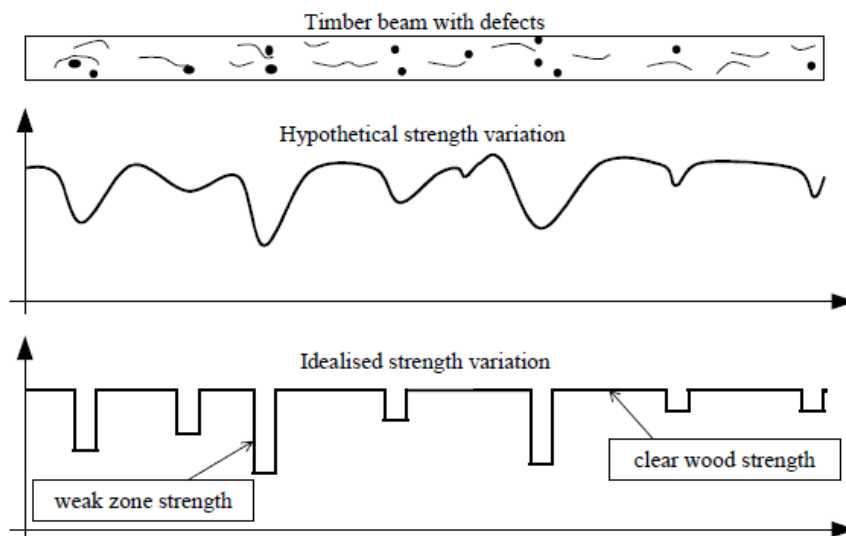


Figure 5.17. Weak zone model from Riberholt et al.(1979). Timber with knots is regarded as weak zones connected with clear wood.

For visual grading, various quantifications of knot measurements can be used, as explained in chapter 3. The simplest one, the single knot parameter, will be used here, and in this case, only the dimension of the knot d perpendicular to the beam axis divided by the depth of the beam h is considered according to equation (5.9) (As mentioned before, this is a simplification, ignoring the 3D-effect):

$$KR = \frac{d}{h} \quad (5.9)$$

The knot with the highest KR -value is assumed to be positioned in the center of the test span over the length of the beam, as this is the most critical position of the beam. In figure 5.18, two positions of the knot over the beam depth are shown. The knot size is the same and therefore also KR for both positions. The first position is with the knot at the edge of the beam in the tension zone, which has the maximum effect on strength and stiffness. The second position is with the knot in the middle of the beam, which has a minimum effect on strength and stiffness. It is assumed that the knot causes a hole with the same height d over the thickness of the beam.

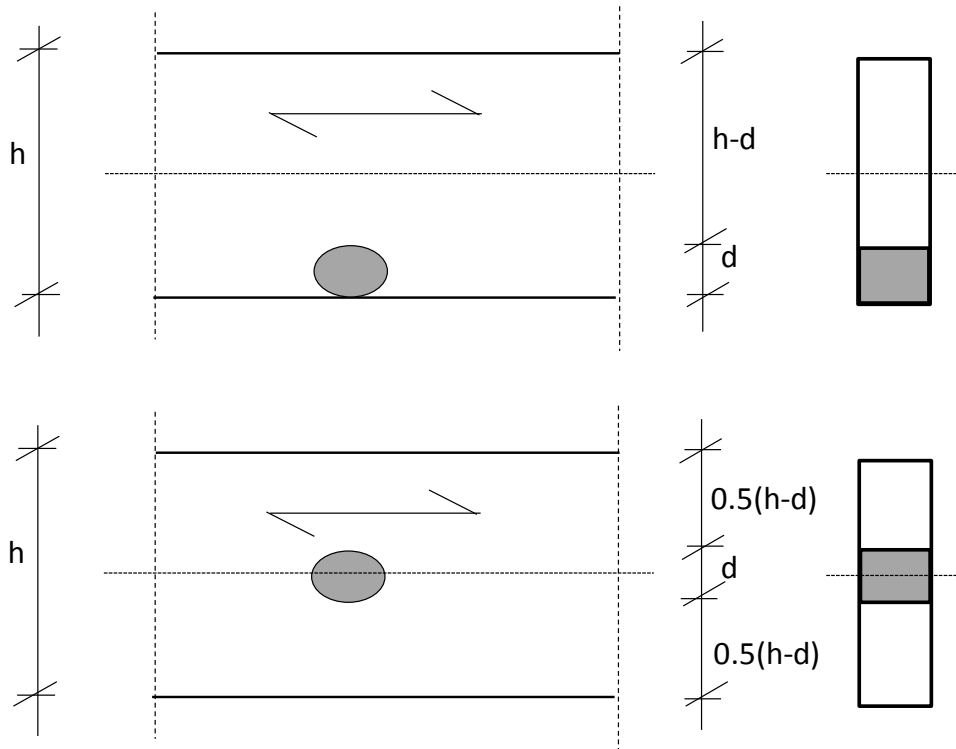


Figure 5.18. Two positions for a knot with the same size. The position at the top has the maximum influence on the strength and the position at the bottom the minimum influence for the KR -value.

The effect on the stiffness over the length of the beam is shown in figure 5.19.

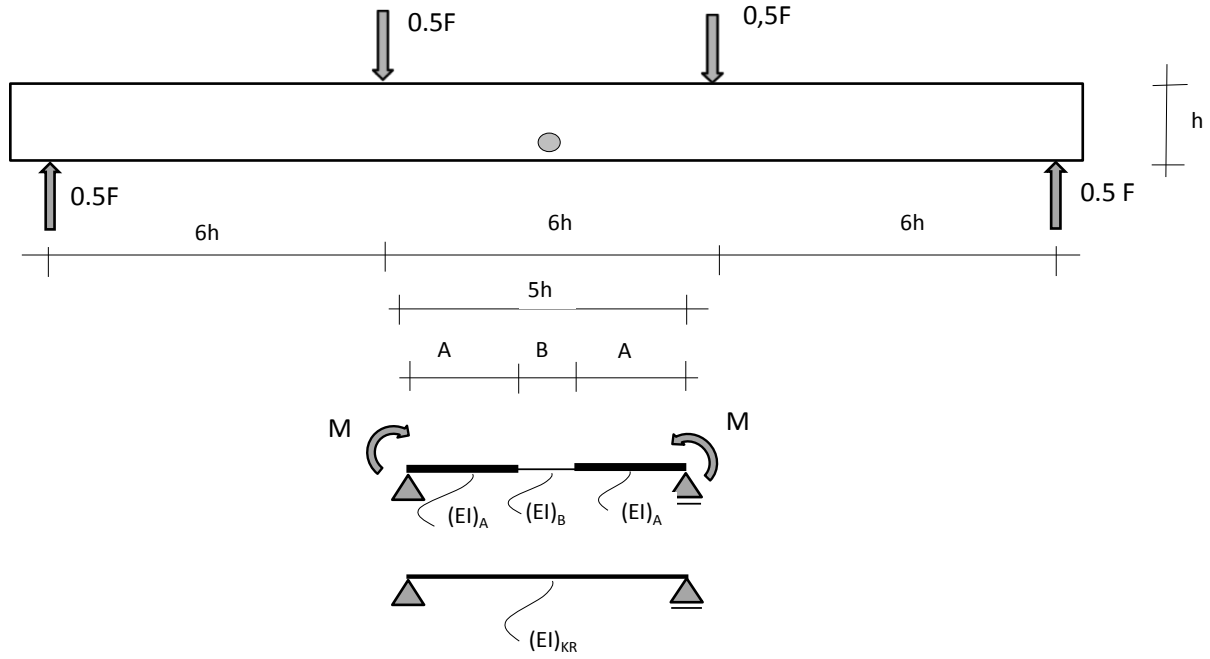


Figure 5.19. Effect of the stiffness of the part of the beam with the knot $(EI)_B$ and the clear wood part $(EI)_A$ on the measured $MOE_{local} (EI)_{KR}$.

It is assumed that the knot is placed exactly in the center of the span. The effect on the bending strength is directly related to the weak zone. Outside the weak zone, the timber is assumed to have the clear wood strength and stiffness. The ratios of the second moment of inertia I and the section modulus W can be calculated for the knot positioned with maximum and minimum influence.

$$I_{max,red} = \frac{1}{12} b(h-d)^3 \quad (5.10)$$

$$I_{min,red} = \frac{1}{12} b(h^3 - d^3) \quad (5.11)$$

$$W_{max,red} = \frac{\frac{1}{12} b(h-d)^3}{\frac{1}{2}(h-d)} = \frac{1}{6} b(h-d)^2 \quad (5.12)$$

$$W_{min,red} = \frac{\frac{1}{12} b(h^3 - d^3)}{\frac{1}{2}h} = \frac{1}{6} b \left(h^2 - \frac{d^3}{h} \right) \quad (5.13)$$

The unreduced section modulus is:

$$W_{full} = \frac{bh^2}{6} \quad (5.14)$$

The bending strength according to equation (5.1) can then be formulated as:

$$f_{m,red} = f_{m,cw} \frac{W_{red}}{W_{full}} \quad (5.15)$$

Where $f_{m,cw}$ is the clear wood strength. The reduction of the strength of the reduced cross section compared with the clear wood strength can be defined as:

$$red_{fm} = \frac{f_{m,red}}{f_{m,cw}} = \frac{W_{red}}{W_{full}} \quad (5.16)$$

These ratios can be calculated for the maximum and minimum influence of the knots:

$$red_{fm,max} = \frac{\frac{1}{6}b(h-d)^2}{\frac{bh^2}{6}} = \frac{(h-d)^2}{h^2} \quad (5.17)$$

$$red_{fm,min} = \frac{\frac{1}{6}b\left(h^2 - \frac{d^3}{h}\right)}{\frac{bh^2}{6}} = \frac{\left(h^2 - \frac{d^3}{h}\right)}{h^2} = 1 - \frac{d^3}{h^3} \quad (5.18)$$

The reduction ratio for the second moment of inertia I becomes:

$$I_{full} = \frac{bh^3}{12} \quad (5.19)$$

$$rd = \frac{I_{red}}{I_{full}} \quad (5.20)$$

$$rd_{max} = \frac{\frac{1}{12}b(h-d)^3}{\frac{bh^3}{12}} = \frac{(h-d)^3}{h^3} \quad (5.21)$$

$$rd_{min} = \frac{\frac{1}{12}b(h^3 - d^3)}{\frac{bh^3}{12}} = \frac{(h^3 - d^3)}{h^3} = 1 - \frac{d^3}{h^3} \quad (5.22)$$

The effect of the knot on the MOE_{local} has to be determined. The MOE_{local} is calculated from the deflection of the beam over a length $5h$ between the load heads where a constant moment is present. The constant moment is present over $6h$ but the deflection to calculate the MOE_{local} is measured over $5h$. The influence length of the knot on the zone with reduced bending stiffness is called B in figure 5.19. This might be a longer distance than the knot itself because the stresses need a certain length to be redistributed from the reduced to the clear section. The clear wood sections have a length A . The ratio between the bending stiffness of sections A and B is also rd .

$$\frac{EI_A}{EI_B} = \frac{I_A}{I_B} = rd \quad (5.23)$$

$$EI_B = \frac{EI_A}{rd} \quad (5.24)$$

The deflection in the center of the beam over the length of $5h$ is:

$$\delta = \frac{M(\frac{1}{2}B)^2}{2EI_B} + \frac{M\frac{1}{2}BA}{EI_B} + \frac{MA^2}{2EI_A} \quad (5.25)$$

When for the length of section B a length of $0.5A$ is assumed and EI_B is expressed as equation (5.24) then equation (5.25) becomes:

$$\delta = \left(\frac{9+16rd}{32rd}\right) \frac{MA^2}{EI_A} \quad (5.26)$$

The deflection in the center of the beam over the length of $5h$ can also be expressed with the equivalent bending stiffness EI_{kr} which is found in a bending test:

$$\delta = \frac{1}{8} \frac{M(B+2A)^2}{EI_{KR}} = \frac{25}{32} \frac{MA^2}{EI_{KR}} \quad (5.27)$$

Equalizing the deflections according to (5.26) and (5.27) gives:

$$EI_{KR} = \frac{25rd}{9+16rd} EI_A \quad (5.28)$$

In the calculation of EI_{KR} , the full second moment of inertia is used and can therefore be written as $E_{KR}I$. Part A of the beam consists of clear wood and therefore I_A is also the full second moment of inertia. Therefore, equation (5.28) can be written as:

$$E_{KR} = \frac{25rd}{9+16rd} E_A \quad (5.29)$$

The part A consists of clear wood and therefore E_A is the *MOE* for clear wood. The reduction ratio of the measured *MOE* compared with the *MOE* for clear wood then becomes:

$$red, ratio_{MOE} = \frac{E_{KR}}{E_A} = \frac{25rd}{9+16rd} \quad (5.30)$$

The minimum and maximum reduction ratio for the same *KR*-value depending on the position of the knot over the depth of the beam can now be formulated by inserting equations (5.21) and (5.22) in equation (5.29):

$$red, ratio_{MOE, max} = \frac{25 \frac{(h-d)^3}{h^3}}{9+16 \frac{(h-d)^3}{h^3}} \quad (5.30)$$

$$red, ratio_{MOE, min} = \frac{25 * (1 - \frac{d^3}{h^3})}{9+16 * (1 - \frac{d^3}{h^3})} \quad (5.31)$$

The reduction of the bending strength and *MOE* as a function of the knot ratio *KR* is visualised in figures 5.20 and 5.21. For comparison with real data, the results for sample L2 of Siberian larch are also plotted in the figures. The unreduced values for the bending strength and MOE_{local} for sample L2 were calculated by taking the average values for these properties of the 12 pieces for which *KR* was zero. The *KR*-values were calculated as the single knot parameter, so in accordance with figure 5.18. Figure 5.21 can also be made with E_{dyn} instead of MOE_{local} which will give the same trend, due to the high correlation of the relationship between the two properties.

Figures 5.20 and 5.21 show that the theoretical and experimental reduction ratios follow the same trends. It is expected that the test data is closer to the minimum influence than to the maximum influence, because the position of the knot with the maximum influence will only occur incidentally. Because of the scatter in the position of the influencing knot in a beam, the regression line will be somewhere in between the maximum and minimum lines. The figures show that the influence of a knot has more influence on the bending strength than on the *MOE*.

In machine grading, the non-destructive measured values of the *MOE* are used. It is therefore interesting to show the change in the ratio $red, ratio_{MOE}/red_{fm}$. In figure 5.22, this is plotted for the theoretical maximum and minimum influence with the test data of sample L2.

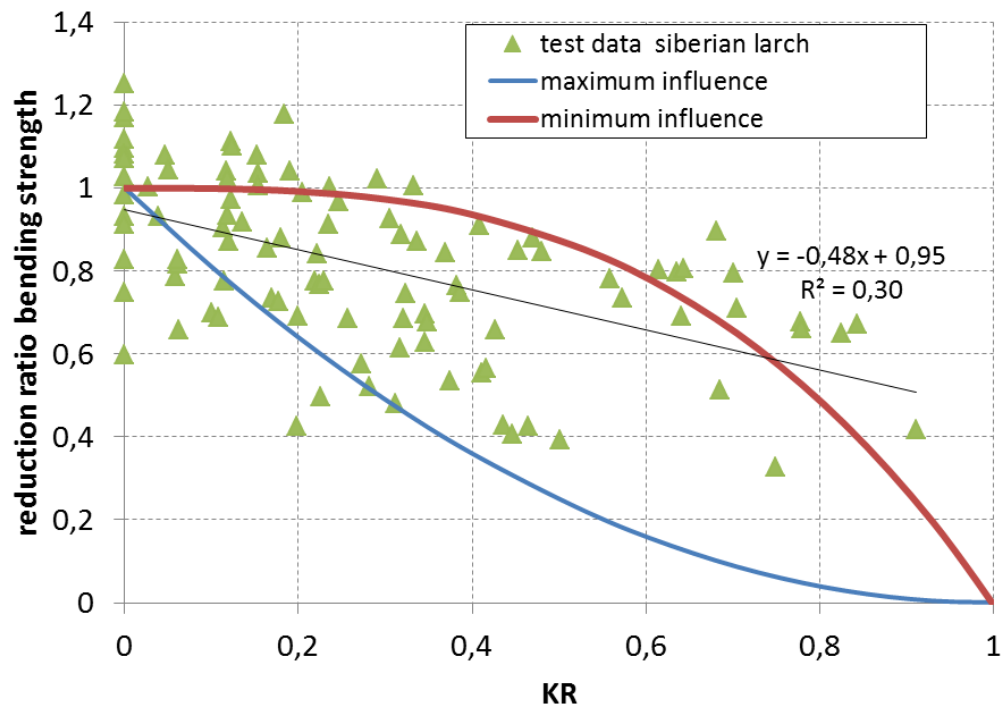


Figure 5.20. Reduction ratio of the bending strength plotted against the knot ratio based on the theory and based on the test data of sample L2.

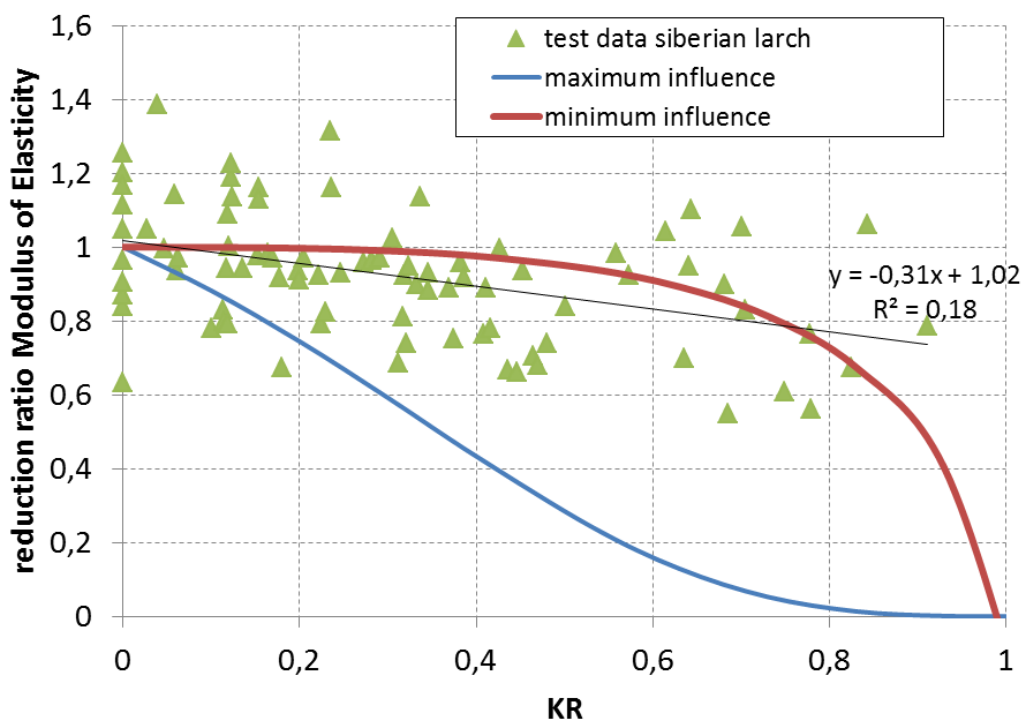


Figure 5.21. Reduction ratio of the MOE plotted against the knot ratio based on the theory and based on the test data of sample L2.

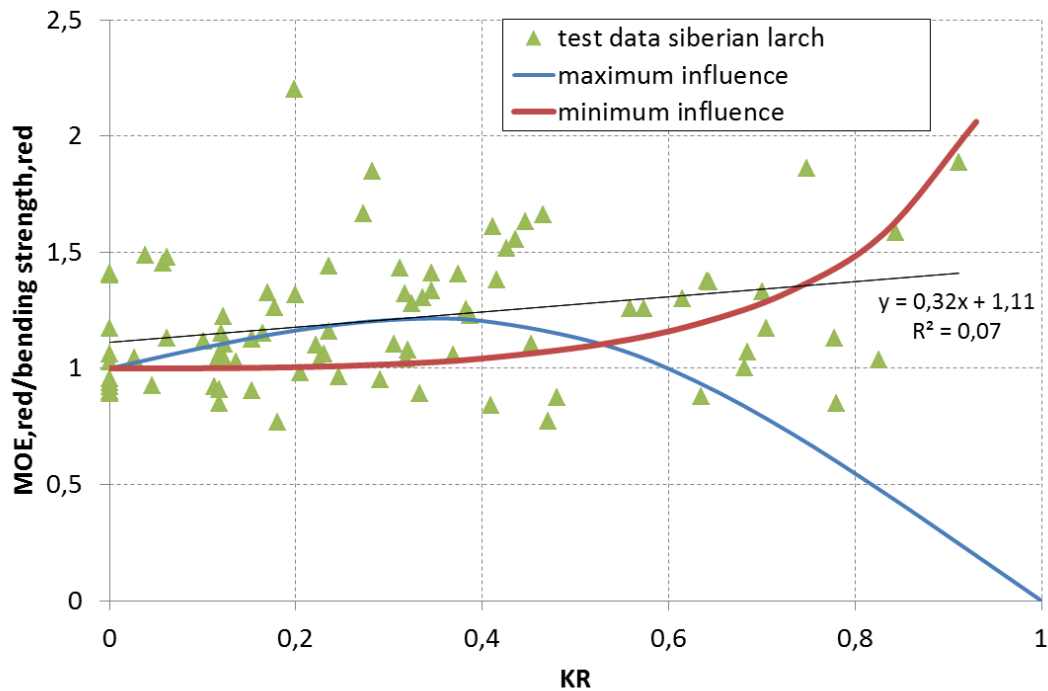


Figure 5.22. $MOE_{red}/f_{m_{red}}$ plotted against the knot ratio based on the theory and for the test data of sample L2.

It is interesting to see that the maximum and minimum ratios for $MOE_{reduced}/f_{m_{reduced}}$ are very close for KR ratios below 0.6. At the intersection point of minimum and maximum ratios at $KR=0.53$, the ratio $MOE_{reduced}/f_{m_{reduced}}$ is increased to 1.1. The regression line of the test data from sample L2 in figure 5.22 show an increase of 16% from $KR=0$ to $KR = 0.53$, but the datacloud also shows that in practice this increase might be difficult to detect because of the scatter in test results. This can explain why for bending tests of species with knots a rather constant value for the ratio between MOE and bending strength is found.

This is illustrated with the test data of 100 pieces of sample L2 in figure 5.23, where the bending strength is plotted against the MOE_{local} . This figure shows a linear relationship between bending strength and MOE .

Figures 5.20 to 5.23 show that with simplified mechanical models experimentally found relationships between knot ratio and bending strength, between knot ratio and MOE , and between MOE and bending strength can be explained. The fact that the MOE is a good predictor of the bending strength for timber beams with knots therefore has a physical basis. The figures show that the correlation between the non-destructive MOE and the bending strength is higher than that between the knot ratio and the bending strength. This is caused by the simplifications in the mechanical model and because the detection of defects in the timber by measuring the MOE is more direct than by visual observations.

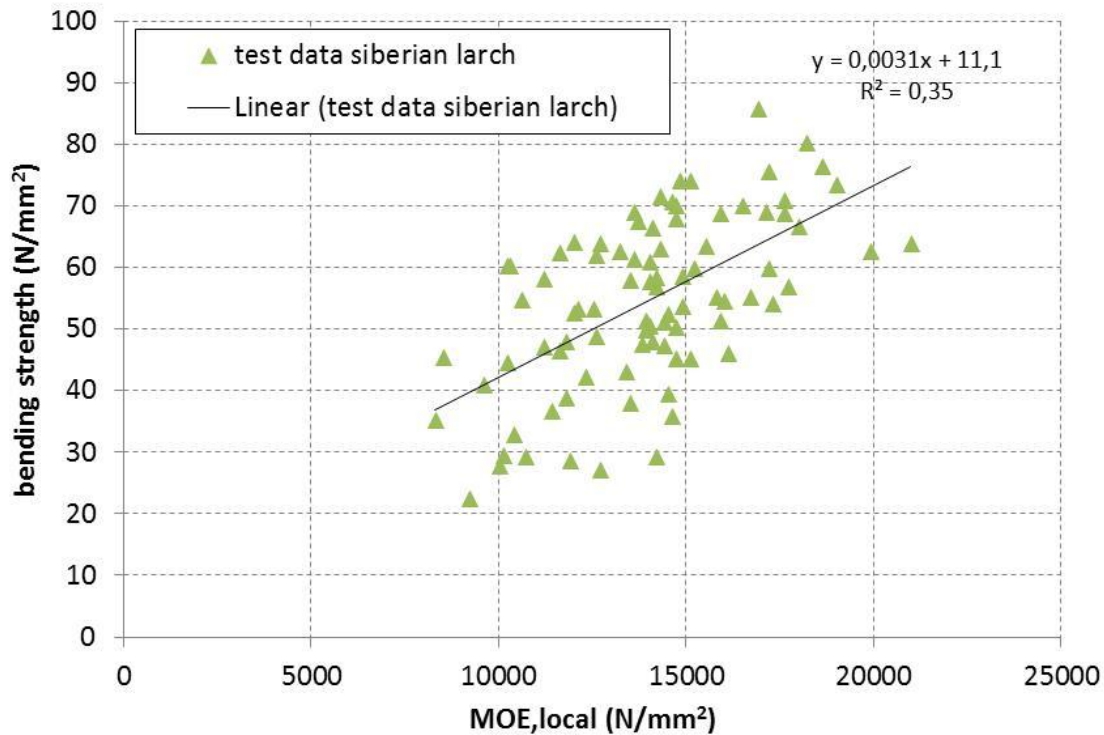


Figure 5.23 Bending strength plotted against MOE_{local} for sample L2.

In figure 5.23 can be observed that there is a mean prediction line and that there is scatter around this line. To be able to safely grade pieces in a strength class, the magnitude of the scatter has to be known. This will be investigated in chapter 6.

The mechanical model in this chapter is based on the definition of KR of figure 5.18. Here, it is assumed that the knot size is the same on both adjacent sides of the timber beam. In reality, the knots will differ in size because knots are cut of branches that grow from the pith of the tree. This can be taken into account by using the knot area ratio KAR which is defined as the ratio between the (interpolated) knot area in the cross section and the area of the whole cross section. The prediction capability of the KAR is better than the KR , because the 3D effect of the influence of knots is better addressed. Denzler (2007) found an improvement of knot ratios based on surface measurements to cross sectional measurements from approximately $r^2 = 0.20$ to $r^2 = 0.36$. The principle of KAR measurements can be used in X-ray machines, as Schajer (2001) showed. X-ray machines can detect differences in density. Because the density of knots is higher than that of clear wood, the density of a part of a board with a knot (or knots) divided by the density of the knot-free parts of the board gives an equivalent of the KAR -value of that part of the board. Schajer assumed a constant relation of the clear wood strength related to the density. By a regression of the estimated strength with the tested strength, he found coefficients of determination in the range of $r^2=0.45-0.7$.

These findings support the assumption that the reduction of strength and stiffness due to knots can be modelled by the concept of clear wood connected with weak zones. These weak zones can be modelled by regarding knots as removed material, causing a reduced cross section. Based on mechanical models, it can be explained that the reduction of strength and stiffness can be predicted by linear relations with dimensional properties of the knots. The accuracy of the measurements of these dimensional properties determine the prediction capability. In this thesis, measurements for *KR* ratios (knot ratio based on surface dimensions of knots) are available for some samples in the database. A linear reduction of the strength and stiffness due to *KR* will be assumed as the level at which a physical explanation is available.

5.2 Prediction models for the strength of structural timber

5.2.1 Introduction

To model the strength of structural timber, the following approach is followed. Based on physical relationships, strength predicting models are formulated. These models contain measurable parameters. The factors determining the influence of these parameters have to be found by comparing the model with experiments. This will be done by linear or non-linear regression. When the predictions are compared with the observations from test results, there will be a scatter around the prediction line. The shape and magnitude of this scatter has to be known to be able to use the prediction models in the grading process. In this section, strength predicting models for clear wood, timber containing grain angle deviation (and no knots), timber containing knots (and no grain angle deviation) and timber containing both grain angle deviation and knots will be formulated.

Based on the findings in chapter 2 and section 5.1 the following assumptions are made:

- The strength and stiffness of clear wood depends on the density.
- The scatter around the prediction lines of the bending strength based on the density for clear wood and the stiffness based on the density for clear wood is considered to be the natural variability of wood.
- Structural timber is considered as clear wood with strength and stiffness reducing characteristics.
- The reduction of strength and stiffness due to grain angle deviation can be described by Hankinsons equations.
- The reduction of strength and stiffness due to the presence of knots is described by regarding the timber as clear wood connected by weak zones. The reduction due to knots on strength and stiffness can be explained by regarding knots as holes.
- The Hankinson reduction equation due to grain angle deviations and the knot ratio reduction equations will be regarded as the physical models explaining the strength and stiffness of structural timber.

In this thesis, it is the objective to develop species independent strength models. For visual grading, grain angle deviations and knot ratios are available as strength and stiffness predicting parameters. For machine grading, the density and the MOE are available as strength and stiffness predicting parameters. The next sections deal with the effect of these parameters on the formulation of the prediction models for strength and stiffness to be used in regression analysis.

5.2.2 Prediction model for the bending strength based on the Modulus of Elasticity and density of timber for clear wood.

The main assumption in this thesis is that structural timber is regarded as clear wood with strength and stiffness reducing characteristics. Therefore, the clear wood strength has to be quantified. In chapter 2 it was found from literature that for small clear wood the density is the basic parameter that determines the strength and stiffness. For timber with knots of structural sizes also a relation of the density with strength and stiffness is assumed for the clear wood zones (see section 5.1). The influence of size will be studied in 5.3.

Based on figures 2.5 and 2.6, the clear wood strength and stiffness is formulated as linear depending on the density:

$$f_{m,0} = C_1\rho + C_{k1} \quad (5.32)$$

$$MOE_0 = C_2\rho + C_{k2} \quad (5.33)$$

Where:

$f_{m,0}$ is the bending strength for clear wood, the zero indication no grain angle deviation.

ρ is the density of clear wood.

MOE_0 is the modulus of elasticity for clear wood, with no grain angle deviation.

C_1 describes the ratio between the clear wood strength and the density.

C_2 describes the ratio between the clear wood stiffness and the density.

Equations (5.32) and (5.33) describe the relationship between datapoints of measurements of the density (ρ) and the bending strength ($f_{m,0}$). That means that the distribution of the observed bending strength values $f_{m,0}$, depends on the distribution type of the density (ρ).

C_{k1} and C_{k2} are factors describing effects causing that the regression lines do not go through the origin. For instance, the loading configuration (three-point or four-point bending tests) will give different values for the same density. This might lead to a change of all factor values. Of course there can be a situation that factors C_{k1} and C_{k2} are zero.

Equations (5.32) and (5.33) describe the mean regression lines. The scatter around these lines will be formulated in equations (5.36) and (5.37). Of course the C -factors can also be stochastic, however this scatter will be combined with the overall scatter around the regression lines.

Factors C_1 , C_2 , C_{k1} , C_{k2} can be derived from regression analysis on test data.

Equation (5.33) can be reformulated as:

$$\rho = \frac{MOE_0}{C_2} - \frac{C_{k2}}{C_2} \quad (5.34)$$

Inserting (5.34) in equation (5.32) gives:

$$f_{m,0} = \frac{C_1}{C_2} MOE_0 - \frac{C_1}{C_2} C_{k2} + C_{k1} \quad (5.35)$$

This explains the good relationship between the MOE and the bending strength that can be found from tests. From figures 2.5 and 2.6, values for $C_1=0.15$ and $C_2=18.17$ are found, with C_{k1} and C_{k2} both being zero. This gives an expected coefficient $0.15/18.17= 0.0083$ between bending strength and MOE, which is very close to the coefficient value of 0.0081 which is found from tests, see figure 2.7.

To be able to use correlation lines for grading purposes, not only the coefficients predicting the mean regression line are required, but also information about the scatter around this line. To get insight in the scatter for clear wood, the test results of 5 different tropical hardwood species tested in the Netherlands in the 1960s are evaluated, reported in Houtinstituut TNO (1961,1961,1961,1962,1962). It concerns samples of approximately 50 clear wood pieces for every wood species with a cross section of 50 x 50 mm². They were tested in a three-point bending test with a span of 700 mm. In table 5.5, the mean density for each wood species is listed together with the coefficients of variation for the density, the bending strength and the MOE. The low c.o.v. of the densities is in line with values given in the Wood Handbook (Ross et al., 2010) for clear wood.

Table 5.5. Coefficients of variation of clear wood samples of 5 wood species.

Wood species	Mean density at		c.o.v. bending strength	c.o.v. MOE
	12% m.c. (kg/m ³)	c.o.v. density		
tola branca	682	0.10	0.16	0.20
peros de campos	770	0.10	0.16	0.15
yang	836	0.07	0.16	0.17
iroko	873	0.07	0.14	0.15
basralocus	892	0.10	0.15	0.16

Something that can be observed is that the coefficient of variation (= standard deviation divided by the mean value) of the bending strength and the MOE are rather constant for increasing density. This means that the scatter for the bending strength and MOE will increase for increasing density. This can be observed in figures 2.5 and 2.6 where for both bending strength and MOE an increasing scatter with increasing density can be seen. This scatter is assumed to be the natural variation on clear wood level. This natural variation is the basis to eventually predict the scatter in prediction models on the level of structural timber with defects. For comparison, the Wood Handbook (Ross et al., 2010) gives for clear wood average coefficients of variation of 0.16 for the bending strength and of 0.20 for the MOE.

The natural variation of the bending strength for pieces with the same density is denoted as an error ε_f . The natural variation of the *MOE* for pieces with the same density is denoted as an error ε_M .

In the following derivation, factors C_{k1} and C_{k2} are assumed to be zero. When they are not zero, they will be included in a constant in the equations for structural timber.

The natural variation for the bending strength ε_f for a certain density value is introduced in equation (5.32) and the natural variation ε_M for the *MOE* for a certain density value is introduced in equation (5.33). Then these equations become:

$$f_{m,0} = \rho C_1 + \varepsilon_f \quad (5.36)$$

$$MOE_0 = \rho C_2 + \varepsilon_M \quad (5.37)$$

The errors ε_f and ε_M are considered to be normally distributed with a mean value of zero. As mentioned before, they increase with increasing density. This will be elaborated further. The objective of this section is to investigate how these errors propagate through the equations for structural timber with strength reducing characteristics. The factors C might not be an exact constant either. These may also be a stochastic, however, the errors in the factors C are neglected because they are expected to be included in ε_f and ε_M .

Equation (5.37) can be written as:

$$\rho C_2 = MOE_0 - \varepsilon_M \quad (5.38)$$

$$\rho = \frac{MOE_0 - \varepsilon_M}{C_2} \quad (5.39)$$

Inserting (5.39) in (5.36) gives:

$$f_{m,0} = \frac{MOE_0 - \varepsilon_M}{C_2} C_1 + \varepsilon_f \quad (5.40)$$

$$f_{m,0} = \frac{C_1}{C_2} MOE_0 - \frac{C_1}{C_2} \varepsilon_M + \varepsilon_f \quad (5.41)$$

To predict the bending strength of clear wood from *MOE*-measurements, the model line becomes:

$$f_{m,0,mod} = \frac{C_1}{C_2} MOE_0 \quad (5.42)$$

And the scatter around this line is

$$\varepsilon_{f_{m,0}} = -\frac{C_1}{C_2} \varepsilon_M + \varepsilon_f \quad (5.43)$$

The overall mean value of $\varepsilon_{f_{m,0}}$ is zero, but when the individual values of $\varepsilon_{f_{m,0}}$ are plotted against the model values according to equation (5.42), the mean value of $\varepsilon_{f_{m,0}}$ for a specific model value does not have to be zero. This depends on whether ε_f and ε_M are correlated or not. When they are two totally independent variables (the coefficient of determination r^2 between the two is then zero, no correlation) then the mean value of $\varepsilon_{f_{m,0}}$ for each model value will be zero. When they are correlated, this will not be the case. In that case the ratio C_1/C_2 is not the factor found in a least squares regression. This will be addressed further on in this section.

To formulate the effect of the correlation between ε_f and ε_M the correlation between two normal distributions will be considered as a starting point.

According to Agnew and Constable (2008), for two correlated normal distributions X_1 and X_2 the conditional probability density function (pdf) for X_2 with x_1 given is:

$$\phi_{X_2|X_1=x_1} = \frac{1}{\sqrt{2\pi\sigma_2^2(1-r^2)}} e^{-\frac{[x_2-\mu_2-r\frac{\sigma_2}{\sigma_1}(x_1-\mu_1)]^2}{2\sigma_2^2(1-r^2)}} \quad (5.44)$$

Where r^2 is the coefficient of determination between X_1 and X_2 .

From (5.44), it can be seen that the expected value of X_2 is:

$$E[X_2|X_1 = x_1] = \mu_2 + r\frac{\sigma_2}{\sigma_1}(x_1 - \mu_1) \quad (5.45)$$

This is the regression line for X_2 based on X_1 . From (5.44) it can be seen that the standard deviation around this regression line is

$$\sigma_{X_2|X_1=x_1} = \sigma_2\sqrt{(1-r^2)} \quad (5.46)$$

This means that when two normal distributions are correlated, there is a constant variance around the regression line when one of them is expressed in the other.

In the case of clear wood ε_f and ε_M are normal distributions that might be correlated.

ε_f and ε_M are expected to be normally distributed around zero with standard deviations σ_{ε_f} and σ_{ε_M} .

ε_M and ε_f can be modeled as

$$\varepsilon_M = X_1\sigma_{\varepsilon_M} \quad (5.47)$$

$$\varepsilon_f = X_2\sigma_{\varepsilon_f} \quad (5.48)$$

With X_1 and X_2 as standard normal distributions $N(0,1)$.

Then ε_f can be modeled for a given value of ε_M as:

$$\varepsilon_f = r\frac{\sigma_{\varepsilon_f}}{\sigma_{\varepsilon_M}}X_1\sigma_{\varepsilon_M} + X_2\sigma_{\varepsilon_f}\sqrt{(1-r^2)} \quad (5.49)$$

$$\varepsilon_f = r\sigma_{\varepsilon_f}X_1 + X_2\sigma_{\varepsilon_f}\sqrt{(1-r^2)} \quad (5.50)$$

The model line for ε_f is:

$$\varepsilon_{f,mod} = r\sigma_{\varepsilon_f}X_1 \quad (5.51)$$

And the residual around this line is

$$res, \varepsilon_{f,mod} = X_2\sigma_{\varepsilon_f}\sqrt{(1-r^2)} \quad (5.52)$$

By applying the formulas of propagation of error it shows that the residuals have a constant variance with a value of

$$var, \varepsilon_{f,mod} = \sigma_{\varepsilon_f}^2(1-r^2) \quad (5.53)$$

And the standard deviation of the residuals is:

$$\sigma, \varepsilon_{f,mod} = \sigma_{\varepsilon_f}\sqrt{(1-r^2)} \quad (5.54)$$

The magnitudes of the standard deviations ε_M and ε_f depend on the coefficient of variation of the *MOE* and bending strength. It was observed that for clear wood the coefficients of variation for the *MOE* and bending strength are almost constant for every density value (see also table 5.5). This is formulated in equations (5.55) and (5.56):

$$\sigma_{\varepsilon_f} = v_F \rho C_1 \quad (5.55)$$

$$\sigma_{\varepsilon_M} = v_M \rho C_2 \quad (5.56)$$

Where v_f is the coefficient of variation for the bending strength and v_M the coefficient of variation for the *MOE*.

Then equations (5.47) and (5.48) become:

$$\varepsilon_M = X_1 v_M \rho C_2 \quad (5.57)$$

$$\varepsilon_f = X_2 v_F \rho C_1 \quad (5.58)$$

Equation (5.50) then becomes:

$$\varepsilon_f = r X_1 v_F \rho C_1 + X_2 v_F \rho C_1 \sqrt{(1 - r^2)} \quad (5.59)$$

The model line for ε_f is:

$$\varepsilon_{f,mod} = r X_1 v_F \rho C_1 \quad (5.60)$$

And the residual around this line is

$$res, \varepsilon_{f,mod} = X_2 v_F \rho C_1 \sqrt{(1 - r^2)} \quad (5.61)$$

By applying the formulas of error propagation, the residuals have a constant variance with a value of

$$var, \varepsilon_{f,mod} = (v_F \rho C_1)^2 (1 - r^2) \quad (5.62)$$

And the standard deviation of the error around the model line is:

$$s_y = v_F \rho C_1 \sqrt{(1 - r^2)} \quad (5.63)$$

This constant variance for the residuals (and the constant standard deviation for the standard error) is the case only when the observed data are pieces with the same density. When a range of densities is regarded, the errors might overlap.

To investigate whether a correlation between ε_M and ε_f exists, the underlying data of figures 2.5 and 2.6 from the Houtvademecum (Wiselius, 2010) are studied. From (5.57) and (5.58) it appears that the correlation between ε_M and ε_f can only be studied for pieces with the same density. For the data from the Houtvademecum there is a whole range of densities. In that case, the correlation between X_1 and X_2 can be studied. This can be done by calculating individual values for $X_1 \cdot v_F$ and $X_2 \cdot v_M$ by modifying equations (5.36) and (5.37), with equations (5.57) and (5.58) inserted:

$$(X_1 v_F)_i = \frac{f_{m,0,i} - \rho C_{1,i}}{\rho C_{1,i}} \quad (5.64a)$$

$$(X_2 v_M)_i = \frac{MOE_{0,i} - \rho C_{2,i}}{\rho C_{2,i}} \quad (5.64b)$$

The $(X_1 v_F)_i$ values are plotted against the $(X_2 v_M)_i$ values in figure 5.24.

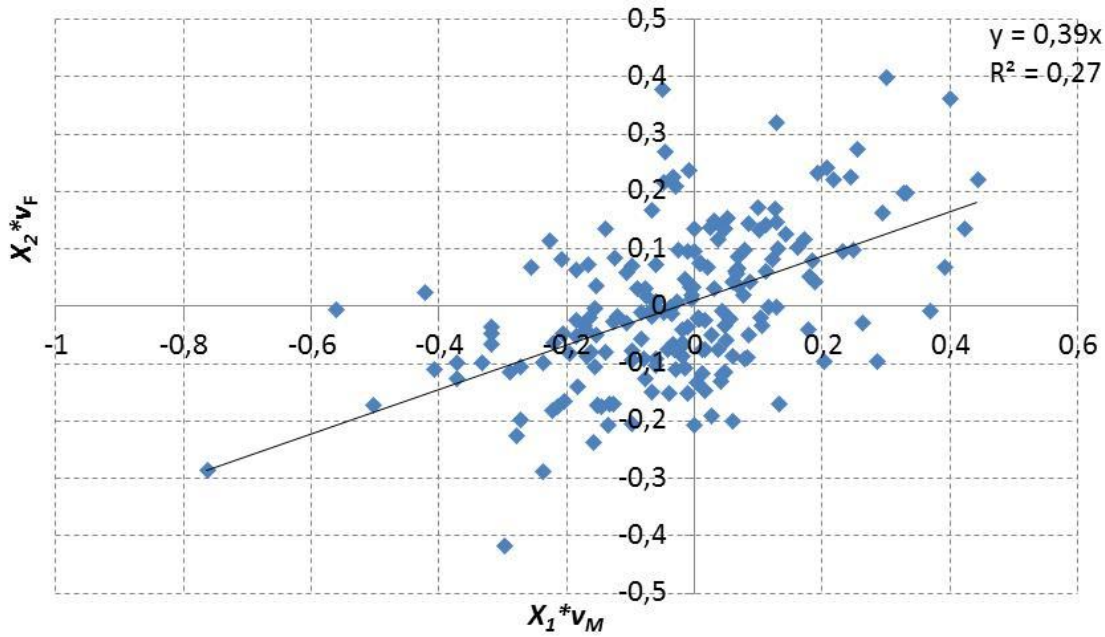


Figure 5.24. $(X_1 \cdot v_F)_i$ values plotted against the $(X_2 \cdot v_M)_i$ values for the dataset of small clear wood sample, data from the Houtvadamecum (Wiselius,2010).

Figure 5.24 shows that $X_1 \cdot v_F$ and $X_2 \cdot v_M$ are correlated. The coefficient of determination that is found is $r^2 = 0.27$. Because v_F and v_M are constants X_1 and X_2 are also correlated with the same value for the coefficient of determination. The correlation is positive, which means that for a specific density when ε_M increases, also ε_f increases. Figure 5.24 gives the correlation of the mean values of 192 species that are evaluated. It is expected that this correlation will also show up when individual test piece values are evaluated. Because X_1 and X_2 are standard normal distributions, the values of v_F and v_M are simply the standard deviations of $X_1 \cdot v_F$ and $X_2 \cdot v_M$. The mean values of $X_1 \cdot v_F$ and $X_2 \cdot v_M$ are both 0. The standard deviation of $X_1 \cdot v_F = 0.13$, so $v_F = 0.13$. The standard deviation of $X_2 \cdot v_M = 0.18$, so $v_M = 0.18$. The expected constant standard deviation around the regression line according to equation (5.46) is $0.13\sqrt{1-0.27} = 0.11$. When the standard error is calculated from the test data also a value of 0.11 is found.

Figure 5.24 shows that $X_1 \cdot v_F$ can be predicted from $X_2 \cdot v_M$ by multiplying it by 0.39. Calculated with equation (5.45) a slope of $0.52 \cdot 0.13 / 0.18 = 0.38$ is found, very close to 0.39. According to (5.45) the relationship between X_1 and X_2 is $X_2 = r \cdot X_1$ (because the standard deviations of both X_1 and X_2 are 1). With $r^2 = 0.27$ found in figure 5.24 this becomes $X_2 = 0.52 \cdot X_1$. Figure 5.25, where X_2 is plotted against X_1 confirms this. X_1 and X_2 are both random variables following a standard normal distribution.

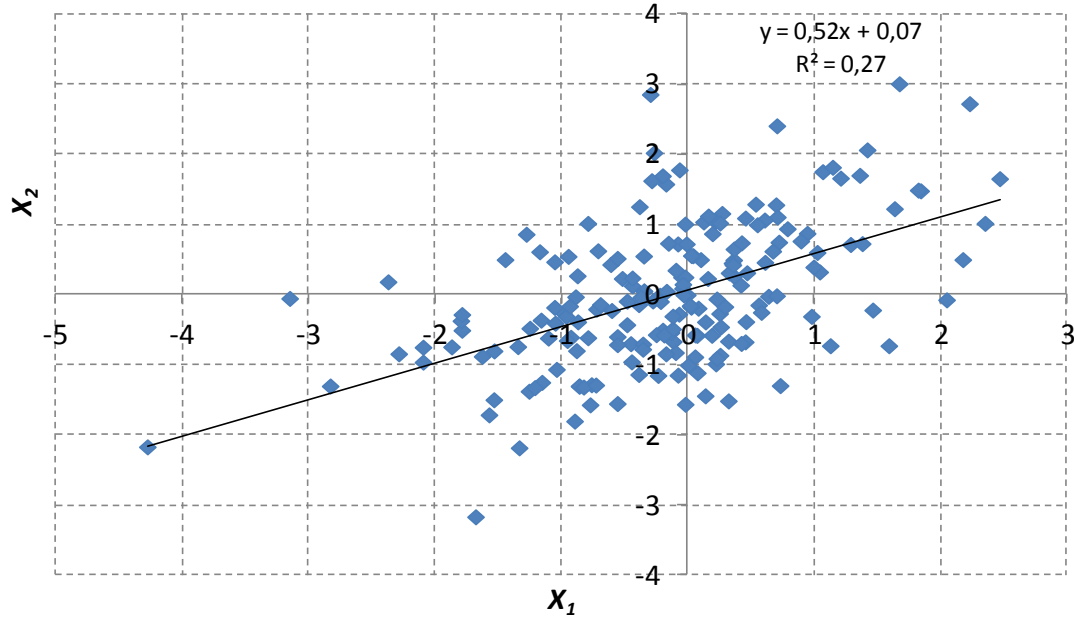


Figure 5.25. $(X_1)_I$ values are plotted against the $(X_2)_i$ values for the dataset of small clear wood samples, data from the Houtvadamecum (Wiselius,2010).

To illustrate the influence of the positive correlation between X_1 and X_2 values for X_1 and X_2 are randomly generated from the standard normal distribution with different coefficients of determination. The mean densities from the database of Houtvadamecum (Wiselius, 2010) are used, together with $v_F = 0.13$ and $v_M = 0.18$, to calculate the bending strength and the *MOE* with subsequently $r^2 = 0.0$, $r^2 = 0.27$ and $r^2 = 1.0$ as coefficients of determination between X_1 and X_2 . The simulated bending strengths are plotted against the simulated *MOEs* in figure 5.26 (the regression lines are forced through the origin). Figure 5.26 shows that the slope of the regression line stays practically the same for the three situations, but that the value of r^2 between X_1 and X_2 has a significant influence on the coefficient of determination between the modeled bending strength and the modeled *MOE*, and by that on the scatter around the regression line. The middle plot of figure 5.26 gives practically the same coefficient of determination as the original data in figure 2.7, which confirms the theoretical considerations in this section.

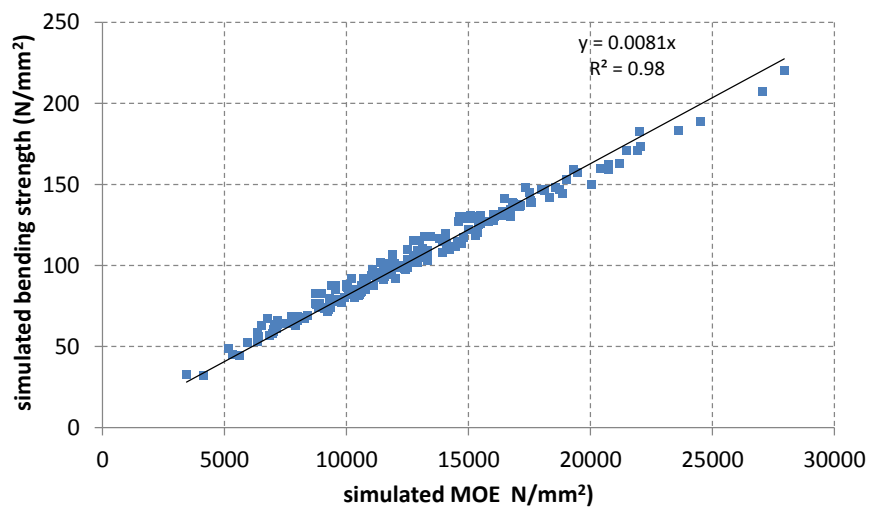
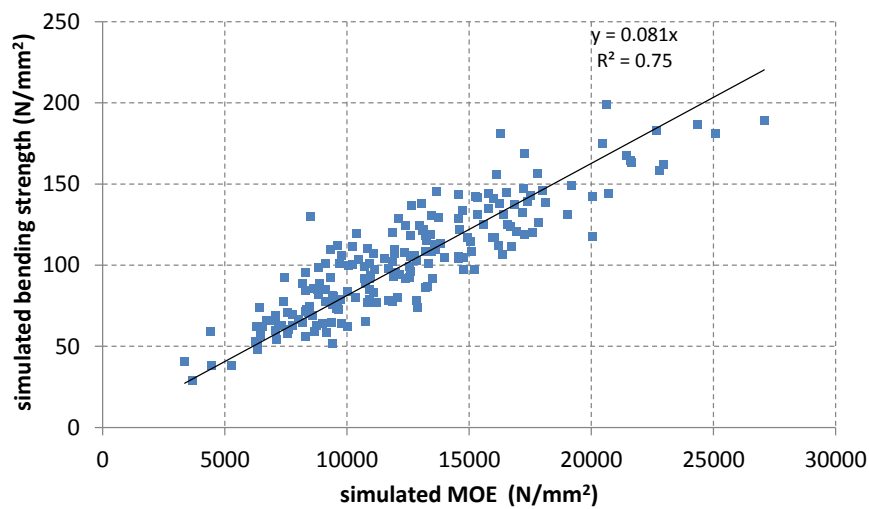
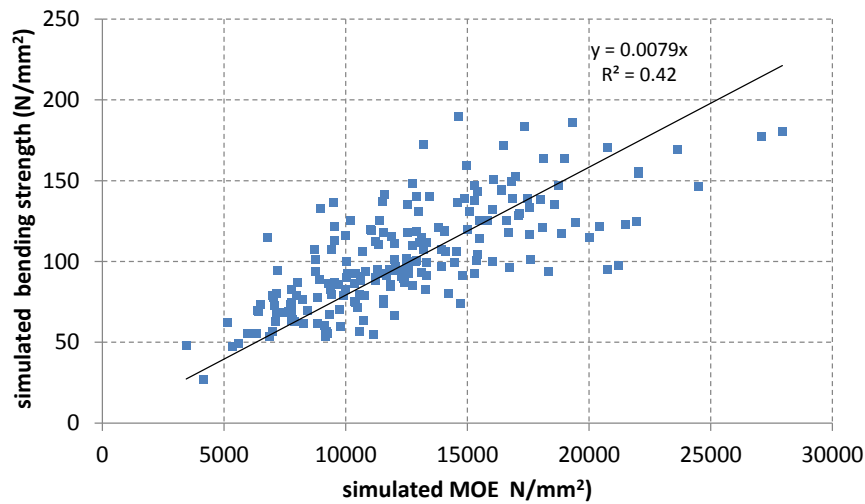


Figure 5.26. Regression plots of simulations of the bending strength and MOE with correlated randomly generated X_1 - and X_2 -values with the densities from the Houtvadamecum (Wiselius, 2010). Coefficients of determination between X_1 and X_2 are $r^2=0$ (top), $r^2=0.27$ (middle) and $r^2=1.0$ (bottom).

Modeling the relationship between the bending strength and MOE for clear wood for fixed densities.

It has been shown that the relationship between MOE and bending strength for clear wood is influenced by the fact that both properties are linearly related to the density and that the errors for both properties in these relations are correlated. To perform species independent grading, a wide range of densities will be included and as a consequence the errors will overlap. The effect on the mean regression line and the scatter around this line has to be investigated and quantified. To do this, first the relationship between bending strength and MOE for clear wood for a fixed density will be studied and after that this will be extended to a density distribution.

Including the correlation between $X_1 \cdot v_F$ and $X_2 \cdot v_M$ in equations (5.41 to 5.43) gives:

$$f_{m,0} = \frac{c_1}{c_2} MOE_0 - \frac{c_1}{c_2} X_1 v_M \rho C_2 + r X_1 v_F \rho C_1 + X_2 v_F \rho C_1 \sqrt{(1 - r^2)} \quad (5.65)$$

$$f_{m,0,mod} = \frac{c_1}{c_2} MOE_0 \quad (5.66)$$

$$\varepsilon_{f_{m,0}} = -X_1 v_M \rho C_1 + r X_1 v_F \rho C_1 + X_2 v_F \rho C_1 \sqrt{(1 - r^2)} \quad (5.67)$$

Equation (5.37) can be rewritten to:

$$MOE_0 = \rho C_2 + X_1 v_M \rho C_2 \quad (5.68)$$

Then (5.65) becomes:

$$f_{m,0} = \rho C_1 + r X_1 v_F \rho C_1 + X_2 v_F \rho C_1 \sqrt{(1 - r^2)} \quad (5.69)$$

And (5.66) becomes:

$$f_{m,0,mod} = \rho C_1 + X_1 v_M \rho C_1 \quad (5.70)$$

When a fixed density is assumed, MOE_0 and $f_{m,0}$ will be normally distributed. The equation for the model line between observed and modelled data can be calculated based on equations (5.66) and (5.65).

A plot is considered where the observations of $f_{m,0}$ (5.65) are plotted on the y-axis against the model values $f_{m,0,mod}$ (5.66) on the x-axis. The datapoints with $(X_1=0, X_2=0)$ and $(X_1=1, X_2=0)$ are on the model line.

For $(X_1=0, X_2=0)$ it follows that:

$$x_1 = \rho C_1$$

$$y_1 = \rho C_1$$

For $(X_1=1, X_2=0)$ it follows that:

$$x_2 = \rho C_1 + C_1 v_M \rho$$

$$y_2 = \rho C_1 + r v_F \rho C_1$$

The slope A can be calculated with:

$$A = \frac{y_2 - y_1}{x_2 - x_1} = \frac{r v_F \rho C_1}{v_M \rho C_1} = r \frac{v_F}{v_M}$$

Now equation $y = Ax + B$ can be formulated as

$$y = A(x - \mu_{x_1}) + \mu_{x_2}$$

This becomes in this case:

$$y = r \frac{v_F}{v_M} \left(\frac{C_1}{C_2} MOE_0 - \frac{C_1}{C_2} \rho C_2 \right) + \rho C_1 = r \frac{v_F}{v_M} \frac{C_1}{C_2} MOE_0 + \rho C_1 - r \frac{v_F}{v_M} \rho C_1$$

Inserting equation (5.66) gives:

$$y = r \frac{v_F}{v_M} f_{m,0,mod} + (\rho C_1 - r \frac{v_F}{v_M} \rho C_1)$$

The residuals around the model line will be according to equation (5.63).

As an example, two datasets are simulated with fixed densities for all pieces. One dataset with a fixed density of $\rho = 400 \text{ kg/m}^3$ and one density of $\rho = 800 \text{ kg/m}^3$.

For $C_1 = 0.15$ and for $C_2 = 18$ is taken. For $v_F = 0.13$ and $v_M = 0.18$. The correlation coefficient between X_1 and X_2 is assumed to be $r = 0.5$. For both densities, 1000 values for X_1 and X_2 are simulated by taken them randomly from a standardized normal distribution, taking into account that they are correlated. With these generated values and the defined constants, the model values $f_{m,0,mod}$ according to equation (5.66) and the observations $f_{m,0}$ according to equation (5.65) are calculated. So both the model values $f_{m,0,mod}$ as the observed data $f_{m,0}$ are simulated !

The results are plotted in figure 5.27.

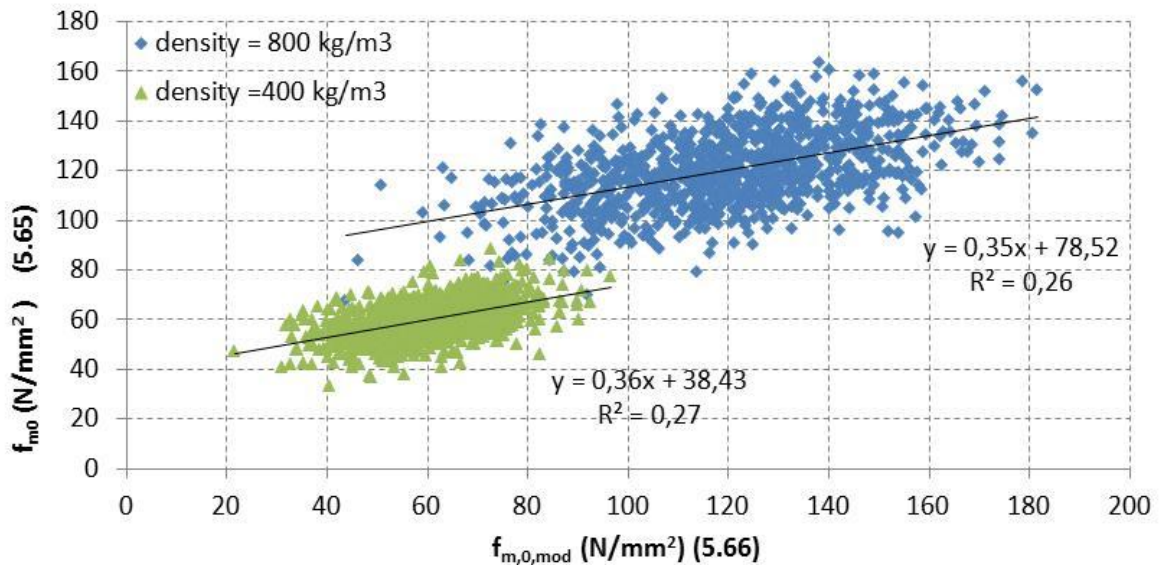


Figure 5.27. Simulated observed data plotted against simulated model data for 2 density values.

The expected regression equation for $\rho = 400 \text{ kg/m}^3$ becomes:

$$y = 0.5 (0.13/0.18) f_{m,0,mod} + (400 \cdot 0.15) - 0.5(0.13/0.18) (400 \cdot 0.15) = 0.36 f_{m,0,mod} + 38.3$$

The expected regression equation for $\rho = 800 \text{ kg/m}^3$ becomes:

$$y = 0.5 (0.13/0.18) f_{m,0,mod} + (800 \cdot 0.15) - 0.5(0.13/0.18)(800 \cdot 0.15) = 0.36 f_{m,0,mod} + 76.7$$

The regression lines through the simulated data practically coincide with the expected equations for the regression lines.

The coefficient of determination r^2 for pieces with the same density between the model values and the observed values will never be bigger than r^2 -value between X_1 and X_2 . This explains that for a sample of a hardwood species with very little variation in density within the sample, the lower the correlation is, the more homogeneous (little variation in grain angle deviation) the pieces of the sample are.

Modeling of the relationship between the bending strength and MOE for clear wood for distributed densities.

Now, supposing that the two generated samples of figure 5.27 are analyzed together to model the relationship between $f_{m,0}$ and $f_{m,0,mod}$. A least squares regression is performed using all these datapoints of these two samples. This will result in figure 5.28.

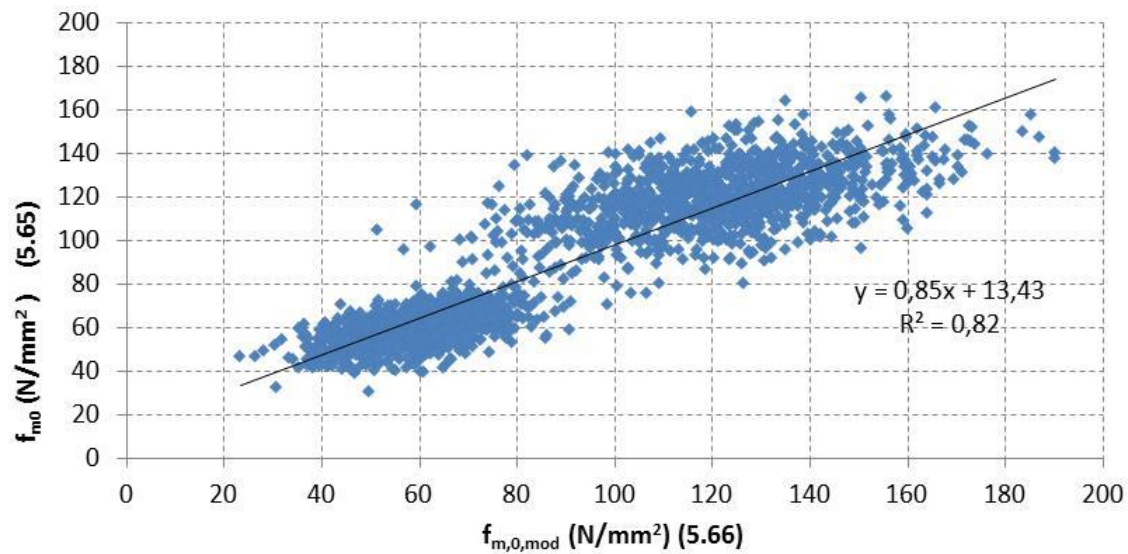


Figure 5.28. Simulated observed data plotted against simulated model when the 2 density values are considered as one data group.

The coefficient of determination r^2 increases significantly to $r^2=0.82$. The mean standard error around the regression line will be zero for all data, but not for individual model values. This can be explained by looking at the regression lines of the individual samples. The mean (x,y) -values for both samples will be on the overall least squares regression line, but all other (x,y) -pairs that are on the sample-regression line will not be on the

overall regression line. That also means that the residuals will not have a constant variance with the overall regression line. The situation of figure 5.28 is an extreme case. Normally the density will not be fixed to exactly two values, but will follow a distribution. This means that the dataclouds for different density values will overlap.

The effect of a least squares estimation will be that the mean of the standard error for every model value is as close to zero as possible. To calculate the slope of the mean regression line from a least squares estimation, the difference of $f_{m,0} - f_{m,0,mod}$ has to be calculated in every model point. Although one of the assumptions in linear regression is that the variance of the standard error is equal over the range of model values, figures 2.5 to 2.7 show that this is not the case for clear wood when the density increases. Therefore, also in every model point, the standard deviation of the standard error has to be calculated. When the regression line is transformed to coincide with $y=x$, the model value has to be multiplied by the intercept of the regression line and increased by the intercept. So in figure 5.28, the model value has to be multiplied by 0.85 and increased by 13.43 to achieve this. The shape of the scatter around the line $y=x$, however, will be the same as that of the original line. This can be used in the calculation of the errors for every model point.

For a certain density, the mean model value and the mean observed value will be on $y=x$ because both X_1 and X_2 are 0. The deviation from mean model values with the line $y=x$ for a value of X_1 for a certain density can easily be calculated with equation (5.67) without the error term due to X_2 around this mean:

$$diff(X_1) = -X_1 v_M \rho C_1 + r X_1 v_F \rho C_1 \quad (5.71)$$

For a certain model value $f_{mod,A}$ the value of X_1 can be calculated with

$$X_1 = \frac{f_{mod,A} - f_{mod,mean,\rho}}{\frac{C_1}{C_2} * \sigma_{\varepsilon,M,\rho}} = \frac{f_{mod,A} - \frac{C_1}{C_2} * MOE_{mean,\rho}}{\frac{C_1}{C_2} * \sigma_{\varepsilon,M,\rho}} \quad (5.72)$$

$MOE_{mean,\rho}$ can be replaced with ρC_2 and $\sigma_{\varepsilon,M}$ can be replaced with $v_M \rho C_2$. Then for a model value $f_{mod,A}$ (5.71) becomes:

$$diff(A, j) = \frac{f_{mod,A} - \rho_j * C_1}{v_M * \rho_j * C_1} \left(-\frac{C_1}{C_2} v_M \rho_j C_2 + r v_F \rho_j C_1 \right) \quad (5.73)$$

$$diff(A, j) = \frac{f_{mod,A} - \rho_j * C_1}{v_M} (-v_M + r v_F) \quad (5.74)$$

With the index j in ρ it is indicated that this is valid for a specific value of the density.

For every value of A , the difference of the mean model value with the line $y=x$ can be calculated with (5.74).

However, when there is a spread in densities, the mean differences and standard errors will overlap. For a certain model value $f_{mod,A}$ these overlaps have to be summed to find the difference with the model in that model point for the whole data. This has to be a weighted summation, depending on the amount of data in a specific model point due to a

certain density. The amount of data for a model value A for a certain density depends on the value of X_1 . So the probability of a certain model A value to occur for a certain density ρ_j is $\emptyset(f_{mod,A})$, being the standard normal distribution.

$$\emptyset(f_{mod,A,j}) = \emptyset\left(\frac{f_{mod,A} - \rho_j C_1}{v_M \rho_j C_1}\right) \quad (5.75)$$

In equation (5.75) the probability of data for model value A for a certain density ρ_j is expressed. To calculate the mean difference value with the line $y=x$ in model value A , we have to calculate the weighted mean difference R due to data in model value A caused by all densities.

The weighted mean difference R is calculated with:

$$R_{mean,A} = \frac{\int_{j=-\infty}^{\infty} pdf(\rho_j) \emptyset(f_{mod,A,j}) diff(A,j) dj}{\int_{j=-\infty}^{\infty} pdf(\rho_j) \emptyset(f_{mod,A,j}) dj} \quad (5.76)$$

$Pdf(\rho_j)$ is introduced in equation (5.76) to calculate the amount of data for model value A due to the amount of data present for every density value. When the density is normally distributed with mean μ_ρ and standard deviation σ_ρ , then the probability density function of the property the density becomes:

$$\emptyset(\rho_j) = \emptyset\left(\frac{\rho_j - \mu_\rho}{\sigma_\rho}\right) \quad (5.77)$$

And (5.76) becomes:

$$R_{mean,A} = \frac{\int_{j=-\infty}^{\infty} \emptyset(\rho_j) \emptyset(f_{mod,A,j}) diff(A,j) dj}{\int_{j=-\infty}^{\infty} \emptyset(\rho_j) \emptyset(f_{mod,A,j}) dj} \quad (5.78)$$

An analytical solution of (5.78) could not be found in literature. Therefore, in calculations a discretization is performed. The minimum and maximum values for j that are applied are -4 and $+4$ with increment steps of 0.1 N/mm^2 . These give sufficiently accurate results.

Then equation (5.78) becomes;

$$R_{mean,A} = \frac{\sum_{j=-4}^4 \emptyset(\rho_j) \emptyset(f_{mod,A,j}) diff(A,j) dj}{\sum_{j=-4}^4 \emptyset(\rho_j) \emptyset(f_{mod,A,j}) dj} \quad (5.79)$$

Secondly, the standard deviation in model point A due to overlapping densities has to be calculated. The standard deviation of the standard error in model point A due to data from a certain density ρ_j is

$$s_{y,j} = v_F \rho_j C_1 \sqrt{(1 - r^2)} \quad (5.80)$$

In this case, the value of X_1 does not have to be known, because the magnitude of the standard error is the same for each X_1 value.

For 2 populations following normal distributions with different amounts of data, the resulting standard deviation can be calculated in the following way:

$$s_{y,mean} = \sqrt{\frac{(n_1-1)[s_{y,1}^2 + (\bar{x}_1 - \bar{x}_{total})^2] + (n_2-1)[s_{y,2}^2 + (\bar{x}_2 - \bar{x}_{total})^2]}{n_1 + n_2 - 1}} \quad (5.81)$$

Where n_1 and n_2 are the number of datapoints for populations 1 and 2, $\bar{x}_1$ and $\bar{x}_2$ are the mean of population 1 and $\bar{x}_2$ population 2, $s_{y,1}$ and $s_{y,2}$ are the standard deviations of population 1 and 2, and $\bar{x}_{total}$ is the mean of all data of populations 1 and 2.

Applying this principle for the “mean” standard deviation in model point A, this becomes:

$$s_{y,A,mean} = \sqrt{\frac{\sum_{j=-4}^4 \varnothing(\rho_j) [s_{y,j}^2 + (R_{mean,A} - diff(A,j))^2] dj}{\sum_{j=-4}^4 \varnothing(\rho_j) dj}} \quad (5.82)$$

To compare the theoretical trend of the magnitude of the standard error of (5.82) with the observed trend of test data (either real test data or simulated test data), it has to be formulated how this observed trend is determined. Theoretically, for every model point, the mean standard error could be calculated from the residuals in that model point. However, this will only work when in every (discretized) model point a large number of test data is available. It is more likely that in a certain chosen interval there might be different amounts of pieces over this interval size over the range of model values, which makes the calculations unstable.

Therefore, the sliding standard deviation of the error is introduced, which calculates the standard deviation of the error over a fixed number of model values. The following procedure was followed:

- all pairs of model values-residuals are ranked, based on the model value, from low to high
- the standard deviation of k consecutive residuals is calculated. This means that the standard deviation is calculated for ranked model values 1 to k , then for 2 to $k+1$ etc. This is called the sliding standard deviation of the error:

$$s_{res,sliding,i} = \text{stdev}(res_{i-k} \dots res_i) \quad (5.83)$$

- For all ranked model values from $i = k$ to n the values of $s_{res,sliding,i}$ is calculated.
- Plotting the values of $s_{res,sliding,i}$ against the model values gives the trend line for the standard error.
- The number of k can be varied. In this thesis $k = 50$ will be used, unless indicated otherwise.

The theory of this section will be demonstrated by two examples of simulated data with different spread in density. Again, in these examples the model values as well as the observations are simulated.

Simulation 1.

The input for the simulations is:

A normally distributed density, with $\mu_\rho = 700 \text{ kg/m}^3$ and $\sigma_\rho = 50 \text{ kg/m}^3$.

The following constants: $C_1 = 0.15$; $C_2 = 18$; $v_F = 0.13$; $v_M = 0.18$, correlation between X_1 and X_2 : $r = 0.5$. Number of simulated datapoints: 2500.

Figure 5.29 shows the regression plot between the simulated model values and the simulated observed values. Figure 5.30 shows the residuals around the regression line of figure 5.29, together with the theoretical mean regression line, calculated in every model point according to (5.79). Figure 5.31 shows the residuals when the model values are adjusted to the equation of figure 5.29, together with the theoretical standard deviation of the error calculated with (5.81) in every model point, and the sliding standard deviation of the error calculated with (5.83).

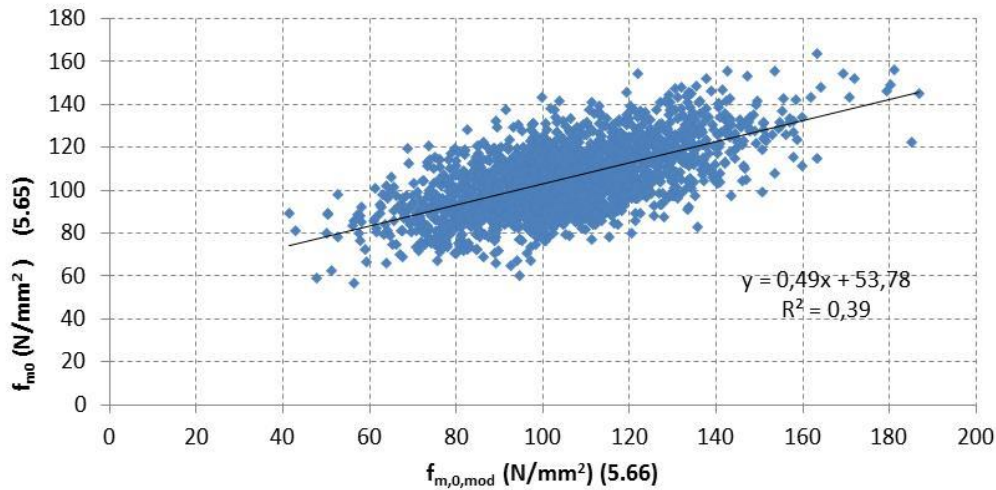


Figure 5.29. Simulated observed data ($n=2500$) plotted against simulated model data for simulation 1.

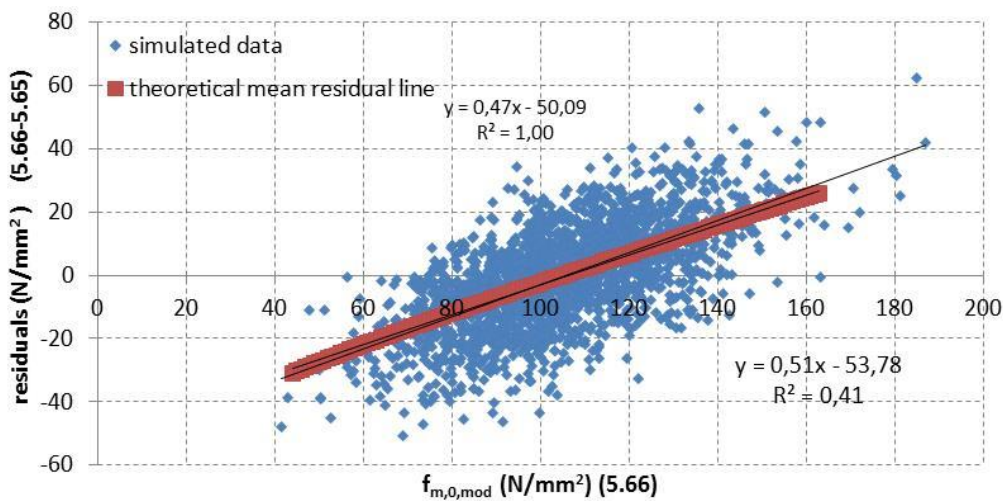


Figure 5.30. Simulated observed residuals ($n=2500$) plotted against simulated model data with the theoretical mean residual line for simulation 1.

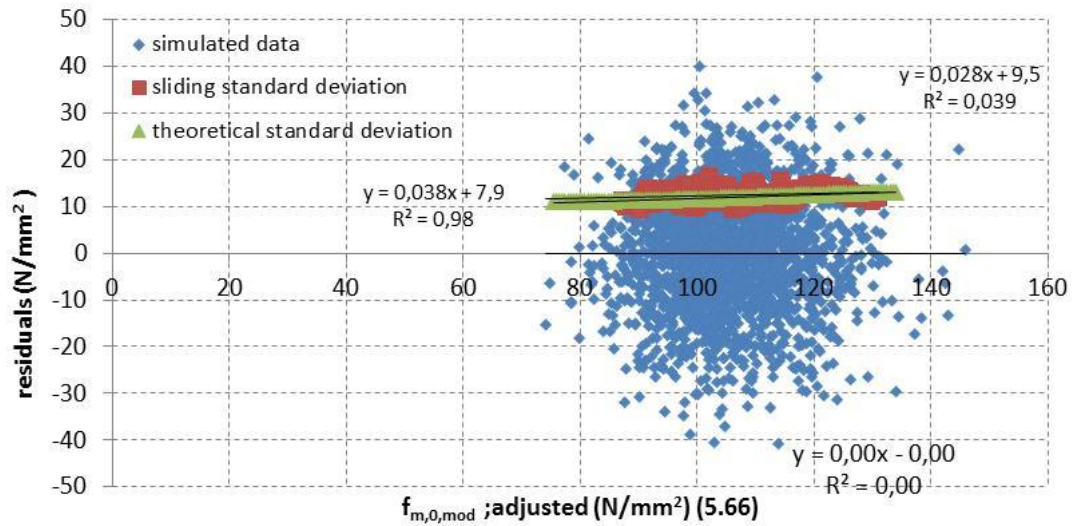


Figure 5.31. Simulated adjusted observed residuals($n=2500$) plotted against simulated model data with theoretically calculated standard deviation in every point and the sliding standard deviation line for simulation 1.

Simulation 2.

The input for the simulations is:

A normally distributed density, with $\mu_\rho = 700 \text{ kg/m}^3$ and $\sigma_\rho = 150 \text{ kg/m}^3$.

The following constants: $C_1 = 0.15$; $C_2 = 18$; $\nu_F = 0.25$; $\nu_M = 0.18$, correlation between X_1 and X_2 : $r = 0.5$. Number of simulated datapoints: 2500.

Figure 5.32 shows the regression plot between the simulated model values and the simulated observed values. Figure 5.33 shows the residuals around the regression line of figure 5.32, together with the theoretical mean regression line, calculated in every model point according to (5.79). Figure 5.34 shows the residuals when the model values are adjusted to the equation of figure 5.32, together with the theoretical standard deviation of the error calculated with (5.81) in every model point, and the sliding standard deviation of the error calculated with (5.83).

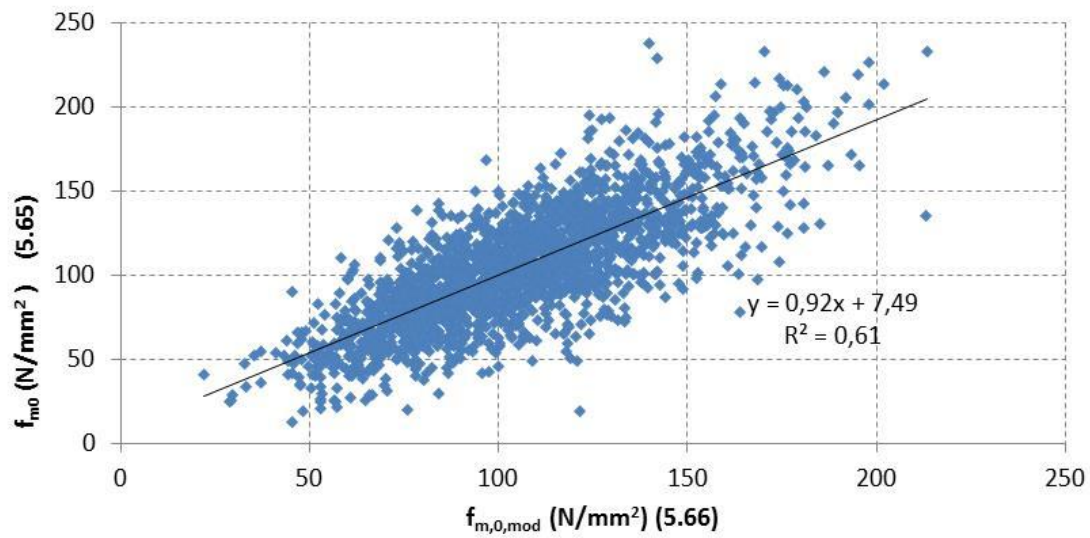


Figure 5.32. Simulated observed data ($n=2500$) plotted against simulated model data for simulation 2.

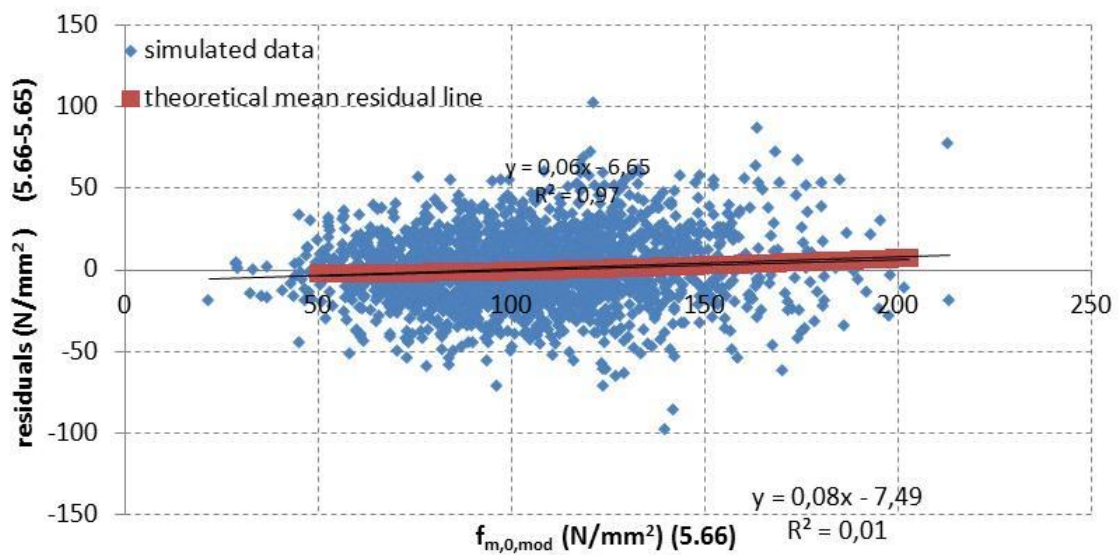


Figure 5.33. Simulated observed residuals ($n=2500$) plotted against simulated model data with the theoretical mean residual line for simulation 2.

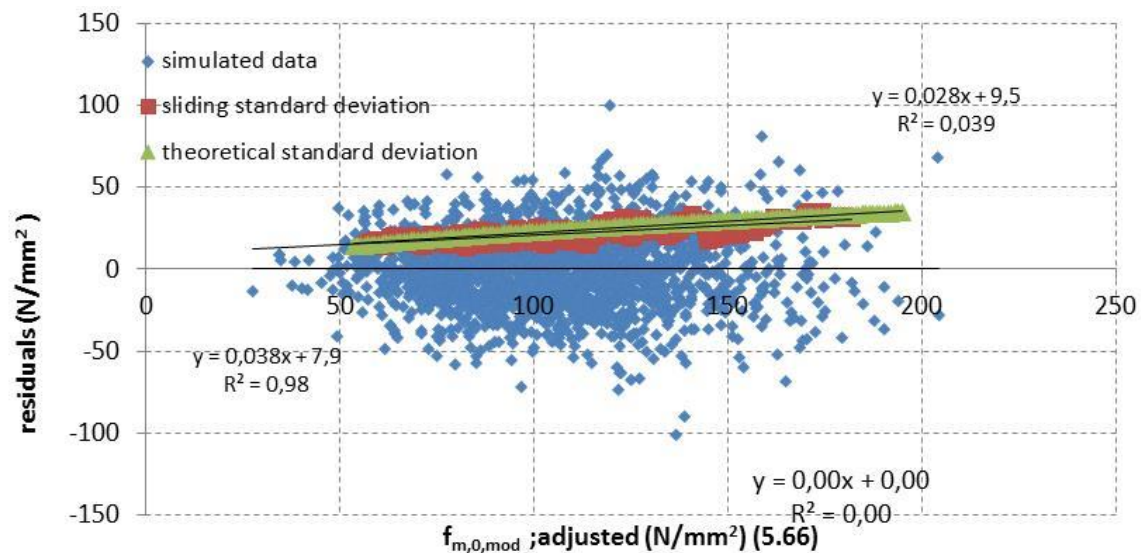


Figure 5.34. Simulated adjusted observed residuals($n=2500$) plotted against simulated model data with theoretically calculated standard deviation in every point and the sliding standard deviation line for simulation 2.

From the simulations, the following conclusions can be drawn:

- The trend of the standard deviation of the standard error around the model line very much depends on the input distribution of the density and the magnitudes of the constants.
- The theoretical trend of the standard deviations very well coincides with the sliding standard deviation. Because for timber with grain angle deviation and knots the theoretical equations become more complex, they will not be derived for these situations, but the sliding standard deviation will be used to evaluate the trend of the standard deviation of the error.

5.2.3 Prediction model for the bending strength based on the MOE and density for timber with grain angle deviation

In section 5.2.2 it was investigated how the observed scatter between model values based on the MOE could be explained for clear wood. Moving to structural timber at gross level, strength and stiffness reducing characteristics such as grain angle deviation and knots occur.

Since these characteristics have the effect of reducing the strength and stiffness, this will also have an effect on the observed scatter between model values and observed values.

Because the equations to describe the failure strengths are different for timber containing knots and containing grain angle deviation, this will be investigated for both situations.

From the test results in chapter 4 it was already found that visual grading is not suited for species independent grading. Therefore, the focus will be on models for machine grading. In this section, the following aspects will be investigated:

- What is the correct model to predict the bending strength based on the density and the MOE for structural timber with grain angle deviation?
- What scatter can be expected around this prediction model due to the natural variability for the bending strength and the MOE for a certain density and the occurrence of grain angle deviation?

In this section, structural timber is considered with grain angle but without the presence of knots. This frequently appears in structural timber from tropical hardwoods. It is assumed that no knots are present.

The grain angle deviation or slope of grain was defined in chapter 3. In figure 5.35 the typical failure mode of a piece of timber with grain angle deviation α for a piece of timber of wood species cumaru is given. In section 5.1.2 the influence of the grain angle deviation on the bending strength and MOE was shown. The influence described by the Hankinsons equations is the 3D grain angle deviation. In chapter 4 it was concluded that this was very difficult to measure visually. However, in the MOE measurements this 3D - effect will be integrated.

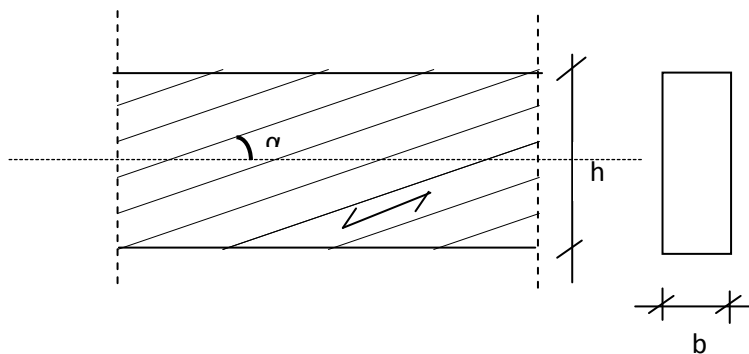


Figure 5.35. (above) Schematizing of grain angle deviation. (below) Example of grain angle deviation α and failure mode in a cumaru beam tested in a four-point bending test.

Reference is made to equations (5.32) and (5.33) for the relationship between the density with the bending strength of clear wood and the relationship between the density and the MOE of clear wood. The constants C_{k1} and C_{k2} are assumed to be zero:

$$f_{m,0} = C_1 \rho \quad (5.84)$$

$$MOE_0 = C_2 \rho \quad (5.85)$$

To make use of the Hankinson relation, the ratios for strength and MOE between parallel (with grain angle 0) and perpendicular to the grain (with grain angle 90) are defined as:

$$C_3 = \frac{f_{m,0}}{f_{m,90}} \quad (5.86)$$

$$C_4 = \frac{MOE_0}{MOE_{90}} \quad (5.87)$$

For the relation between $f_{m,0}$ and $f_{m,90}$ and the relation between MOE_0 and MOE_{90} the Hankinson relation with a value of $n=2$ is assumed. The bending strength for timber with a grain angle deviation α then becomes:

$$f_{m,\alpha} = \frac{f_{m,0}}{C_3 \sin^2(\alpha) + \cos^2(\alpha)} \quad (5.88)$$

This can be rewritten as:

$$f_{m,\alpha} = \frac{f_{m,0}}{(C_3-1)\sin^2(\alpha)+1} \quad (5.89)$$

The formula of $f_{m,\alpha}$ is given for a certain dimension h_{ref} (depth). The influence of depth will be studied in the next section.

For the Modulus of Elasticity the formula becomes:

$$MOE_\alpha = \frac{MOE_0}{(C_4-1)\sin^2(\alpha)+1} \quad (5.90)$$

When equation (5.84) is inserted in equation (5.88) and equation (5.85) is inserted in equation (5.90) then equations (5.85) and (5.90) become:

$$f_{m,\alpha} = \frac{\rho C_1}{(C_3-1)\sin^2(\alpha)+1} + C_5 \quad (5.91)$$

$$MOE_\alpha = \frac{\rho C_2}{(C_4-1)\sin^2(\alpha)+1} \quad (5.92)$$

The factor C_5 brings effects into account caused by for instance load configurations.

Evaluating formulas (5.91) and (5.92) it can be concluded that for timber without knots both the bending strength and the modulus of elasticity depend on the density, the grain angle, and the ratios of the bending strength and modulus of elasticity between parallel and perpendicular to the grain. The density and grain angle deviation can be regarded as two independent uncorrelated variables.

In chapter 4 it was already observed that grain angle deviation is difficult to measure, but it was shown in section 5.1 that grain angle deviation has an effect on both the *MOE* and the bending strength. Because in practice the *MOE* can be measured accurately by means of the *MOE_{dyn}*, by measuring the *MOE* indirect information about the grain angle deviation is gathered, which can be used to predict the bending strength.

In practice, the measured *MOE* is in fact *MOE_α*, because the value of the *MOE* is influenced by the (possible) varying value of α . Based on a non-destructive measurement of *MOE_α* an average of the grain deviation α can be estimated. When *MOE_α* is measured, equation (5.92) can be rewritten to (5.93) to express the square of sine α in the measured density ρ and *MOE_α*:

$$\sin^2(\alpha) = \left(\frac{\rho C_2}{MOE_\alpha} - 1 \right) \frac{1}{C_4 - 1} \quad (5.93)$$

Now the formula for $\sin^2(\alpha)$ according to equation (5.93) can be inserted in equation (5.91):

$$f_{m,\alpha} = \frac{\rho C_1}{(C_3 - 1) \left(\frac{\rho C_2}{MOE_\alpha} - 1 \right) \frac{1}{C_4 - 1} + 1} + C_5 \quad (5.94)$$

After rewriting this gives:

$$f_{m,\alpha} = \frac{\rho C_1 (C_4 - 1) MOE_\alpha}{(C_3 - 1) C_2 \rho + (C_4 - C_3) MOE_\alpha} + C_5 \quad (5.95)$$

Based on the measured density and *MOE*, the bending strength can be predicted without measuring the grain angle. In practice, this prediction will be more accurate than the equation with grain angle, since the grain angle has to be measured visually and is therefore more sensitive to measuring errors. This will show in the correlation coefficient and the magnitude of the error term of the model.

(5.94) can be rewritten as:

$$f_{m,\alpha,mod} = \frac{\rho MOE_\alpha}{D_1 \rho + D_2 MOE_\alpha} + D_3 \quad (5.96)$$

With

$$D_1 = \frac{(C_3 - 1) C_2}{C_1 (C_4 - 1)} \quad (5.97)$$

$$D_2 = \frac{(C_4 - C_3)}{C_1 (C_4 - 1)} \quad (5.98)$$

$$D_3 = C_5 \quad (5.99)$$

D_1 and D_2 can be calculated from the constants C_i when they are known. The constants C_i can be determined based on measurements of the grain angle and density and test results for the bending strength and *MOE*. If only density and *MOE* measurements are available, D_1 , D_2 and D_3 can directly be obtained by a regression analysis.

fm, α could also be obtained by a regression of the form $fm, \alpha = A * MOE + B$, which can be described as a pure curve fitting regression model. Equation (5.96) can be regarded as a (simplified) physical based model, where the values of the constants are derived by a regression analysis based on relevant physical parameters as input. Equation (5.96) is therefore the “correct” model, although this does not mean that the pure curve fitting regression model will automatically give worse predictions within certain limits. With the “correct” model, however, the shape of the scatter around the prediction can be estimated and the model is less sensitive.

Estimation of the scatter of the prediction model for the bending strength based on MOE and density of structural timber with grain angle deviation.

The next step is to include the errors due to the natural variation of the errors for the MOE and the bending strength based on the density and the correlation between them.

Equations (5.90) and (5.93) can be rewritten as:

$$MOE_{\alpha} = \frac{\rho C_2 + \varepsilon_M}{(C_4 - 1) \sin^2(\alpha) + 1} \quad (5.100)$$

$$\sin^2(\alpha) = \left(\frac{\rho C_2 + \varepsilon_M}{MOE_{\alpha}} - 1 \right) \frac{1}{C_4 - 1} \quad (5.101)$$

Inserting (5.101) in equation (5.91) and including the error ε_f :

$$f_{m, \alpha} = \frac{\rho C_1 + \varepsilon_f}{(C_3 - 1) \left(\frac{\rho C_2 + \varepsilon_M}{MOE_{\alpha}} - 1 \right) \frac{1}{C_4 - 1} + 1} \quad (5.102)$$

Equation (5.102) can be rewritten as:

$$f_{m, \alpha} = \frac{(\rho * C_1 + \varepsilon_f) MOE_{\alpha} (C_4 - 1)}{\rho (C_3 C_2 - C_2) + MOE_{\alpha} (C_4 - C_3) + \varepsilon_M (C_3 - 1)} \quad (5.103)$$

Inserting (5.57) and (5.58) in (5.103) gives:

$$f_{m, \alpha} = \frac{(\rho C_1 + r X_1 v_F \rho C_1 + X_2 v_F \rho C_1 \sqrt{(1 - r^2)}) MOE_{\alpha} (C_4 - 1)}{\rho (C_3 C_2 - C_2) + MOE_{\alpha} (C_4 - C_3) + X_1 v_M \rho C_2 (C_3 - 1)} \quad (5.104)$$

In section 5.2.2 it was found that the error term X_1 is responsible for the deviation of the model value with the mean least squares estimation line and the error term X_2 is responsible for the scatter around this line.

The error terms including X_2 from (5.104) are:

$$\varepsilon_{X_2} = \frac{(X_2 v_F \rho C_1 \sqrt{(1 - r^2)}) MOE_{\alpha} (C_4 - 1)}{\rho (C_3 C_2 - C_2) + MOE_{\alpha} (C_4 - C_3) + X_1 v_M \rho C_2 (C_3 - 1)} \quad (5.105)$$

MOE_{α} can be written as:

$$MOE_{\alpha} = \frac{\rho C_2 + X_1 v_M \rho C_2}{(C_4 - 1) \sin^2(\alpha) + 1} \quad (5.106)$$

Inserting (5.106) in (5.105):

$$\varepsilon_{X_2} = \frac{\left(X_2 v_F \rho C_1 \sqrt{(1-r^2)} \right) \frac{\rho C_2 + X_1 v_M \rho C_2}{(C_4-1) \sin^2(\alpha) + 1} (C_4-1)}{\rho (C_3 C_2 - C_2) + \frac{\rho C_2 + X_1 v_M \rho C_2}{(C_4-1) \sin^2(\alpha) + 1} (C_4 - C_3) + X_1 + X_1 v_M \rho C_2 (C_3 - 1)} \quad (5.107)$$

Ignoring the parts with X_1 (5.107) becomes:

$$\varepsilon_{X_2} = \frac{X_2 v_F \rho C_1 \sqrt{(1-r^2)}}{(C_3-1) \sin^2(\alpha) + 1} \quad (5.108)$$

And with the formulas for error propagation the standard error s_y becomes:

$$s_y = \frac{v_F \rho C_1 \sqrt{(1-r^2)}}{(C_3-1) \sin^2(\alpha) + 1} \quad (5.109)$$

Equation (5.109) shows that the error around the model line increases with increasing density, but decreases with increasing grain angle deviation. It can be seen that with grain angle deviation of zero, the standard error is the same as for clear wood, as expected. For a range of densities the errors will overlap.

The deviation from the model line (5.96) with the theoretical constants C_i compared with the mean least squares estimation model line is caused by the distribution of the densities in the dataset. This deviation for every model value from the model line and the standard deviation of the model standard error in every model value can also theoretically be derived as was done for clear wood in section 5.2.2. The principle is the same as for clear wood, however, the formulas for timber with grain angle deviation are more complicated.

The objective of this section is to find out the shape of the scatter that will be found in a least squares estimation on real data. In this thesis, the finding of section 5.2.2 will be used, namely that the sliding standard deviation of the residuals is a good estimation of the trend of the scatter. With equation (5.108) this trend can be compared.

The effect of the deviation of the theoretical model line (5.96) with a least squares estimation will be a transformation of the model line with the shape $y = A f_{mod,\alpha} + B$

The adjusted model values then will be:

$$f_{m,\alpha,mod,adj} = f_{m,\alpha,mod} A + B \quad (5.110)$$

Or, when equation (5.96) is adapted:

$$f_{m,\alpha,mod,adj} = \frac{\rho MOE_{\alpha}}{D_{1a}\rho + D_{2a}MOE_{\alpha}} + D_{3a} \quad (5.111)$$

With

$$D_{1a} = \frac{D_1}{A} \quad (5.112)$$

$$D_{2a} = \frac{D_2}{A} \quad (5.113)$$

$$D_{3a} = A * D_3 + B \quad (5.114)$$

5.2.4 Prediction model for the bending strength based on the MOE and density for timber with knots

To model the influence of knots again it is assumed that for clear wood there is linear relationship of the bending strength and stiffness of clear wood with the density. See equations (5.84) and (5.85). As for timber with grain angle deviation also for timber with knots the density and MOE are the measurement parameters for machine grading. It is now assumed that the grain angle deviation is zero.

In this section will be investigated:

- What is the correct model to predict the bending strength based on the density and the MOE for structural timber with knots ?
- What scatter can be expected around this prediction model due to the natural variability of the bending strength and *MOE* for a certain density and the occurrence of knots ?

The influence of the knots is characterized by a parameter, which in this thesis called KR, the knot ratio as defined in chapter 3. In figure 5.36 an example is given for the failure due to the presence of a knot in structural timber:

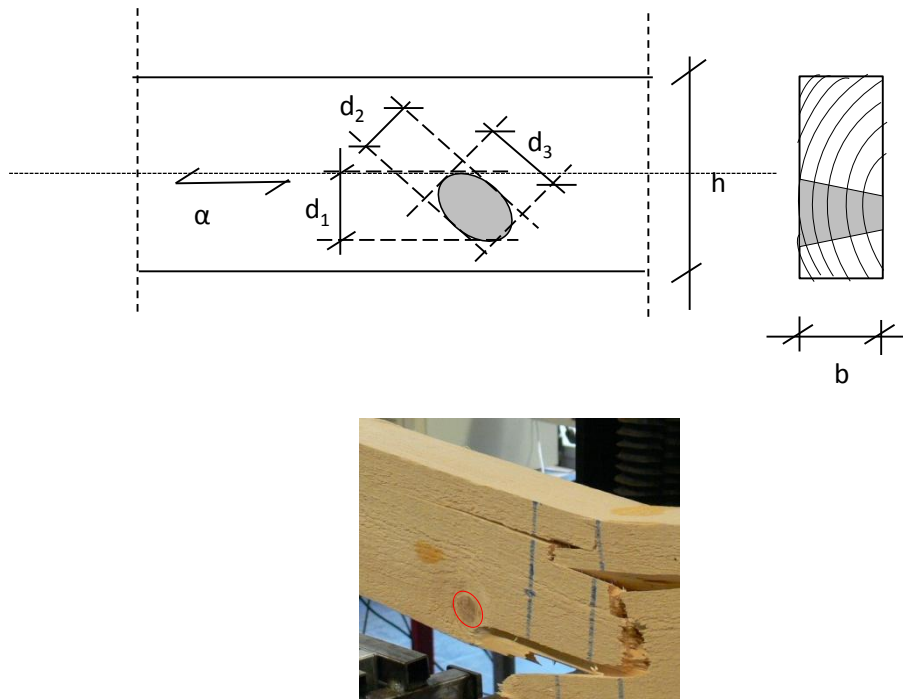


Figure 5.36 Schematizing of a knot (above). Example of a knot in a timber beam and the associated failure pattern it causes in a bending test (below) according to figure 3.1.

In section 5.1.3 it was described that the observed linear reduction of strength and stiffness (see figures 5.20 and 5.21) can be explained from theoretical considerations.

The bending strength due to the reduction of knots can then be described by equation (5.115):

$$f_{KR} = f_{m,0} * (1 - C_6KR) + C_7 \quad (5.115)$$

Factor C_7 brings the systematic effects into account that can be caused by for instance the load configuration or size effects. The term $(1 - C_6KR)$ indicates a linear reduction of the bending strength with increasing KR -value. The parameter KR gives a 2D- representation of an actual 3D-effect. However, in the measurements of the MOE the 3D-effect will be influencing the results. The purpose of the 2D-parameter KR is in this section only to give a physical basis for the prediction formulas based on machine measurements. However, research has shown that also the 3D-parameter KAR shows a linear relationship with the bending strength and the MOE .

Inserting equation (5.84) in equation (5.115) gives:

$$f_{KR} = \rho C_1(1 - C_6KR) + C_7 \quad (5.116)$$

The density and KR are two independent uncorrelated variables.

The MOE due to the reduction of knots can be described by equation (5.117):

$$MOE_{KR} = MOE_0(1 - C_8KR) \quad (5.117)$$

By inserting equation (5.85) in equation (5.117) gives:

$$MOE_{KR} = \rho C_2(1 - C_8KR) \quad (5.118)$$

KR can be expressed in MOE_{KR} :

$$KR = \frac{\rho C_2 - MOE_{KR}}{\rho C_2 C_8} \quad (5.119)$$

Inserting (5.119) into (5.116) for the bending strength gives:

$$f_{KR} = \rho C_1 \left(1 - C_6 \frac{\rho C_2 - MOE_{KR}}{\rho C_2 C_8} \right) + C_7 \quad (5.120)$$

This can be rewritten as:

$$f_{KR} = \rho C_1 \left(1 - \frac{C_6}{C_8} \right) + \frac{C_1 C_6}{C_2 C_8} MOE_{KR} + C_7 \quad (5.121)$$

(5.121) can be rewritten as:

$$f_{KR,mod} = D_4 \rho + D_5 MOE_{KR} + D_6 \quad (5.122)$$

With

$$D_4 = C_1 \left(1 - \frac{C_6}{C_8} \right) \quad (5.123)$$

$$D_5 = \frac{C_1 C_6}{C_2 C_8} \quad (5.124)$$

$$D_6 = C_7 \quad (5.125)$$

The factors D_i can be estimated from the C_i factors when KR values and density values are available, but can also be derived directly from a least squares regression with density and MOE as predicting parameters.

Equation (5.122) describes a linear relation for $f_{KR,mod}$ with the density and the MOE , whereas $f_{m,a,mod}$ in equation (5.96), describes a non-linear relation with the density and MOE . The “correct” prediction models for timber with grain deviation or timber with knots describe a different relation with the density and MOE .

In the next section, the scatter around the model line is investigated for timber with knots.

Modelling the variation in the model for the bending strength based on the presence of knots with density and MOE as predicting parameters.

Introducing ε_f and ε_M in equations (5.116) and (5.118)

$$f_{KR} = (\rho C_1 + \varepsilon_f)(1 - C_6KR) + C_7 \quad (5.126)$$

$$MOE_{KR} = (\rho C_2 + \varepsilon_M)(1 - C_8KR) \quad (5.127)$$

KR can be expressed in MOE_{KR} :

$$KR = \frac{(\rho C_2 + \varepsilon_M) - MOE_{KR}}{(\rho C_2 + \varepsilon_M) C_8} \quad (5.128)$$

Inserting (5.127) in (5.125):

$$f_{KR} = (\rho C_1 + \varepsilon_f) \left(1 - C_6 \frac{(\rho C_2 + \varepsilon_M) - MOE_{KR}}{(\rho C_2 + \varepsilon_M) C_8} \right) \quad (5.129)$$

or

$$f_{KR} = (\rho C_1 + \varepsilon_f) \left(1 - \frac{C_6}{C_8} \left[1 - \frac{MOE_{KR}}{(\rho C_2 + \varepsilon_M)} \right] \right) \quad (5.130)$$

By applying equations (5.57) and (5.58) equation (5.130) becomes:

$$f_{KR} = \left(\rho C_1 + r X_1 v_F \rho C_1 + X_2 v_F \rho C_1 \sqrt{(1 - r^2)} \right) \cdot \left(1 - \frac{C_6}{C_8} \left[1 - \frac{MOE_{KR}}{(\rho C_2 + X_1 v_M \rho C_2)} \right] \right) \quad (5.131)$$

In section 5.2.2 it was found that the error term X_1 is responsible for the deviation of the model value with the least squares estimation line and the error term X_2 is responsible for the scatter around this line.

The error terms including X_2 from (5.130) are:

$$\varepsilon_{KR, X_2} = \left(X_2 v_F \rho C_1 \sqrt{(1 - r^2)} \right) \left(1 - \frac{C_6}{C_8} \left[1 - \frac{MOE_{KR}}{(\rho C_2 + X_1 v_M \rho C_2)} \right] \right) \quad (5.132)$$

MOE_{KR} can be written as:

$$MOE_{KR} = (\rho C_2 + X_1 v_M \rho C_2) (1 - C_8 KR) \quad (5.133)$$

The equation (5.132) becomes:

$$\varepsilon_{KR, X_2} = X_2 v_F \rho C_1 \sqrt{(1 - r^2)} (1 - C_6 KR) \quad (5.134)$$

With the formulas for error propagation, the standard error of the model line for a certain model value can be calculated:

$$s_{KR, y} = \left(v_F \rho C_1 \sqrt{(1 - r^2)} \right) (1 - C_6 KR) \quad (5.135)$$

Equation (5.135) shows that the standard deviation of the error around the model line increases with increasing density and reduces with increasing knot ratio.

Comparing the equations for the prediction models for structural timber containing grain angle deviation with those for structural timber containing knots, it can be concluded that different prediction models are derived from the physical failure models. Also the

standard deviation of the error around the model line differs. From this it can be concluded that species independent grading models based on *MOE* and density are possible, when structural timber is divided in two groups: timber containing grain angle deviation and no knots and timber with knots containing no grain angle deviation.

5.2.5 Prediction model for the bending strength based on the presence of both grain angle deviation and knots

For softwoods and temperate hardwoods a grain angle deviation of zero can normally be assumed. At present in tropical hardwoods the presence of knots is very rare. However, in some cases, batches with timber with knots come on the market. The expectation is that this will happen more often in the future. The difference with the presence of knots in softwoods is that in tropical hardwoods the knots appear in combination with grain angle deviation. Another aspect is that the knots in tropical hardwoods are less systematically distributed.

To model this for strength, equations (5.91) and (5.116) can be combined, since they are both reduction formulas:

$$f_{m,\alpha,KR} = \frac{\rho C_1(1-C_6KR)}{(C_3-1)\sin^2(\alpha)+1} + C_9 \quad (5.136)$$

To model the stiffness, equations (5.92) and (5.117) can be combined since they are also both reduction formulas:

$$MOE_{\alpha,KAR} = \frac{\rho C_2(1-C_8KR)}{(C_4-1)\sin^2(\alpha)+1} \quad (5.137)$$

Since both $f_{m,\alpha,KAR}$ and $MOE_{m,\alpha,KAR}$ depend on both knot ratio and grain angle deviation it is not possible to express $f_{m,\alpha,KAR}$ as a function of only density and measured MOE. This means that it is not possible to make a prediction model with only measured density and measured MOE that can predict both the influence of knots and of grain angle deviation. Therefore, separate strength models have to be made for timber with knots and timber with grain angle deviation. In practice, for tropical hardwood where knots are very limited present, a maximum knot ratio is given in the visual grading standards. In NEN 5493 this maximum knot ratio is limited to 0.2. To investigate the influence of knots on the model for grain angle deviations simulations can be performed. The strength can then be modeled by introducing the scatter for the strength and stiffness.

Equation (5.136) then becomes

$$f_{m,\alpha,KR} = \frac{(\rho C_1 + \varepsilon_f)(1-C_6KR)}{(C_3-1)\sin^2(\alpha)+1} + C_9 \quad (5.138)$$

Including equation (5.59) :

$$f_{m,\alpha,KR} = \frac{(\rho C_1 + r X_1 v_F \rho C_1 + X_2 v_F \rho C_1 \sqrt{(1-r^2)})(1-C_6 KR)}{(C_3-1)\sin^2(\alpha)+1} + C_9 \quad (5.139)$$

Equation (5.137) becomes:

$$MOE_{\alpha,KR} = \frac{(\rho C_2 + \varepsilon_M)(1-C_8 KR)}{(C_4-1)\sin^2(\alpha)+1} \quad (5.140)$$

Including equation (5.57):

$$MOE_{\alpha,KR} = \frac{(\rho C_2 + X_1 v_M \rho C_2)(1-C_8 KR)}{(C_4-1)\sin^2(\alpha)+1} \quad (5.141)$$

5.3 Adjustment factors for depth and moisture content on the bending strength of structural timber.

In chapter 2, the influence of size and moisture content on the bending strength of timber is described. It was noticed that the grading method could influence the outcome of the size effect. Another aspect is the influence of moisture content on the bending strength, especially for tropical hardwoods, since they are usually tested at a higher moisture content than the reference value of 12%. In section 5.2, the prediction models for machine grading are defined. With these models, the influence of size and moisture content on the bending strength can be investigated. The adjustments for density and the MOE are already determined in chapter 3.

To determine the adjustment factors for size and moisture content a testing program was performed with a tropical hardwood species (greenheart) and a softwood species (spruce). Because for spruce not enough data for high moisture contents was available, only the influence of size is studied for spruce.

Adjustment factor for size and moisture content on the bending strength of greenheart

In table 5.6, the test material used in this investigation is listed. To provide extra data, additional specimens were prepared, cut from the remaining test pieces of sample GR2.

All specimens were tested in a four-point bending test with a span of 18 times the depth. The MOE_{local} and MOE_{global} were not measured for all specimens. Because the MOE_{dyn} was measured for all specimens, the MOE_{dyn} is used in the model.

In table 5.7, the test results are listed. Because the focus is on the influence of the depth, samples GRA+GRB, and samples GRI + GRJ are combined. At testing, the values for the bending strength, MOE_{dyn} and density are given for the measured values with the moisture content.

Table 5.6. Moisture contents, sizes and number of specimens of the greenheart samples.

Sample ID	Mean m.c.(%)	Relation with samples of table 3.1.	thickness (mm)	Depth (mm)	n
GRA	15	Cut from GR2	20	20	30
GRB	15	Cut from GR2	40	20	14
GRC	15	Cut from GR2	20	40	14
GRD	15	GR2	50	110	24
GRE	13	GR4	30	75	54
GRF	25	GR2	50	110	43
GRG	25	GR3	70	160	29
GRH	25	GR4	30	75	54
GRI	42	Cut from GR2	20	20	60
GRJ	42	Cut from GR2	40	20	28
GRK	42	Cut from GR2	20	40	14

Table 5.7. Test results for the greenheart samples from table 5.6.

Sample ID	Mean m.c.(%)	n	Depth (mm)	Bending strength (N/mm ²)		E _{dyn} (N/mm ²)		density (kg/m ³)	
				mean	cov	mean	cov	mean	cov
GRA +GRB	15	44	20	179.2	0.14	28100	0.11	1070	0.05
GRC	15	14	40	142.6	0.12	27600	0.08	1050	0.04
GRD	15	24	110	97.6	0.30	25700	0.11	1010	0.06
GRE	13	54	75	100.1	0.34	26100	0.20	1000	0.05
GRI+GRJ	42	88	20	126.1	0.18	25500	0.10	1210	0.03
GRK	42	14	40	103.1	0.27	26000	0.09	1220	0.02
GRH	25	54	75	85.6	0.27	23400	0.11	1060	0.05
GRG	25	29	110	91.4	0.23	23500	0.16	1070	0.07
GR F	25	43	160	77.6	0.27	26200	0.11	1040	0.04

To investigate the influence of size and moisture content, the model according to equation (5.96) is used with the adjustment factors for size and moisture content included in the model:

$$f_{m,mod,mc,size} = \left[\frac{\rho_{12} MOE_{dyn,12}}{D_1 \rho_{12} + D_2 MOE_{dyn,12}} + D_3 \right] \left(1 - kb_{mc} \frac{mc-12}{13} \right) \left(\frac{150}{h} \right)^{k_h} \quad (5.142)$$

For moisture contents above 25% m.c. a value of 25 is applied.

The input values for density and MOE_{dyn} are adjusted to 12% moisture content according to equation (3.11) for density and equation (4.4) for the MOE_{dyn}

The following factors are found by performing a non-linear regression analysis:

- $D_1 = 144.8$
- $D_2 = 3.63$
- $D_3 = -22.4$
- $k_h = 0.26$
- $kb_{mc} = 0.15$

In figure 5.37, the regression plot between the measured bending test values for all specimens of greenheart against the model values calculated with equation (5.141) and the factor values as listed above are given.

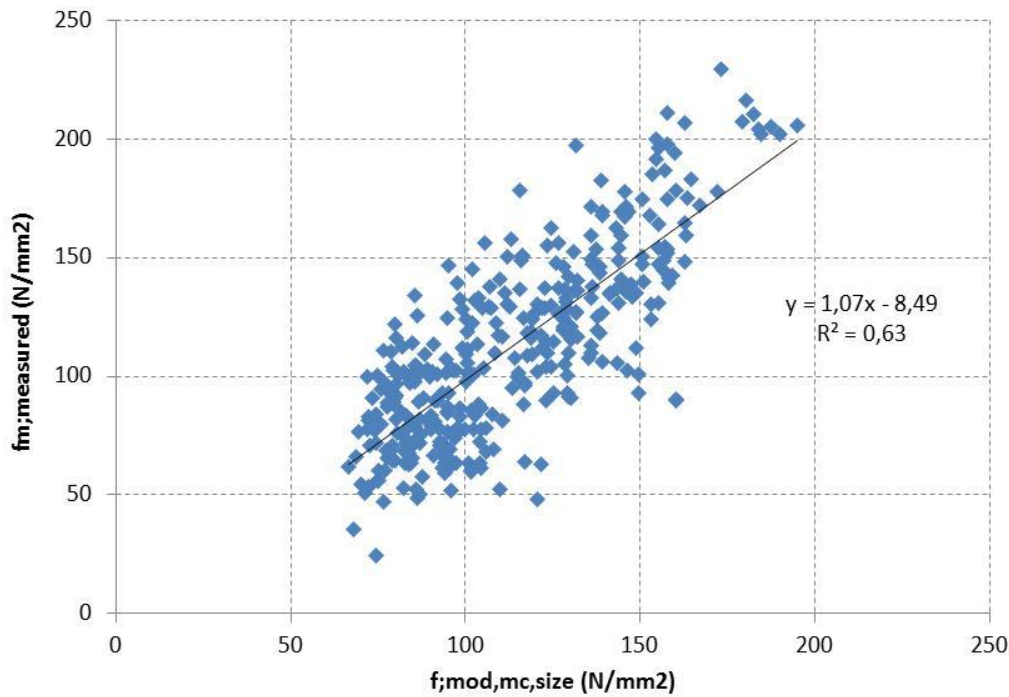


Figure 5.37. The measured bending test values for all greenheart specimens plotted against the model values calculated with equation (5.142)

Equation (5.142) describes how the model values, calculated with as input the density-values and MOE_{dyn} -values at 12% moisture content, have to be modified for size and moisture content to predict the bending strength values at the moisture content of the test. The factors found can also be used the other way around, to adjust the test values for the influence of size and moisture content, to obtain the bending strength at the reference moisture content of 12% and the reference height of 150 mm:

$$f_{m;mc12,h150} = \frac{f_{m;mc}}{\left[\left(1 - kb_{mc} \frac{mc-12}{13} \right) \left(\frac{150}{h} \right)^{k_h} \right]} \quad (5.143)$$

These values can be directly compared with the model values of equation (5.95), with the input parameters density and MOE_{dyn} also adjusted to 12% m.c.

A significant k_h - factor of 0.26 is found for the influence of depth. However, to investigate whether the size effect also appears for structural sizes the plots in figures 5.38 and 5.39 are constructed in the following way:

The test results for the bending strength were adjusted only for the influence of moisture content according to equation (5.144):

$$f_{m;mc12} = \frac{f_{m;mc}}{\left[\left(1 - 0.15 \frac{mc-12}{13} \right) \right]} \quad (5.144)$$

For moisture contents above 25%, for the m.c., a value of 25 is applied.

The model values were calculated with:

$$f_{m,model,12} = \left[\frac{\rho_{12} MOE_{dyn,12}}{144.8 \rho_{12} + 3.63 MOE_{dyn,12}} - 22.4 \right] \quad (5.145)$$

The input values for density and MOE_{dyn} are the values adjusted to 12% moisture content.

In figure 5.38, the mean adjusted bending strength for moisture content according to (5.144) for every depth is plotted against the depth. This graph shows that there is an influence of size and the power value of 0.28 is very close the found value of $k_h=0.26$, but that it dampens out for larger sizes. Sizes for structural use normally start at a depth of 70 mm, and when the datapoints for sizes from 75 mm and above are evaluated, also a horizontal regression line could be found.

In figure 5.39, the model values adjusted for moisture content according to equation (5.144) are plotted against the model values calculated with (5.145). The datapoints are clustered in sizes with depths of 20 mm, 40 mm and above 40 mm. The data clouds for depths of 20 mm and 40 mm can be clearly identified. Above 40 mm the datacloud is homogeneous.

Since in the database of table 3.1 the minimum depth is 75 mm the conclusion is that for temperate hardwoods in this thesis no depth effect will be applied to adjust the test data of the bending strength for size and only a moisture content correction will be applied.

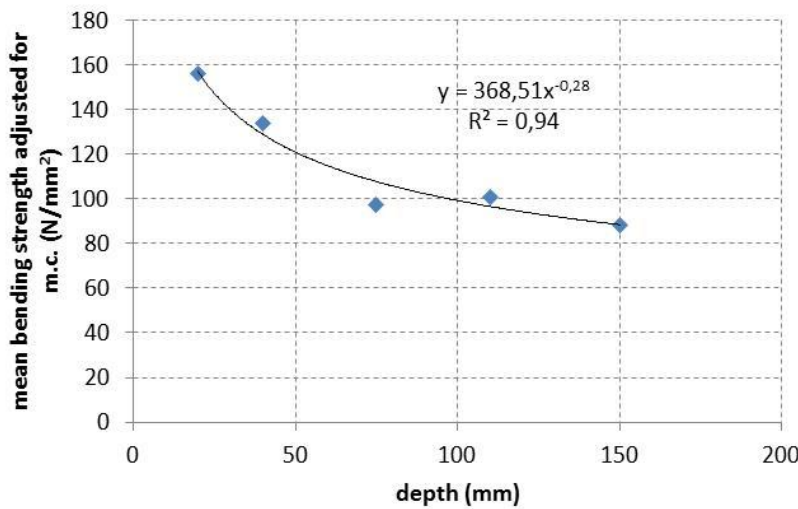


Figure 5.38. Mean bending strength adjusted for moisture content for every depth according to (5.144) plotted against the depth of the greenheart dataset

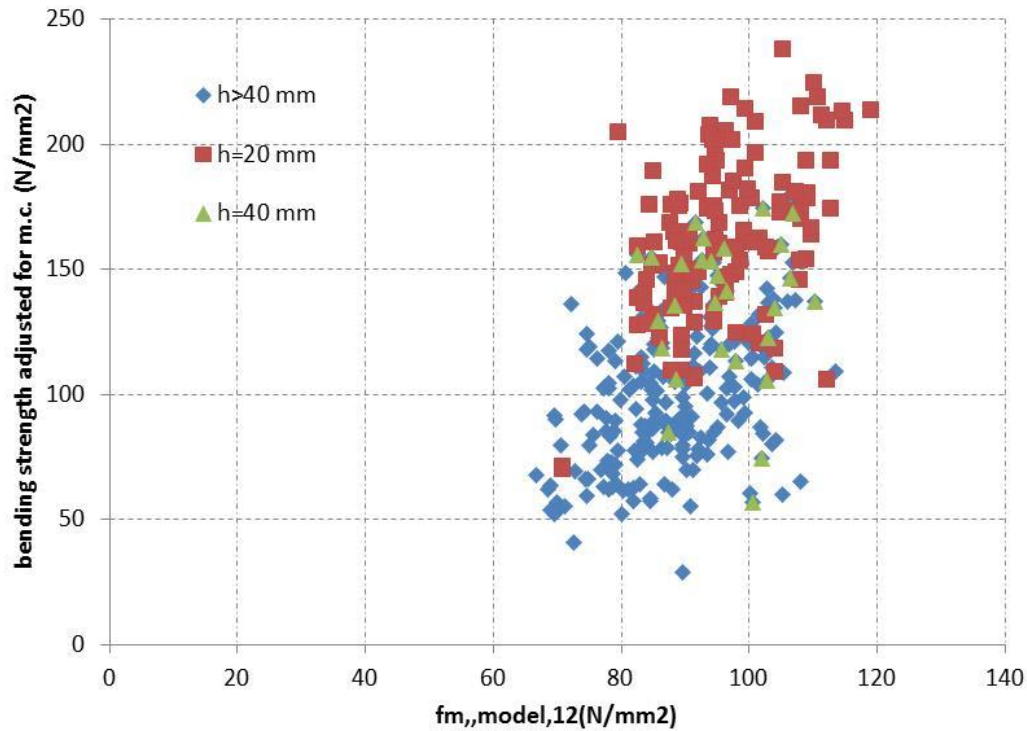


Figure 5.39. Bending strength values adjusted for moisture content according to (5.144) plotted against the model values according to (5.145) for greenheart clustered for depths of 20mm, 40 mm and above 40 mm.

Adjustment factor for size on the bending strength of spruce

Most softwoods species are tested and used at the reference moisture content of 12%. In chapter 2 it was found from literature that the influence of moisture content on the bending strength decreases for timber with increasing knot ratios, and that therefore in test standards no adjustment of the bending strength test values is applied. In this thesis, there is not sufficient data to investigate this in depth, so this principle will be applied for wood species for which the main failure mode for structural timber is due to the presence of knots. This means that for the temperate hardwoods species from table 3.3 and the softwood species from table 3.5 no adjustment for moisture content on the bending strength was applied.

In table 5.8, the test material used to investigate the influence of size on the bending strength for spruce is listed. To provide extra data, additional specimens were prepared, cut from remaining test pieces of sample S2.

All specimens were tested in a four-point bending test with a span of 18 times the depth. The MOE_{local} and MOE_{global} were not measured for all specimens. Because the MOE_{dyn} was measured for all specimen, the MOE_{dyn} is used in the model.

In table 5.9 the test results are listed. Because the focus is on the influence of the depth, samples SPA+SPB are combined. At testing the values for the bending strength, MOE_{dyn} and density are given for the measured values with the moisture content.

Table 5.8. Moisture contents, sizes and number of specimens of the spruce samples.

Sample ID	Mean m.c.(%)	Relation with samples of table 3.1.	thickness (mm)	Depth (mm)	<i>n</i>
SPA	12.4	Cut from SPD	20	20	40
SPB	12.4	Cut from SPD	40	20	40
SPC	12.4	Cut from SPD	20	40	40
SPD	11.3	From S2	60	150	30
SPE	12.4	From S2	50	100	24
SPF	15.3	From S2	145	285	50
SPG	12.6	From S3	50	150	77
SPH	13.7	From S3	70	173	80
SPI	19.8	From S3	95	245	37

Table 5.9. Test results for the spruce samples from table 5.8.

Sample ID	Mean m.c.(%)	<i>n</i>	Depth (mm)	Bending strength (N/mm ²)		<i>E_{dyn}</i> (N/mm ²)		density (kg/m ³)	
				mean	cov	mean	cov	mean	cov
SPA+SPB	12.4	80	20	69.1	0.19	11300	0.18	450	0.09
SPC	12.4	40	40	48.7	0.28	11300	0.20	450	0.09
SPD	11.3	30	150	50.8	0.21	15100	0.10	490	0.08
SPE	12.4	24	100	50.8	0.16	15400	0.07	510	0.08
SPF	15.3	50	285	35.0	0.28	11800	0.15	450	0.08
SPG	12.6	77	150	43.4	0.34	13500	0.19	470	0.09
SPH	13.7	80	173	35.2	0.32	11700	0.21	440	0.11
SPI	19.8	37	245	38.4	0.20	13000	0.15	490	0.07

To investigate the influence of size, the model according to equation (5.122) is used with the adjustment factor for size in the model:

$$f_{mod,size} = [D_4\rho_{12} + D_5E_{dyn,12} + D_6] \left(\frac{150}{h}\right)^{k_h} \quad (5.146)$$

The input values for density and MOE_{dyn} are adjusted to 12% moisture content, the bending test is unadjusted for moisture content.

The following values for the factors are found in a linear least squares regression:

- $D_4 = -0.0071$
- $D_5 = 0.00304$
- $D_6 = 4.94$
- $k_h = 0.30$

In figure 5.40, the regression plot between the measured bending test values for all specimens of spruce against the model values calculated with equation (5.146) and the factor values as listed above are given.

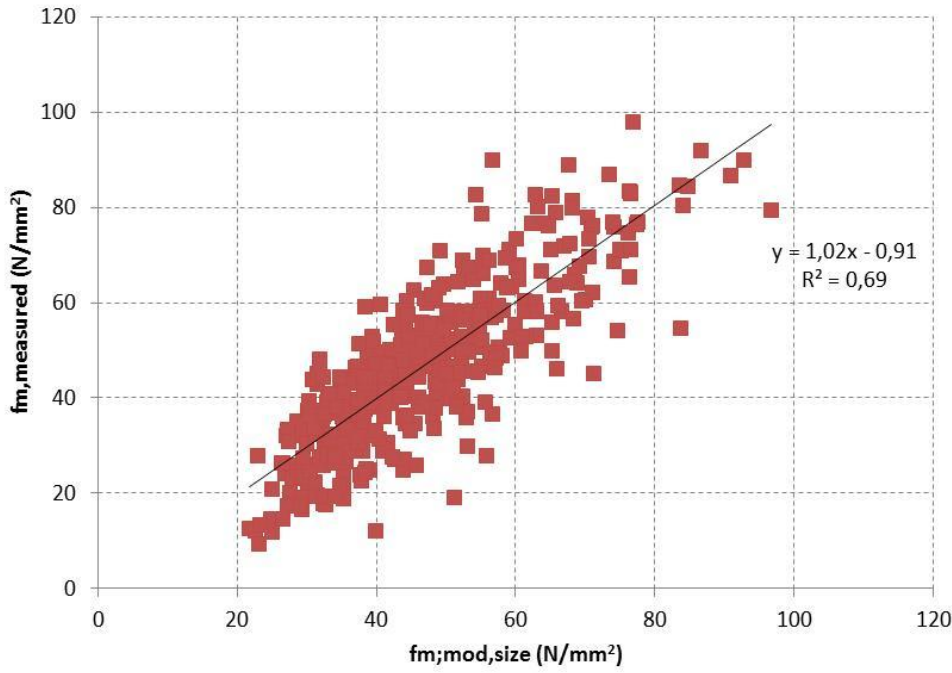


Figure 5.40. The measured bending test values for all spruce specimens plotted against the model values calculated with equation (5.146)

As for greenheart, a significant depth factor of $k_h=0.3$ is found for spruce .

To investigate whether this depth factor also exists for structural sizes, the unadjusted bending strength values will be plotted against the model values without including the effect of size.

The model values are calculated with:

$$f_{mod} = [-0.0071\rho_{12} + 0.00304E_{dyn,12} + 4.94] \quad (5.147)$$

The input values for density and MOE_{dyn} are adjusted to 12% moisture content.

In figure 5.41, the mean unadjusted bending strength for every depth is plotted against the depth. This graph shows that there is an influence of size and the power value of 0.22 is slightly lower than the value found of $k_h=0.30$, and that it dampens out for larger sizes. When the smallest depth of 20 mm is removed, then the regression line gives a power of 0.18 (dashed regression line in figure 5.41).

It could be argued whether there is a slight reduction trend for the larger sizes, but from table 5.9 it can be seen that the sizes $h=173$ mm and $h=285$ mm with the lowest mean bending strength, also have the lowest MOE_{dyn} . Therefore, this is probably not a size effect but a quality effect. This is confirmed by the plot in figure 5.42, where the unadjusted bending strength values for every piece are plotted against the model values without influence of size according to (5.146). The data clouds for $h=20$ mm and $h=40$ mm can be clearly distinguished; above $h=40$ mm a homogeneous cloud can be observed.

Since in the database of tables 3.3 and 3.5 the minimum depth is 75 mm, the conclusion is that in this thesis no depth effect will be taken into account to adjust the test data of the bending strength for size for temperate hardwoods and softwoods.

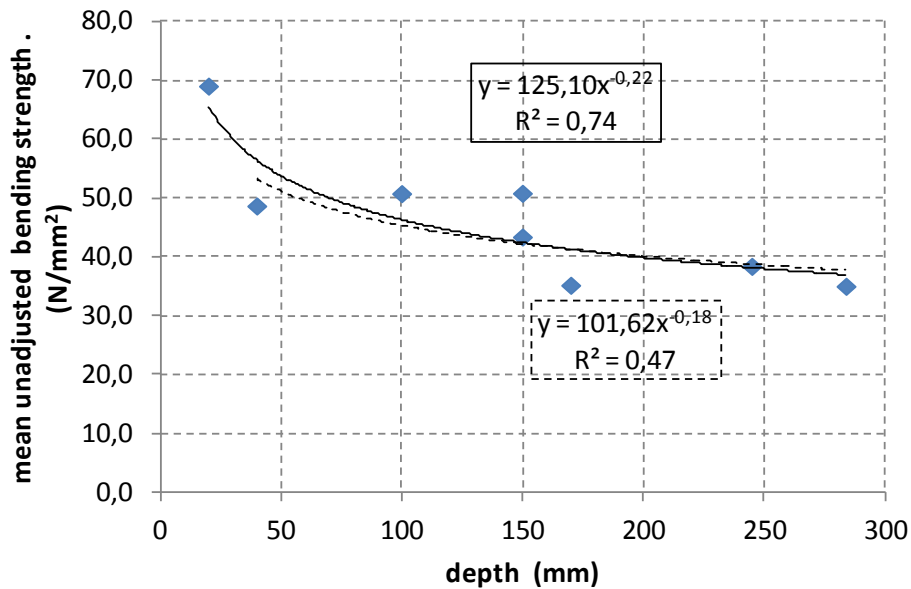


Figure 5.41. Mean unadjusted bending strength values plotted against the depth of the spruce dataset. Solid regression line for depths from 20 mm to 284 mm. Dashed regression line for depths of 40 mm to 284 mm.

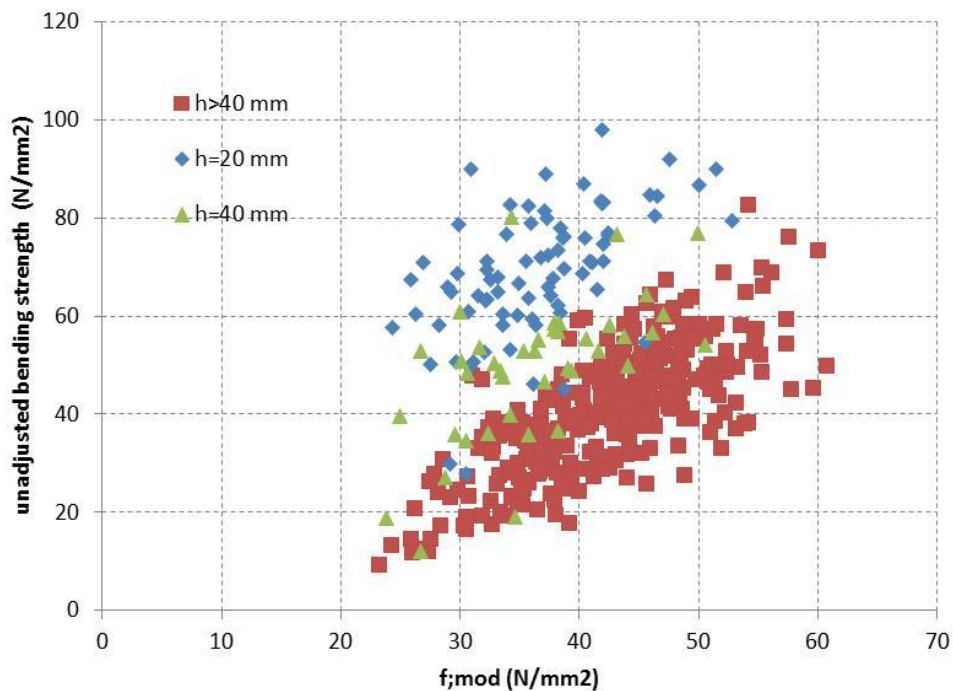


Figure 5.42. Unadjusted bending strength values) plotted against the model values according to (5.147) for spruce clustered for depths of 20 mm, 40 mm and above 40 mm.

5.4 Verification of the developed models for tropical hardwood and softwood.

5.4.1 Introduction

In sections 5.4.2 and 5.4.3 the theory applied in this chapter will be verified on a dataset of tropical hardwood and a dataset of softwood. Because for these datasets both quantified data for every piece for the strength reducing characteristics (grain angle deviation and knot ratio) as well as machine measurements (density and MOE_{dyn}) are available, the expected scatter around the regression line can be verified by direct least squares regression on the test data.

5.4.2 Verification for a dataset of tropical hardwood

For the verification of the theory for tropical hardwoods, a dataset of greenheart and massaranduba was used. The pieces from greenheart were from sample GR 4 and from massaranduba from sample MAS5. The prepared pieces used in section 5.1.2 with a cross section of 50 x 50 mm² were cut from remaining parts after testing from sample MAS5. They were included to cover a wider range of grain angle deviations and are called sample MAS5-C.

In table 5.10, the property values of the datasets are given. The bending strength was adjusted to 12% moisture content according to section 5.3. The density and MOE_{dyn} were adjusted to 12% m.c. according to chapter 3. Because the depth of the pieces in sample MAS5-C was below 75 mm, they were adjusted for size according to section 5.3. For the other two samples no adjustment for size was made.

Table 5.10. Material properties for the verification samples of tropical hardwoods

Sample ID	Slope of grain		Bending strength (N/mm ²)		MOE_{dyn} (N/mm ²)		Density (kg/m ³)		n
	mean	stdev	mean	stdev	mean	stdev	mean	stdev	
MAS5	0.05	0.02	128.9	23.8	27200	3700	1050	60	55
GR4	0.08	0.03	98.5	27.8	26400	2800	980	50	104
MAS5-C	0.21	0.10	61.7	34.0	18500	4800	1040	60	19

With a non-linear regression analysis using equations (5.91) and (5.92), the constants C_i ($i=1-5$) can be determined. Because the data of table 5.10 is unbalanced with respect to the distribution of the slope of grain data, only sample MAS5-C is used for this analysis. This dataset is balanced with respect to slope and grain, and also contains the most accurate measurements.

The results from the non-linear regression equations with equation (5.91) as model for the bending and equation (5.92) as model for MOE_{dyn} are:

- $C_1 = 0.13$
- $C_2 = 24.1$
- $C_3 = 28.2$
- $C_4 = 9.3$
- $C_5 = 8.5$

In analogy with equations (5.64a) and (5.64b), the values for $(X_1 v_F)$ and $(X_2 v_M)$ can now be calculated. The equations for timber with grain angle deviation become:

$$(X_1 v_F)_i = \frac{(f_{m,i} - C_5)[(C_3 - 1)\sin^2(\alpha) + 1] - \rho C_{1,i}}{\rho C_{1,i}} \quad (5.148)$$

$$(X_2 v_M)_i = \frac{MOE_i[(C_4 - 1)\sin^2(\alpha) + 1] - \rho C_{2,i}}{\rho C_{2,i}} \quad (5.149)$$

In these equations $f_{m,i}$ and MOE_i are the measured bending strength and MOE_{dyn} .

The $(X_1 v_F)_i$ values for sample MAS5-C are plotted against the $(X_2 v_M)_i$ values in figure 5.43.

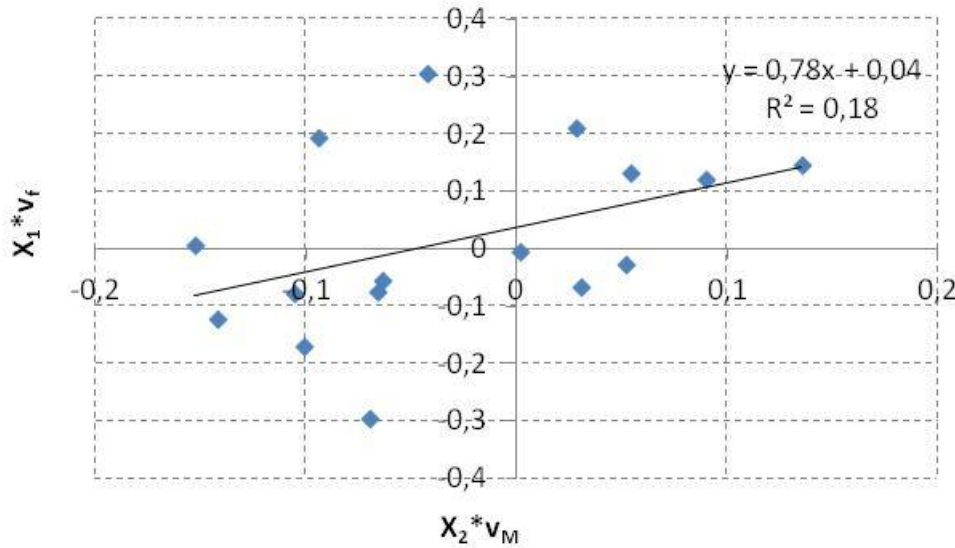


Figure 5.43. $(X_1 v_F)_i$ values are plotted against the $(X_2 v_M)_i$ values for the sample MAS5-C.

Figure 5.41 shows that $X_1 \cdot v_F$ and $X_2 \cdot v_M$ are correlated, the found coefficient of determination $r^2 = 0.18$. The mean values are close to zero. The values of v_F and v_M can be directly calculated from the standard deviation of $X_1 \cdot v_F$ and $X_2 \cdot v_M$. This gives $v_F = 0.15$ and $v_M = 0.09$.

With the derived C-values the theoretical D-factors can be calculated according to equations (5.97), (5.98) and (5.99):

$$D_1 = \frac{(28.2-1)24.1}{0.13(9.3-1)} = 597.0$$

$$D_2 = \frac{(9.3-28.2)}{0.13(9.3-1)} = -17.2$$

$$D_3 = -8.5$$

Now the model according to equation (5.96) is used with one adjustment:

As was noticed, the factor C_2 might not be constant but also stochastic. Simulations performed by the author show that this might have a significant influence on the stability of the model. This can be overcome by applying a maximum value for MOE_α . A maximum value of the density multiplied by 25 seems to be a safe factor and is kept in the equation. Applying the factor 25 multiplied by the density makes sure that the model gives safe predictions. Equation (5.96) is therefore adjusted to

$$f_{m,\alpha,mod} = \frac{\rho \min(MOE_\alpha; \rho 25)}{597.0 \rho \pm 17.2 \min(MOE_\alpha; \rho 25)} - 8.5 \quad (5.150)$$

In figure 5.44, the tested bending strengths are plotted against the model values of the all samples calculated with equation (5.150). In figure 5.45, the residuals are plotted against the model values.

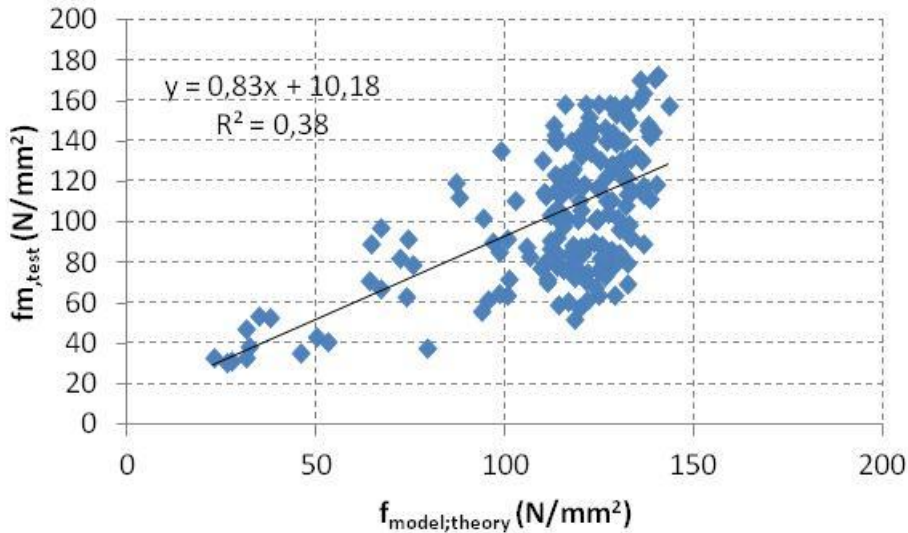


Figure 5.44. Bending test values plotted against model values calculated with (5.150)

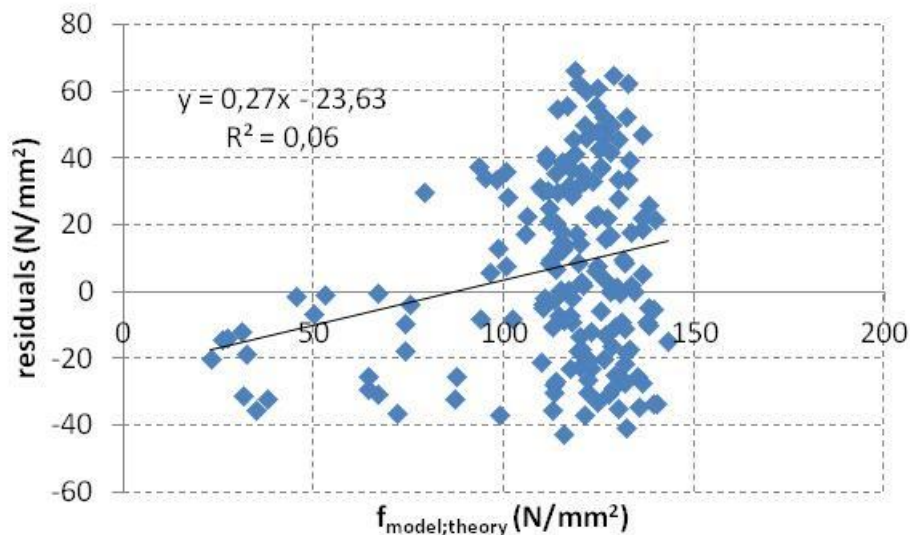


Figure 5.45. Residuals of the theoretical model plotted against theoretical model values. (5.150)

As expected, the regression line in figure 5.44 is not coinciding with $y=x$ and the residuals are not distributed around zero for all model values (figure 5.45).

The model values are now adjusted with the equation of figure 5.44.

That gives:

$$D_{1a}=719.2$$

$$D_{2a}= -20.7$$

$$D_{3a}=3.1$$

The regression plot with the adjusted model values can now be directly compared with a direct least squares regression on the data. In the least squares regression, the sum of the residuals is minimized and the D-factors are derived directly. The factor 25 times the density as a maximum for the MOE_{dyn} , is also applied.

The D-factors found from a least squares regression are:

$$D_1=376$$

$$D_2= -7.6$$

$$D_3=-18.3$$

The factors found in the direct least squares equation are different from the adjusted theoretical factors. This is probably due to the fact that the least squares regression is very sensitive to the input distributions.

The regression plots, however, are very similar. See figures 5.46 and 5.47. For both models the regression lines coincide with $y=x$ and figure 5.47 shows that the theoretical

scatter pattern of the residuals is similar to the scatter pattern of the residuals from a least squares equation.

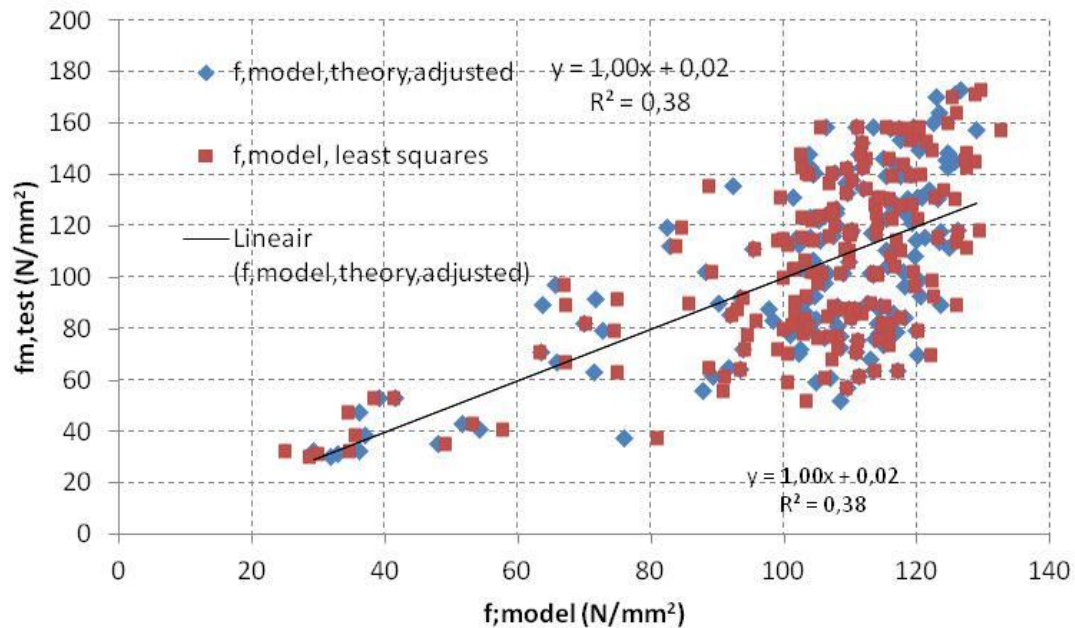


Figure 5.46. Bending test values plotted against theoretical and least squares model values.

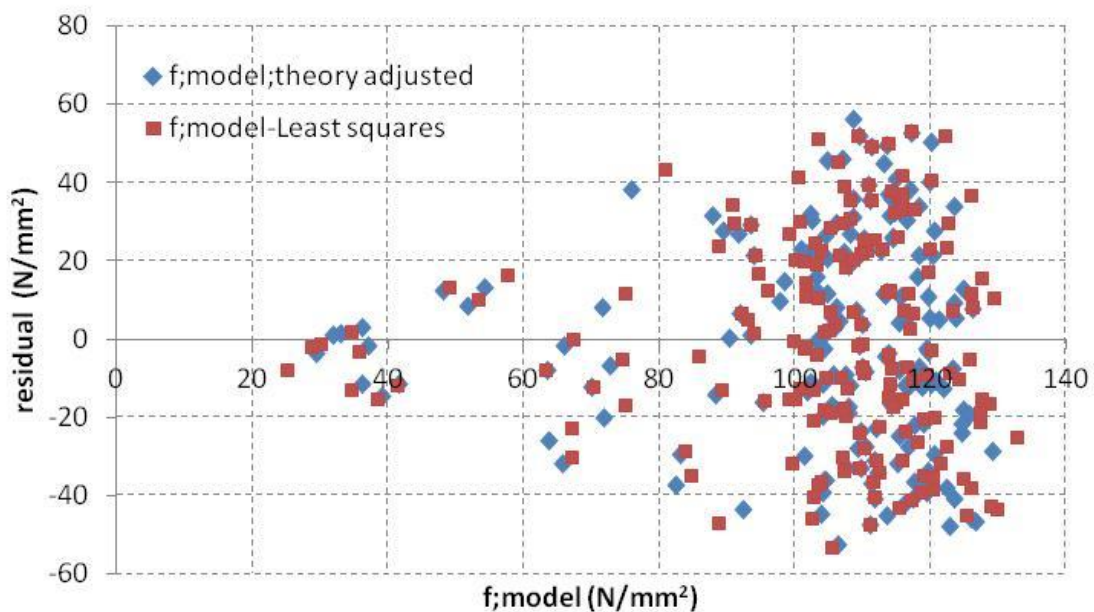


Figure 5.47. Residuals of the theoretical and least squares model plotted against theoretical and least squares model values.

Figure 5.47 shows that the standard deviation of the errors increases very strongly with increasing model values. This can be explained from equation (5.109). For this dataset,

there is not much variation in density, so the standard deviation of the error can be explained by the grain angle deviation.

Because of the small amount of data, the sliding standard deviation cannot be calculated. However, the shape of the trend of the residuals can be studied by performing simulations.

A dataset of 2500 pieces is simulated with the input for the properties and factors according to tables 5.11 and 5.12. In table 5.11, the distributions of the density, the slope of grain, and X_1 and X_2 are listed. The properties for every piece are randomly drawn from these distributions. Only X_1 and X_2 are correlated. In table 5.12 the values of the constants are listed.

Table 5.11. Input distributions for the simulations

Property	mean	Standard deviation	Distribution type
Density	1010	67	normal
Grain angle deviation	$\alpha=0.12$	$\beta=1.5$	weibull
X_1	0	1	normal
X_2	0	1	normal

Table 5.12. Values for constants used in the simulations

Property	
C_1	0.13
C_2	24.1
C_3	28.2
C_4	9.3
C_5	-8.5
v_F	0.15
v_M	0.09
r^2 between X_1 and X_2	0.18

Tables 5.11 and 5.12 are used as input for the simulations. However, it was found that especially for the greenheart sample, the constant C_2 was higher. Therefore, in the simulations, a value of 26 was used for C_2 . To calculate the D-values in the equations the value in table 5.12 was used.

In figures 5.48 to 5.52, the results of the simulations are given together with the test results. In figure 5.48, the bending strength is plotted against the slope of grain and in figure 5.49 the MOE_{dyn} against the slope of grain.

In figure 5.50 the bending strength is plotted against the MOE_{dyn} , which clearly shows that the relation between these properties is non-linear when pieces with large slope of grains are included. Figure 5.51 shows the bending strength plotted against the adjusted model values for the simulations and for the test data the bending strength plotted against model values obtained by a direct least squares regression on the data. Figure 5.52 shows the

residuals against the model values. Figure 5.52 shows that the residuals of the simulations have the same shape as the residuals of the test data. The sliding standard deviation of the residuals of the simulated data shows an increasing trend with increasing model values. This is in agreement with the developed theory.

It can be concluded that the trend and shape of the test data for the samples of greenheart and massaranduba are in line with what is expected according to the developed theory. The sliding standard deviation gives a good estimation of the trend of the standard error with increasing model values.

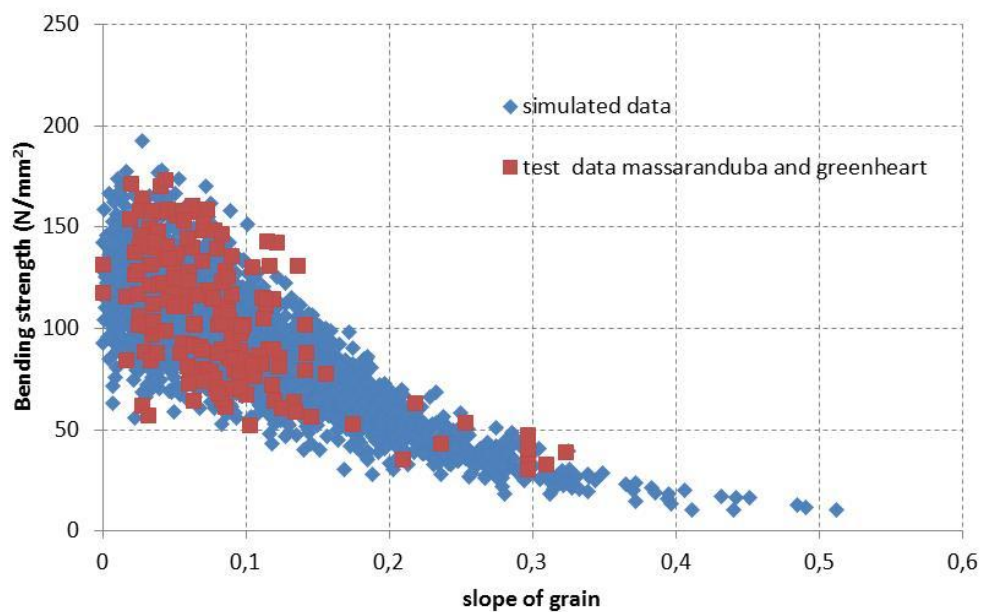


Figure 5.48. Bending strength plotted against the slope of grain for the test samples and the simulations.

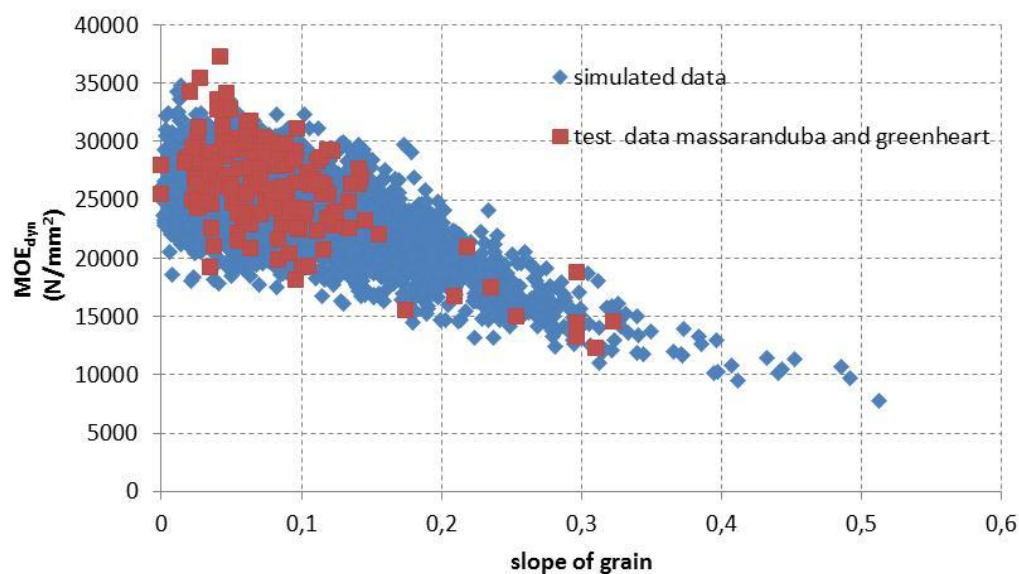


Figure 5.49. MOE_{dyn} plotted against the slope of grain for the test samples and the residuals.

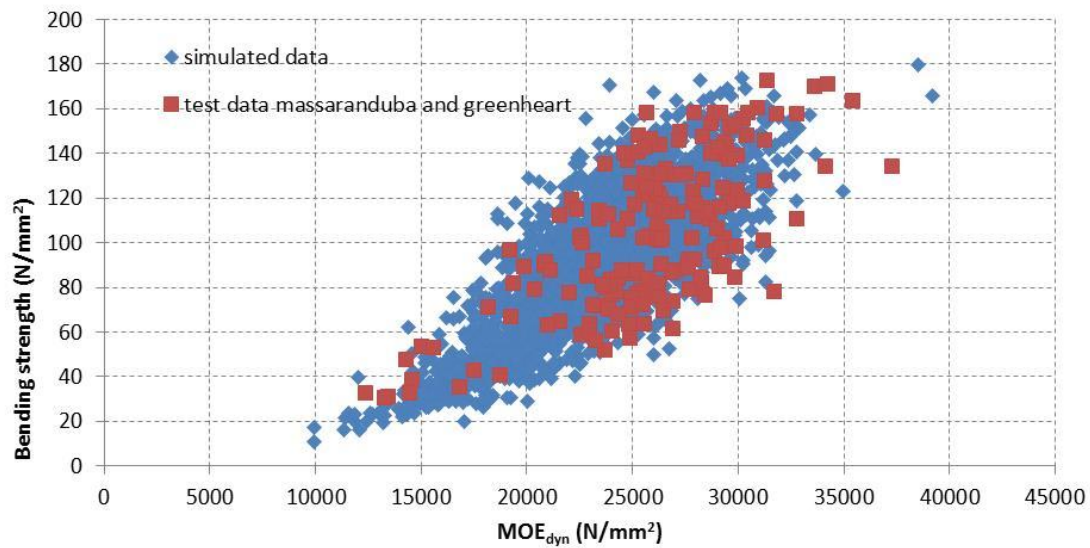


Figure 5.50. Bending strength plotted against MOE_{dyn} for the test samples and the residuals.

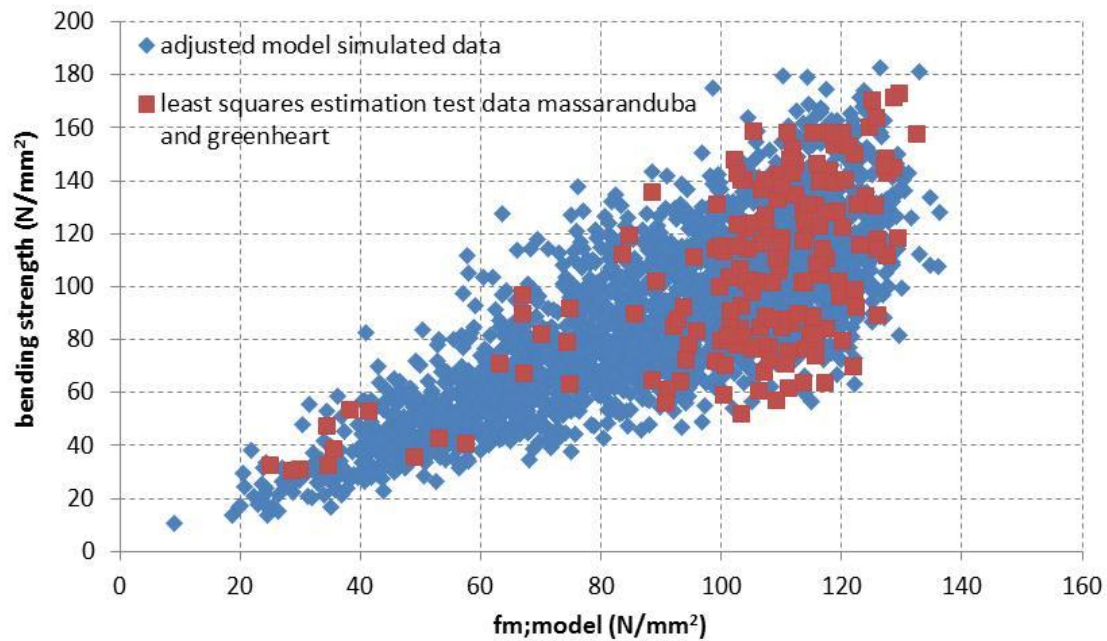


Figure 5.51. Bending strength plotted against the model values for the least squares estimation on the test samples and for the adjusted model on the simulated data.

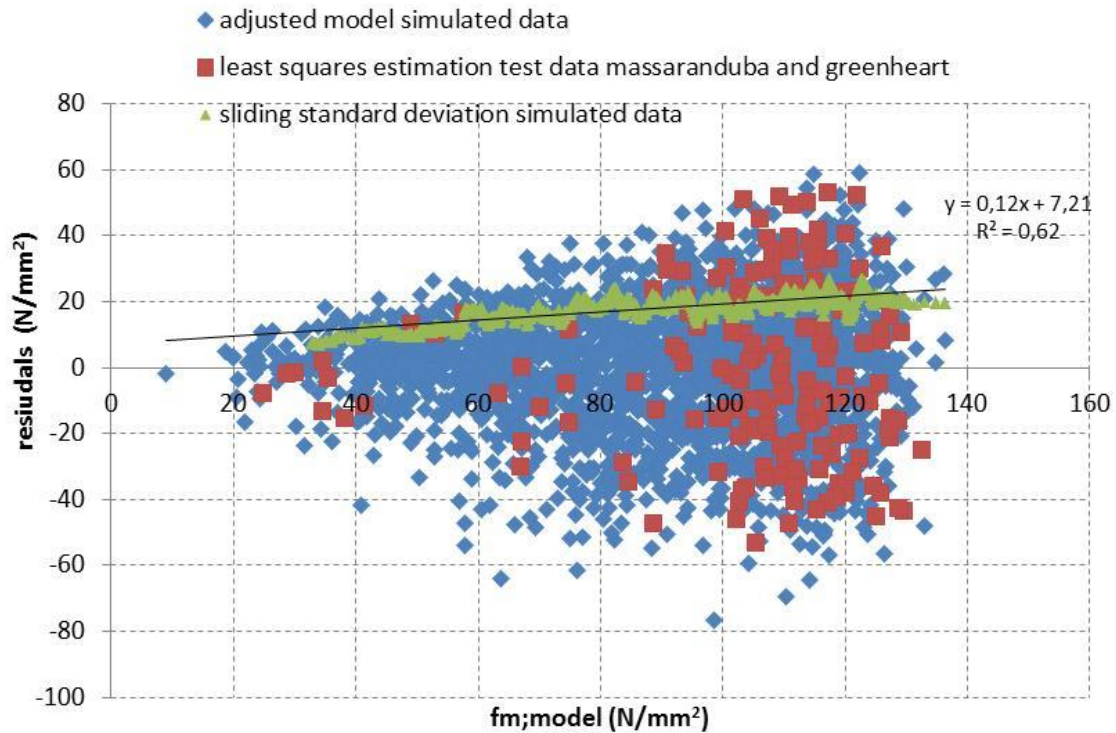


Figure 5.52. Residuals plotted against the model values for the least squares estimation on the test samples and for the adjusted model on the simulated data + the sliding standard deviation for the simulated data.

5.4.3 Verification for a dataset of softwood

For the verification of the theory for softwoods sample D1 of species douglas is used. In table 5.13, the property values of the dataset are given. According to section 5.3 no adjustment for size and moisture content for the bending strength was applied. The density and MOE_{dyn} were adjusted to 12% m.c. according to chapter 3. Only slight adjustments were necessary since the mean moisture content of the sample was 13.6 %. To describe the influence of knots, the group knot ratio was used according chapter 3.

Table 5.13. Material properties for the verification sample of douglas

Sample ID	Group knot ratio		Bending strength (N/mm ²)		MOE_{dyn} (N/mm ²)		Density (kg/m ³)		n
	mean	stdev	mean	stdev	mean	stdev	mean	stdev	
D1	0.24	0.12	48.6	17.0	14300	3100	570	60	356

With a non-linear regression analysis using equations (5.116) and (5.118), the constants C_i ($i=1,2,6,7,8$) are determined.

The results of the non-linear regression equations with equation (5.116) as model for the bending and equation (5.118) as model for MOE_{dyn} are:

- $C_1 = 0.11$
- $C_2 = 26.9$
- $C_6 = 1.03$
- $C_7 = -0.03$
- $C_8 = 0.37$

In analogy with equations (5.64a) and (5.64b), the values for $(X_1 v_F)$ and $(X_2 v_M)$ can now be calculated. The equations for timber with knots deviation become:

$$(X_1 v_F)_i = \frac{f_{m,i} - C_7}{\rho C_{1,i}(1 - C_6 GKR)} - 1 \quad (5.151)$$

$$(X_2 v_M)_i = \frac{MOE_i}{\rho C_{2,i}(1 - C_6 GKR)} - 1 \quad (5.152)$$

In these equations $f_{m,i}$ is the measured bending strength and MOE_i is the measured MOE_{dyn} .

The $(X_1 v_F)_i$ values for sample D1 are plotted against the $(X_2 v_M)_i$ values in figure 5.53.

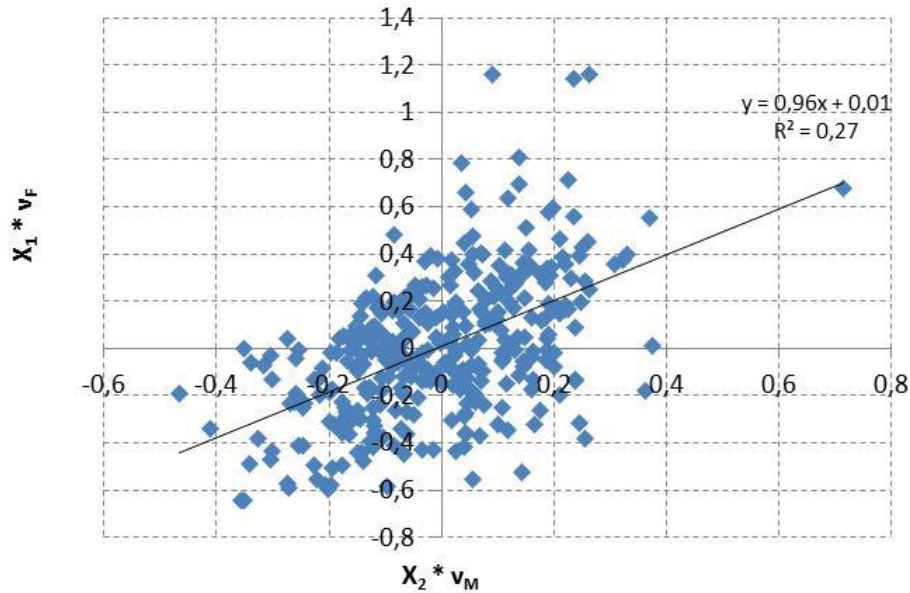


Figure 5.53. $(X_1 \cdot v_F)_i$ values are plotted against the $(X_2 \cdot v_M)_i$ values for the sample D1.

Figure 5.53 shows that $X_1 v_F$ and $X_2 v_M$ are correlated; the found coefficient of determination $r^2=0.27$. The mean values are practically zero. The values of v_F and v_M can be directly calculated from the standard deviation of $X_1 \cdot v_F$ and $X_2 \cdot v_M$. This gives $v_F=0.29$ and $v_M = 0.16$.

The values for v_F and v_M are higher than for dataset MAS5-C of tropical hardwoods. That is because in the values found for sample D2, also the measuring error for GKR is included, where the grain angle deviations of MAS5-C were specifically prepared to a target value.

With the derived C-values, the theoretical D-factors can be calculated according to equations (5.123), (5.124) and (5.125):

$$D_4 = 0.11 \left(1 - \frac{1.03}{0.37} \right) = -0.2$$

$$D_5 = \frac{0.11 \cdot 1.03}{26.9 \cdot 0.37} = 0.012$$

$$D_6 = -0.03$$

Now, the model according to equation (5.122) becomes:

$$f_{KR,mod} = -0.2\rho + 0.012 MOE_{KR} - 0.03 \quad (5.153)$$

In figure 5.54, the tested bending strengths are plotted against the model values of all samples calculated with equation (5.153). In figure 5.55, the residuals are plotted against the model values.

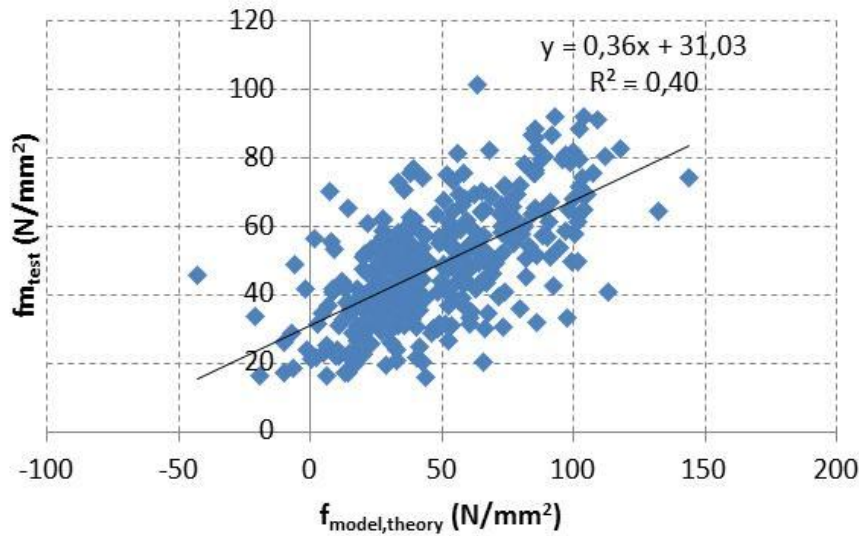


Figure 5.54. Bending test values plotted against model values calculated with (5.153)

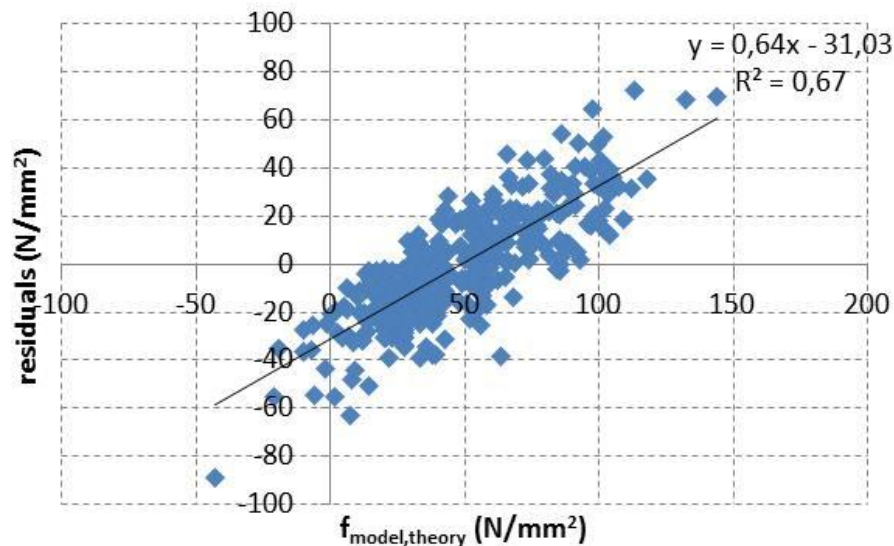


Figure 5.55. Residuals of the theoretical model values plotted against the theoretical model values. (5.153)

As expected, the regression line in figure 5.54 is not coinciding with $y=x$ and the residuals are not distributed around zero for all model values (figure 5.55).

The model values are now adjusted with the equation of figure 5.54.

This gives:

$$D_{4a} = -0.07$$

$$D_{5a} = 0.0042$$

$$D_{6a} = 31.0$$

The regression plot with the adjusted model values can now be directly compared with a direct least squares regression on the data. In the least squares regression, the sum of the residuals is minimized and the D-factors are derived directly.

The D-factors found from a least squares regression are:

$$D_4 = 0.0015$$

$$D_5 = 0.0037$$

$$D_6 = -3.7$$

The factors found in the direct least squares equation are different from the adjusted theoretical factors. The regression plots, however, just as for the tropical hardwood example, are very similar. See figures 5.56 and 5.57. For both models, the regression lines coincide with $y=x$ and figure 5.57 shows that the theoretical scatter pattern of the residuals is similar to the scatter pattern of the residuals from a least squares equation.

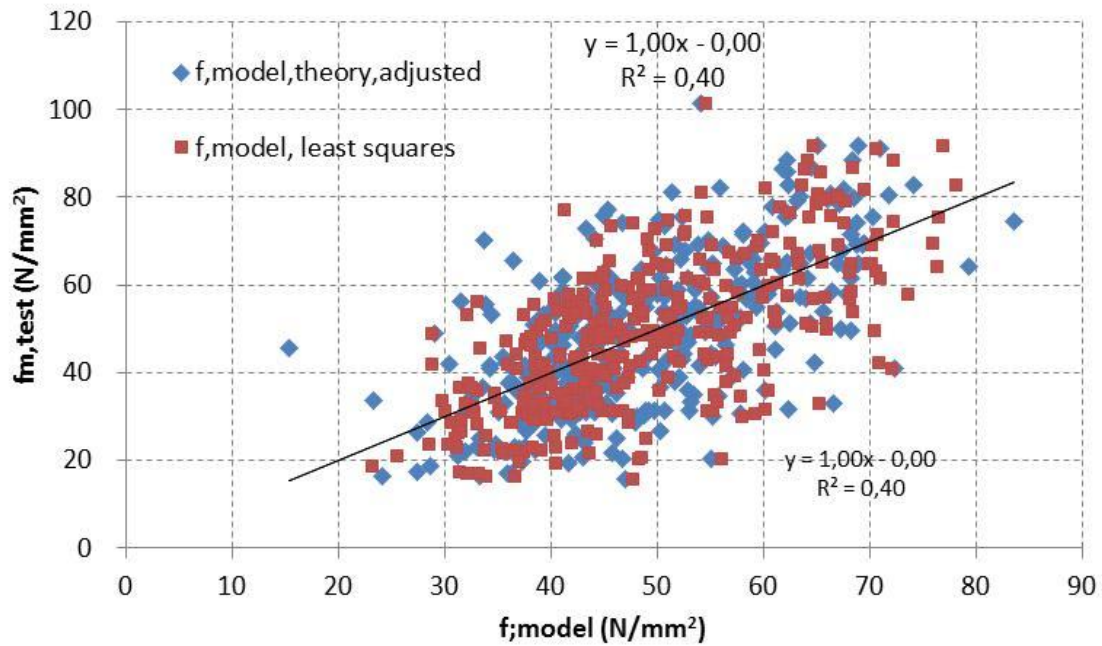


Figure 5.56. Bending test values plotted against theoretical and least squares model values.

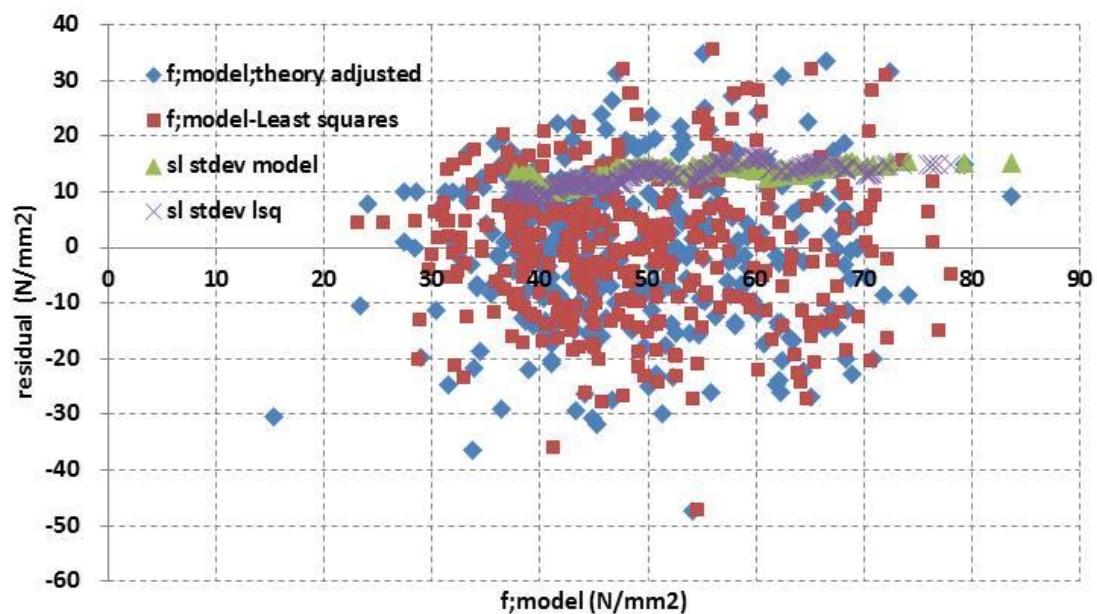


Figure 5.57. Residuals of the theoretical and least squares model plotted against theoretical and least squares model values. The theoretical and least squares sliding standard deviations are also plotted.

Figure 5.57 shows that the standard deviation of the errors very slightly increases with increasing model values. This can be explained from equation (5.135). When the knot ratio is evenly distributed for all densities, the increase of the standard deviation of the standard error mainly depends on the density. When a rather homogeneous sample is

investigated, only a slight increase is expected. This is the reason that in practice for datasets of softwood this slight increase is difficult to detect and a constant standard deviation of the error can be assumed. When the standard deviation is calculated with equation (5.135) and a mean value for the density and the group knot ratio value is used, a value of 13.0 N/mm² is found. The sliding standard deviation from the least squares regression at the mean model value is 12.7 N/mm².

It can be concluded, as with the dataset for tropical hardwood, that the sliding standard deviation from a least squares regression analysis is in line with the expected theoretical trend.

In figures 5.58 to 5.62, the results of simulations are shown. The input distributions of table 5.14 for density and group knot ratio are derived from the actual measured data of softwood sample D2.

Table 5.14. Input distributions for the simulations

Property	mean	Standard deviation	Distribution type
Density	575	62	normal
GKR	$\alpha=0.27$	$\beta=2.2$	weibull
X_1	0	1	normal
X_2	0	1	normal

Table 5.15. Values for constants used in the simulations

Property	
C_1	0.11
C_2	26.9
C_6	28.2
C_8	9.3
C_7	0.03
v_F	0.29
v_M	0.16
r^2 between X_1 and X_2	0.27

In figures 5.58 to 5.62, the results of the simulations are given together with the test results. The figures show that the actual observations are well described by the simulations. The trend of the standard sliding deviations of the residuals in figure 5.62 fit very well with the trend of the sliding standard deviation from the test data of douglas.

It can be concluded that the trend and shape of the test data for the sample of douglas is in line with what is expected from the developed theory. The sliding standard deviation gives a good estimation of the trend of the standard error with increasing model values. It can be

concluded that the strength of structural timber can be described when it is simulated with stochastic parameters.

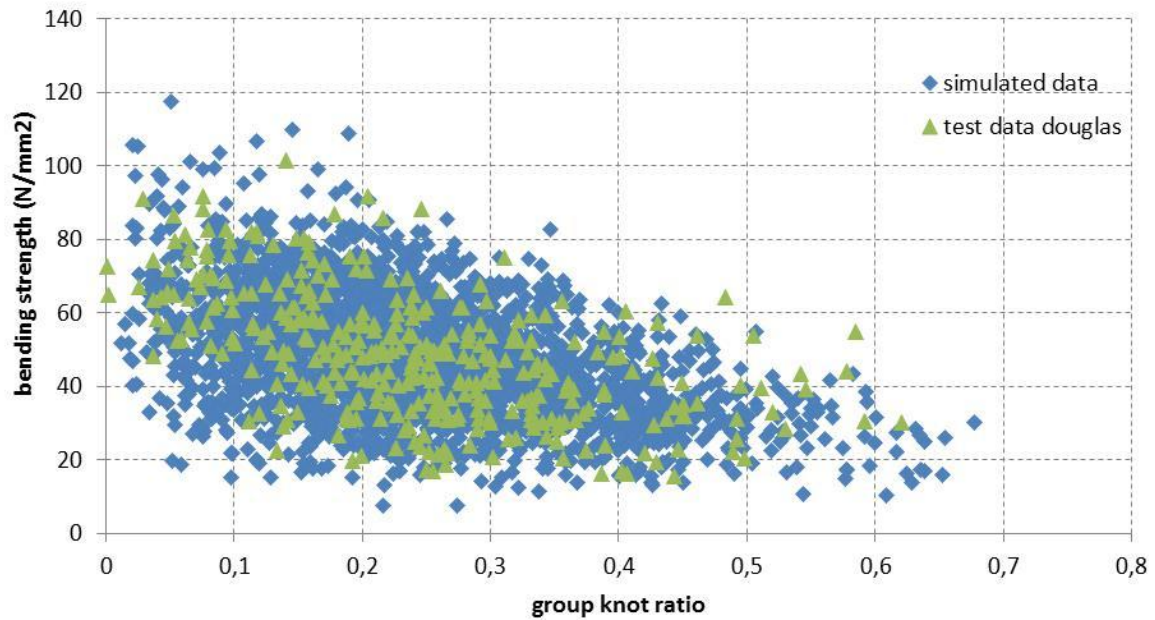


Figure 5.58. Bending strength plotted against the group knot ratio for the test samples and the simulations.

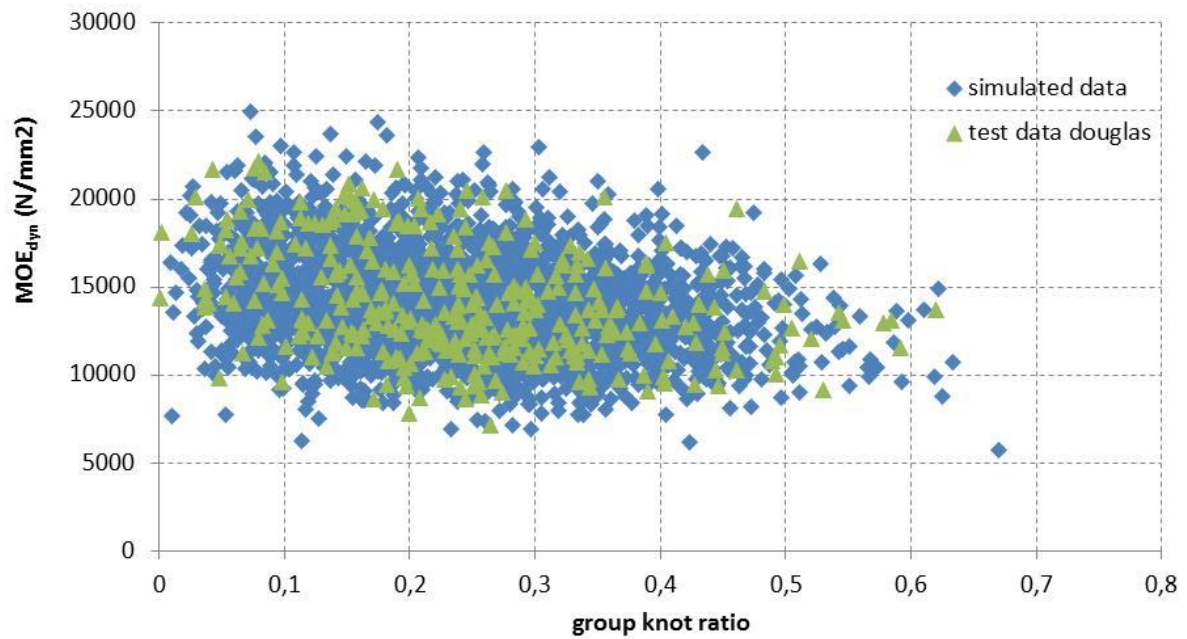


Figure 5.59. MOE_{dyn} plotted against the group knot ratio for the test samples and the residuals.

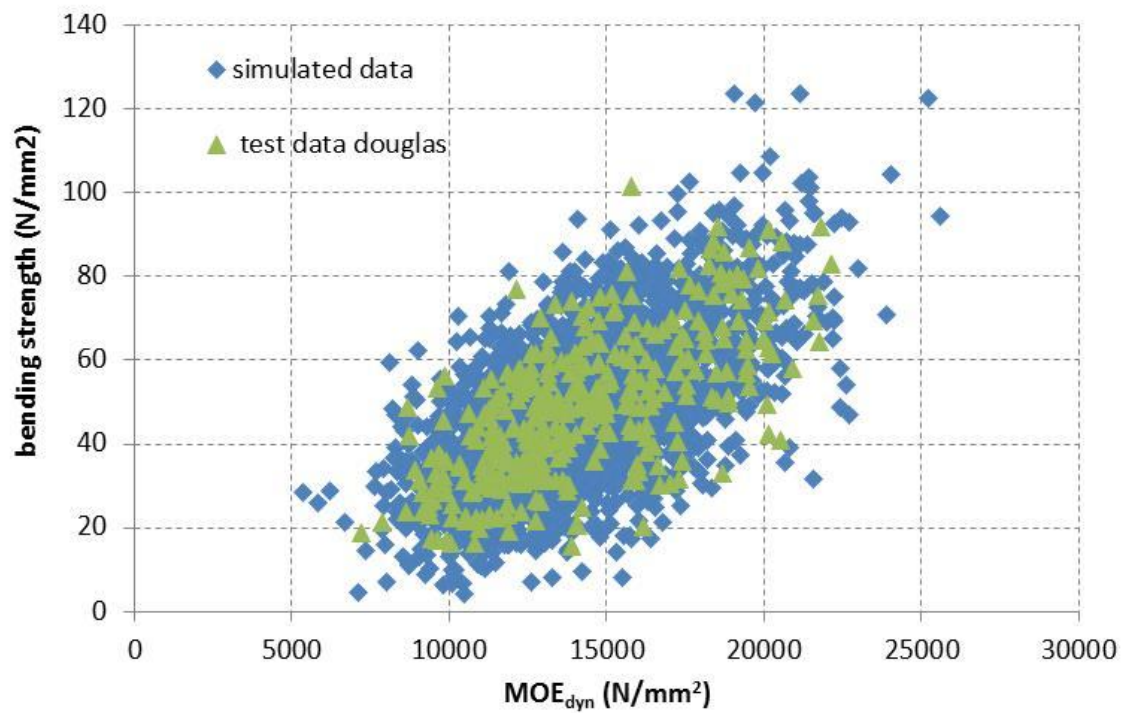


Figure 5.60. Bending strength plotted against MOE_{dyn} for the test samples and the residuals.

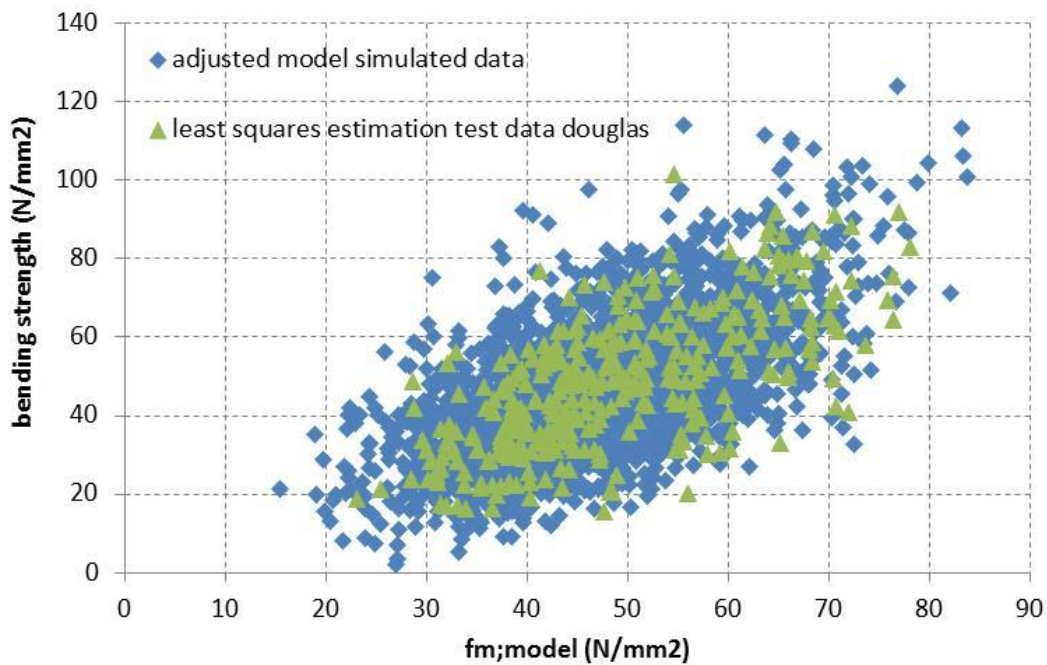


Figure 5.61. Bending strength plotted against the model values for the least squares estimation on the test samples and for the adjusted model on the simulated data.

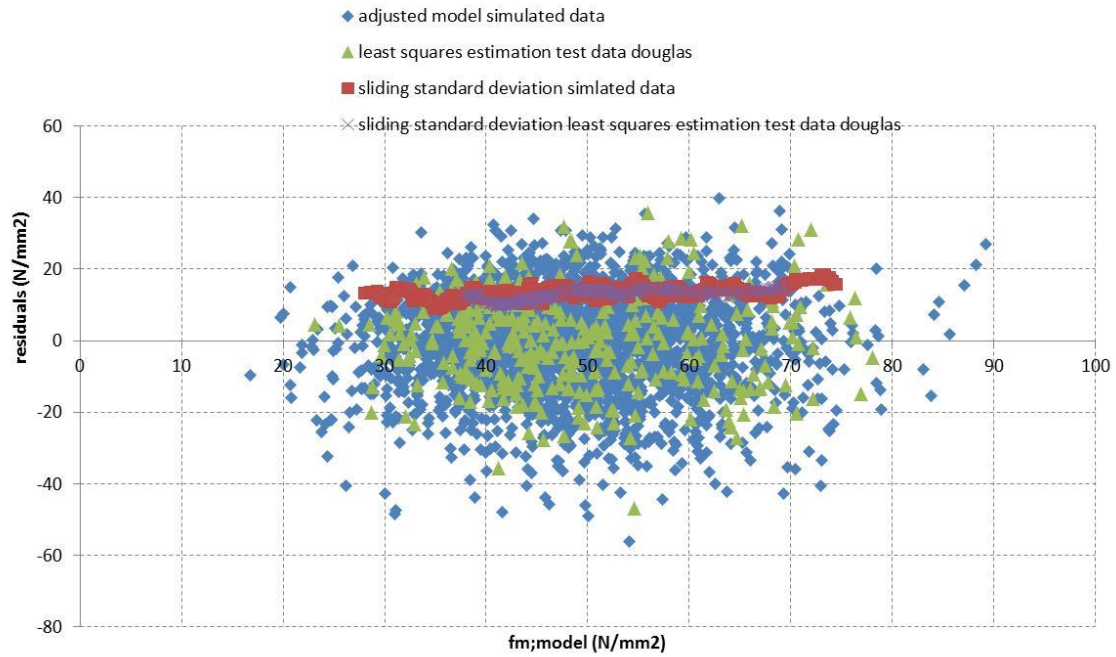


Figure 5.62. Residuals plotted against the model values for the least squares estimation of the bending strength on the test samples and for the adjusted model on the simulated data + the sliding standard deviation for the simulated data.

Implementation of the developed theory on experimental data

6.1 Introduction

In chapter 5, theoretical models were derived to predict the strength of clear wood, the strength of structural timber with knots and the strength of structural timber with grain angle deviation. These models were verified on real test data where detailed information on the strength reducing characteristics was available. It was shown that with the derived models the scatter around the prediction lines could be well predicted by the developed theory. It has been shown that the effect of the strength reducing characteristics on the predicting parameters for machine grading (density and *MOE*) is species independent, when a division is made between timber containing knots and timber containing grain angle deviation.

The outcome of a least squares estimation depends on the input distribution of the density and the strength reducing characteristics. For the datasets listed in chapter 3, the coefficients for the predicting models and the scatter around these models will be derived in this chapter.

From the analysis of the test results in chapter 4, it was concluded that species independent grading is not possible when the method of visual grading is applied. The reasons for that are:

- The density is a necessary parameter for species independent grading, but it is not possible to determine quantified values for the density for all wood species based on a visual examination of the timber.
- For tropical hardwoods, the slope of grain is the main influencing parameter on the strength and stiffness of the timber. However, it has been shown that this parameter cannot be determined with the required accuracy by visual inspection.

In contrast to visual grading, with machine grading the density and the slope of grain (the density directly by measuring the weight and dimensions, the slope of grain indirectly by measuring the Modulus of Elasticity) can be quantified for all species. Therefore, this chapter will discuss species independent grading for machine grading. The input parameters to predict the bending strength are the density and the MOE_{dyn} , the dimensions and the moisture content.

The density is measured directly from weight measurements and the dimensions. Adjustment of the density to the reference moisture content of 12% is performed in accordance with chapter 3.

The MOE_{dyn} is calculated from frequency measurements, density and length. Adjustment of the MOE_{dyn} to the reference moisture content of 12% is performed in accordance with chapter 3. To evaluate the reference MOE , which is the MOE_{local} , an adjustment factor has to be applied in accordance with chapter 4.

In section 6.2, the factors for the model equations for the bending strength for the datasets listed in chapter 3 will be determined based on the test data. With the quantified model, species independent grading will be performed in section 6.3.

6.2 Species independent strength modelling

Modelling the bending strength of the dataset of tropical hardwoods

The datasets of tropical hardwood of table 3.1 are classified as timber containing grain angle, but only limited knots. Some pieces were removed from the analysis based on visual override criteria. These pieces are listed in annex B. Machine strength grading for timber always has to be performed in combination with a visual override.

Equation (5.96) is used to model the bending strength for the dataset of table 3.1. Equation (5.96) is repeated here as equation (6.1)

$$f_{m,\alpha,mod} = \frac{\rho \cdot \min(MOE_{\alpha}; C_{10}\rho)}{D_1\rho + D_2 \cdot \min(MOE_{\alpha}; C_{10}\rho)} + D_3 \quad (6.1)$$

The term MOE_{α} is used in equation (6.1) to point out that the measured MOE takes into account the slope of grain (Therefore, only limited knots are allowed in the timber when this equation is used). The measured MOE_{dyn} values are used as input in this equation. Compared to equation (5.96) an extra factor C_{10} is introduced to address the scatter in the factor C_2 in equation (5.33). This factor, like all C -factors, is in this thesis handled as an absolute value, but in reality this factor can also be stochastic. The factor C_{10} ensures that the model values of equation (6.1) are not overestimated. The result of equation (6.1) is a value in N/mm^2 . This means that the D -factors will have different units to give the result of (6.1) in N/mm^2 . However, because the D -values are determined by regression analyses, these units have no specific physical meaning and the D -values will be listed without units. To determine the values of the D -factors in the model equations, a least squares regression will be performed on the data. There are two aspects of the dataset that have to be addressed in the regression:

- The distribution of the model values is not uniform (rectangular), but close to normally distributed. This means that the data clustered around the mean will have the largest influence on the regression output.

- The scatter around the regression line increases with increasing model values. This means no equal variance of the residuals. The data with residuals with the largest variance will have the largest influence on the regression analysis.

To overcome these two problems the following approach has been followed:

- An ordinary non-linear least squares regression is performed. The procedure to perform the non-linear least squares regression is explained in appendix A. With the D-values found the data can be ranked from low to high, based on the model values calculated with equation (6.1).
- Then, all data is divided in groups, based on the model values, calculated with equation (6.1) in the following way:
 - o Group 1: model values from 45 N/mm² to 50 N/mm²
 - o Group 2: model values from 50 N/mm² to 55N/mm²
 - o Group x: etc
 - o Group:18: model values from 130 N/mm² to 135N/mm²
- From every group, 10 datapoints are randomly drawn. This gives 180 datapoints for which the model values have a uniform rectangular distribution, giving every model value group equal influence.
- Then a non-linear ordinary least squares regression is performed on the dataset of 180 datapoints. The procedure how this was performed is described in appendix A. The model values of the 180 datapoints were ranked from low to high and the sliding standard deviations (with $n= 20$) are calculated. Slope w of these sliding standard deviations against the model values is calculated by performing a linear least squares regression, assuming that this line goes through the origin. Because no transformations are made yet on the residuals this is called an ordinary least squares equation (OLS). See equation (6.2) :

$$sliding\ standard\ deviation_i = w \cdot f_{model,OLS,i} \quad (6.2)$$

- After that the weighted residuals are calculated according to equation (6.3)

$$res_{weighted,i} = \frac{res_{OLS,i}}{w \cdot f_{model,OLS,i}} \quad (6.3)$$

- Then a non-linear least squares estimation was performed where the sum of squares of the weighted residuals is minimized. The weighted residuals will have equal variance over the range of model values.

Figure 6.1 shows the residuals of a run from the ordinary least squares regression. The sliding standard deviation of the residuals shows an increasing trend and a value for w of 0.18 is found. In figure 6.2 the residuals from the weighted least squares regression is shown on the same dataset. Now the residuals show an equal standard deviation of the error over the whole range of model values.

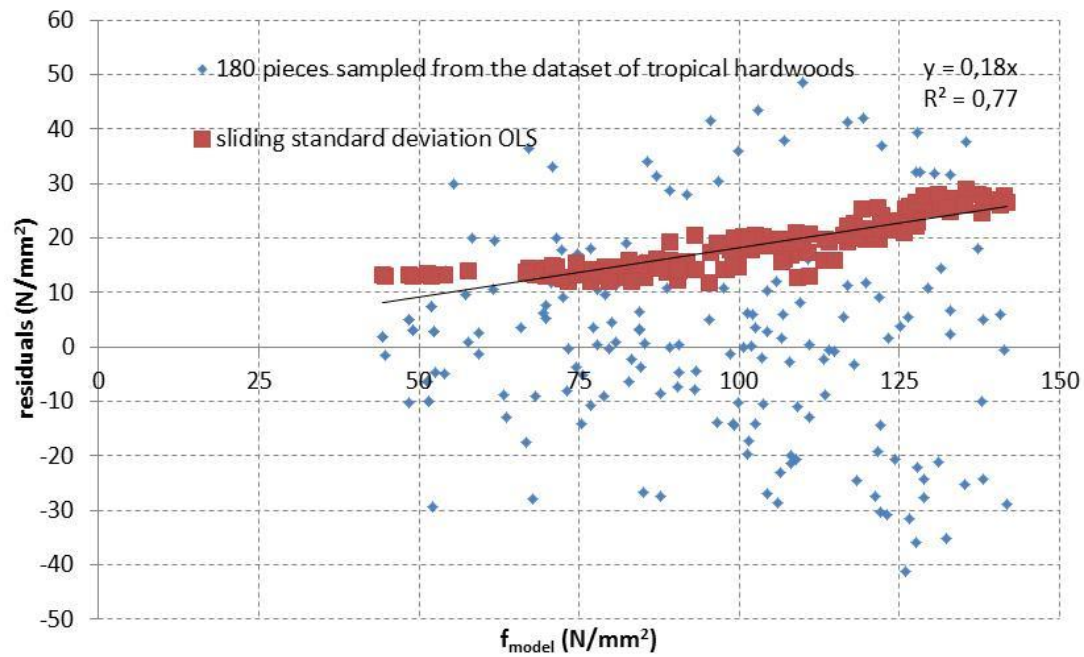


Figure 6.1. Residuals plotted against the model values for a run with 180 pieces from an ordinary least squares regression together with the sliding standard deviation.

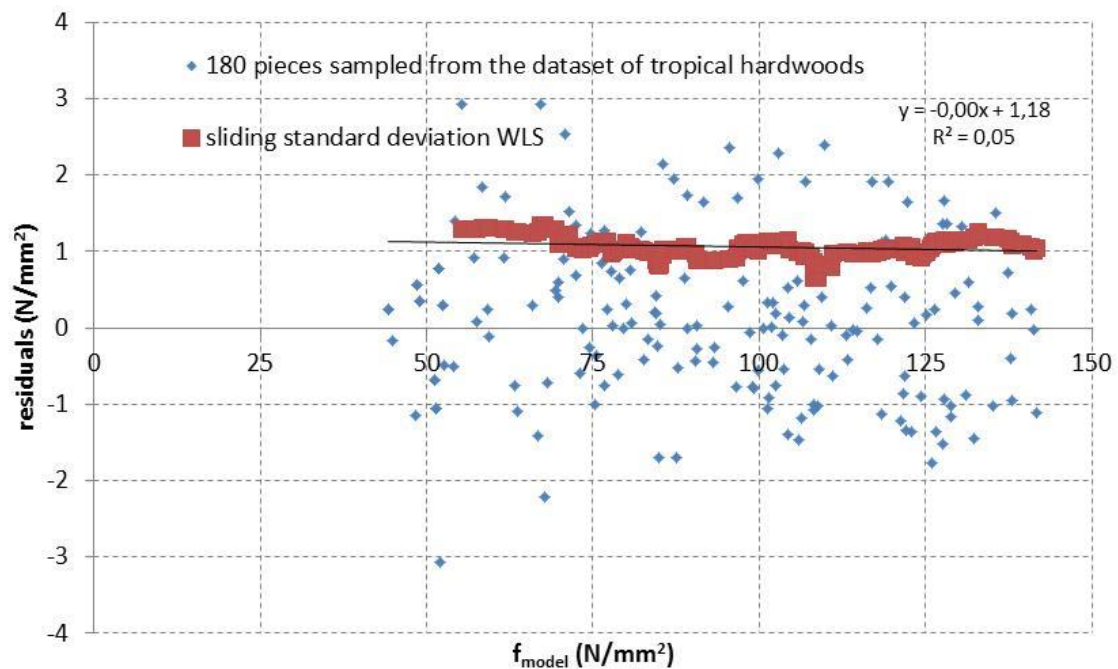


Figure 6.2. Residuals plotted against the model values for the same run as in figure 6.1, but now from a weighted least squares regression together with the sliding standard deviation.

In table 6.1, the results of the obtained D-values of equation (6.1) are given. The means and standard deviations are based on 100 runs. In every run, 10 datapoints for every model group were randomly drawn.

Table 6.1. Factor values obtained by ordinary and weighted least squares analysis based on 100 runs.

Regression method	Factor values					
	D_1		D_2		D_3	
	mean	stdev	mean	stdev	mean	stdev
Ordinary Least Squares regression	166.0	33.1	0.65	1.0	-20.7	7.5
Weighted Least Squares regression	183.8	32.0	0.15	0.98	-12.9	5.9

Table 6.1 shows that a weighted least squares regression gives different factor-values than an ordinary least squares regression. The mean D-factors from the weighted least squares regressions will be used in the grading model. Figure 6.3 shows for the total sample the bending strength plotted against the model values calculated according to equation (6.4):

$$f_{m,\alpha,mod} = \frac{\rho \cdot \min(MOE_{\alpha}; 25\rho)}{183.8\rho + 0.15 \min(MOE_{\alpha}; 25\rho)} - 12.9 \quad (6.4)$$

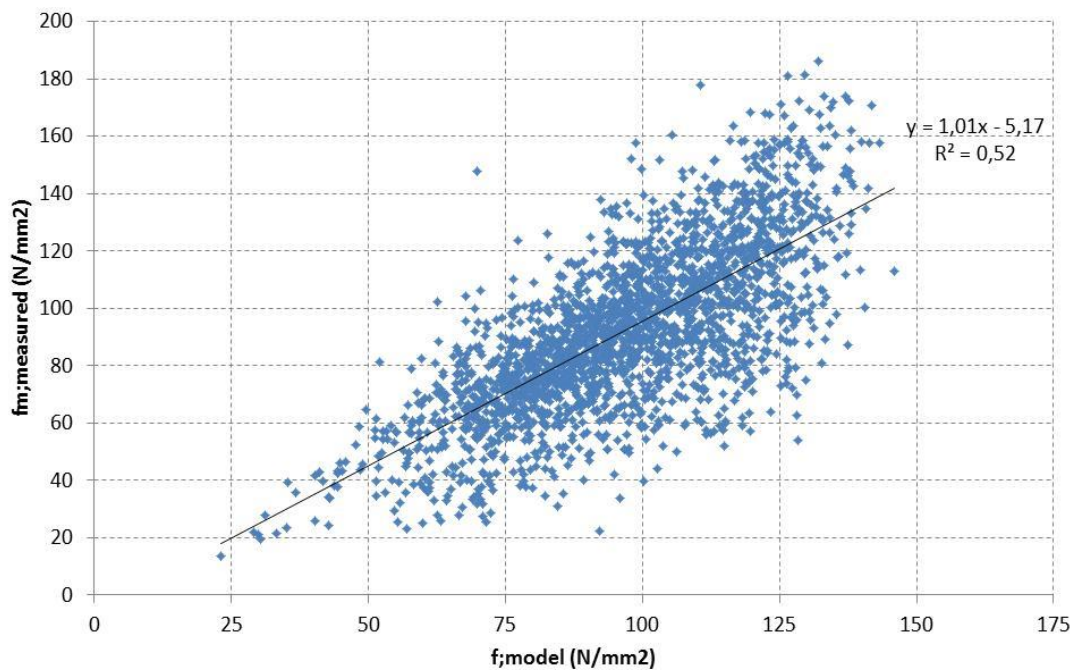


Figure 6.3. Bending strength values for the entire dataset of tropical hardwoods plotted against the model values with the factors obtained from a weighted least squares regression.

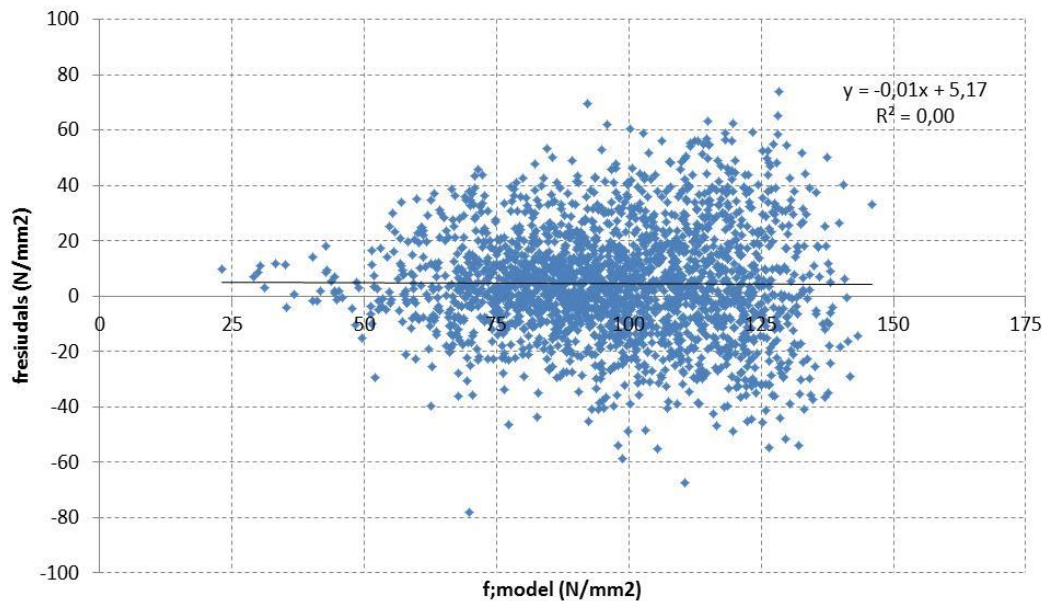


Figure 6.4. Residuals for the entire dataset of tropical hardwoods plotted against the model values with the factors obtained from a weighted least squares regression.

Figure 6.4 shows the residuals of the model of equation (6.4).

Figure 6.3 shows that, as a result of the WLS-analysis, the prediction line for the entire set does not coincide with $y=x$, but slightly deviates. The slope and intercept have to be entered as the values for A and B in equation (3.33) to apply the theory of section 3.4. Therefore the values for A and B are determined for each 100 runs every time on 180 sampled pieces. These values are listed in table 6.2.

Table 6.2 Mean and standard deviation for the factors A and B from 100 runs

Factor values		
property	A	B
Mean	1.01	-4.63
Standard deviation	0.02	2.18

The mean values for A and B will be used to derive the settings.

To determine settings with the theory of section 3.4, the ratios of the residuals with the model value has to be known. These are calculated for every run. The procedure is followed: The D-factor values obtained by a weighted least squares regression was used to calculate the model values. The residuals are plotted against the model values and the sliding standard deviation as calculated for every model point. Then a linear regression line is fitted through the sliding standard deviation point. Once by assuming the intercept was zero (equation 6.5) and once with an intercept included (equation 6.6).

$$\text{sliding standard deviation}_i = w_{1,WLS} \cdot f_{model,WLS,i} \quad (6.5)$$

$$\text{sliding standard deviation}_i = w_{2,WLS} \cdot f_{model,WLS,i} + w_{3,WLS} \quad (6.6)$$

The values obtained for the factors from equations (6.5) and (6.6) are listed in table 6.3.

Table 6.3. Mean and standard deviations for the factors according to (6.5) and (6.6).

Factor values			
property	$w_{1,WLS}$	$w_{2,WLS}$	$w_{3,WLS}$
Mean	0.16	0.18	2.41
Standard deviation	0.04	0.01	3.55

Now, the question is which equation for the standard error should be used. If the used standard error is too large for a certain range of model values, the grading will be less efficient in that range, if the used standard error is too small for a certain range of model values, the grading will be unsafe.

When only one parameter $w_{1,WLS}$ is used, the 95% fractile would be 0.22, but this might be unsafe for lower model values. When the 95% fractiles for both $w_{2,WLS}$ and $w_{3,WLS}$ would be used, this would be very conservative. Therefore, it was decided to use the mean value of 0.18 for $w_{2,WLS}$ and the 95% fractile of 8.0 for $w_{3,WLS}$. In that case the ratio of the standard error for higher model values becomes very close to 0.22. In this way, safe and economic values are ensured for both low and high model values.

Modeling the bending strength of the dataset of temperate hardwoods and softwoods

Because both softwoods and temperate hardwoods contain knots, which are the major cause of failure they are combined for species independent grading. For the analysis, the datasets of tables 3.2 and 3.3 were merged.

For softwoods and temperate hardwoods it was found that the standard error slightly increases with increasing model values, due to increasing density. For the datasets of softwoods and temperate hardwoods a least squares regression is performed and the sliding standard error is calculated. The D-factors according to model (5.122) are obtained by an ordinary least squares regression. Equation (5.122) is repeated here as equation (6.7):

$$f_{KR,mod} = D_4\rho + D_5MOE_{KR} + D_6 \quad (6.7)$$

The term MOE_{KR} is used in equation (6.7) to point out that the measured MOE takes into account the knots present in the timber (Therefore, only limited slope of grain is allowed in the timber when this equation is used). As for timber for which equation (6.1) is used, the measured MOE_{dyn} values are used as input in this equation.

In table 6.4 the values obtained for the D-factors are listed.

Table 6.4. Values for the D-factors of model (6.7) obtained by linear regression.

Factor	value
D_4	0.025
D_5	0.0031
D_6	-7.3

In figure 6.5, the measured bending strength values are plotted against the model values. The data for softwoods and temperate hardwoods are separately marked. It can be seen that they both fit in the same model. The assumption was that for both species groups the presence of knots cause the same failure mechanisms and therefore the “knot model” can be used for both species groups. In figure 6.6, the sliding standard error of the model is presented. The sliding standard error is slightly increasing with increasing model value with the formula: $y = 0.11x + 5.6$.

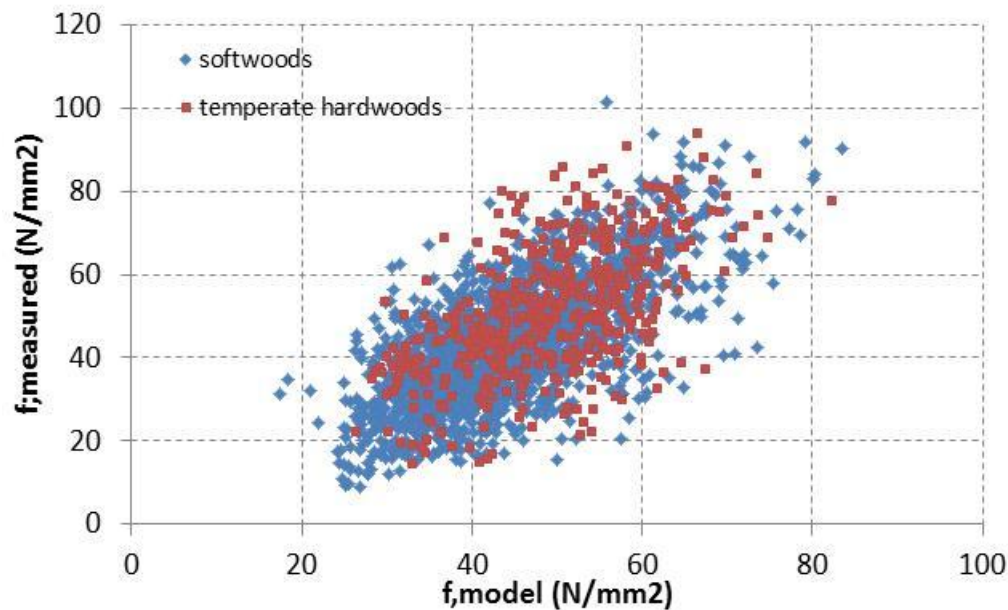


Figure 6.5. Bending strength values plotted against the model values according to (6.7)

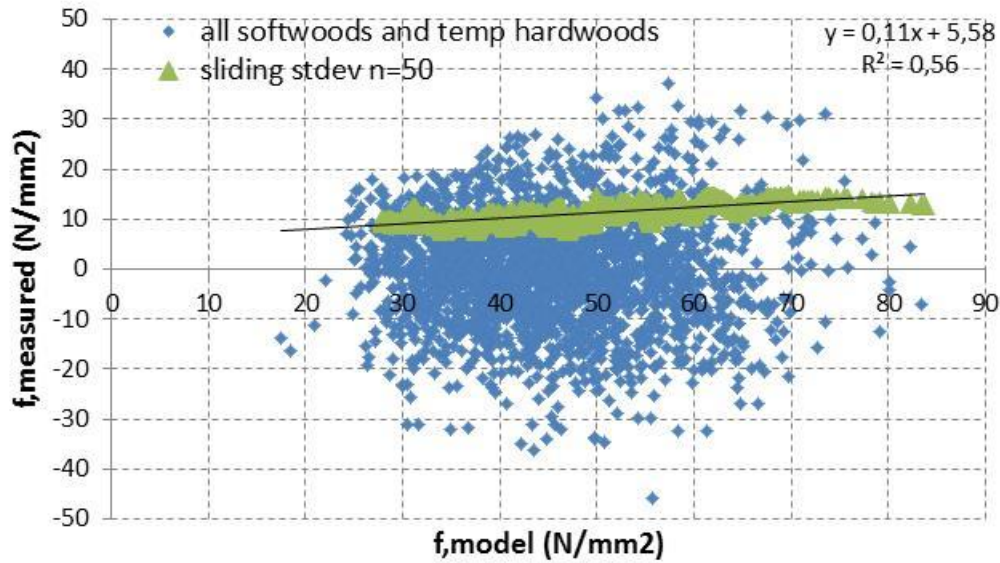


Figure 6.6. residuals plotted against the model values according to (6.7) together with the sliding standard deviation.

6.3 Species independent strength grading

6.3.1 Species independent strength grading of the dataset of tropical hardwoods

With the method derived in section 3.4, settings for machine grading for tropical timber can be derived based on the model parameters derived in section 6.2. The model and constants are given in equation (6.4):

$$f_{m,\alpha,model} = \frac{\rho \min(MOE_{\alpha}; 25.0\rho)}{183.8\rho + 0.15 \min(MOE_{\alpha}; 25.0\rho)} - 12.9$$

For the standard deviation of the error:

$$\sigma_{m,\alpha,model} = 0.18f_{m,\alpha,model} + 8.0$$

The distribution properties are:

- $f_{m,\alpha,model} : \mu = 97.2 \text{ N/mm}^2, \sigma = 20.4 \text{ N/mm}^2$
- $f_{m,measured} : \mu = 92.7 \text{ N/mm}^2, \sigma = 28.4 \text{ N/mm}^2$
- Relation between $f_{m,\alpha,model}$ and $f_{m,measured}$:

$$f_{m,measured} = A f_{m,\alpha,model} + B, \text{ where } A=1.01 \text{ and } B= -4.6.$$

(in this case $f_{m,\alpha,model} = IP_{fm}$ as defined in section 3.4).

Grading of the entire dataset of tropical hardwoods

The settings for the grade combination D70-D50-D30 in accordance with the method described in section 3.4.

In table 6.5 the found settings are listed. The values for $p(i)$ -high are limited to 0.15. This is because the mean value of $p(i)$ should be 0.05, and allowing too high $p(i)$ -values makes the method more sensitive to anomalies in the data distributions. The yield is the percentage of the total amount of pieces, assigned to a certain strength class or reject.

Table 6.5. Settings for grade combination D70-D50-D30-reject for the whole sample

Strength class	$f_{m,stat}^{required}$ (N/mm ²)	$IP_{fm, low}$ (N/mm ²)	Expected Yield (%)	$p(i),low$	$p(i),high$	p_{char}
D70	70.0	112.0	23.3	0.00	0.081	0.049
D50	50.0	80.0	56.7	0.017	0.11	0.049
D30	30.0	52.0	18.7	0.02	0.14	0.040
reject			1.0			

In figure 6.7. the $p(i)$ -values for the intervals of the IP_{fm} -values of the strength classes D70, D50 and D30 are given together with the distribution of the model values.

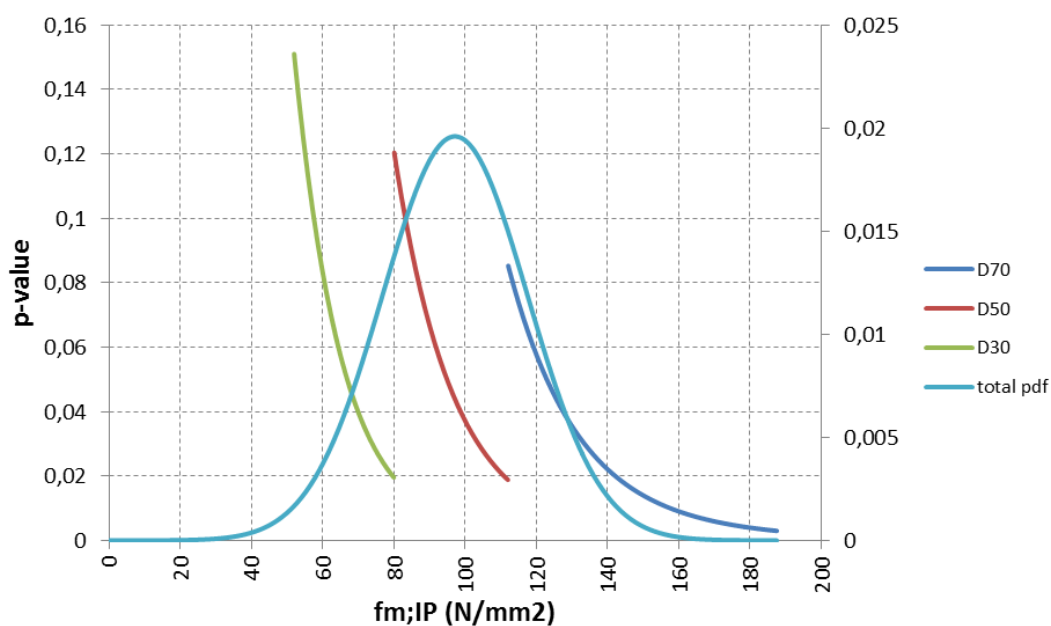


Figure 6.7. p -values plotted against the model values for the settings of table 6.5.

The whole sample is now graded with the settings ($=IP_{fm, low}$) from table 6.5. The results are presented in table 6.6.A for the bending strength and in table 6.6.B for the MOE and density. In section 3.4 it was stated that it made no sense to calculate the 5% fractiles according to standardised methods to verify the settings. The purpose of the listing of 5% fractiles is only to get a feeling what results the standardised methods give.

The mean $p(i)$ -values gives valuable information because when this is close to 0.05 then it means that the assumptions for the distribution of IP_{fm} (in this case a normal distribution is assumed) is correct.

Table 6.6.A Descriptive statistics of graded data with settings from table 6.5.

Strength grade	$n=$	Yield (%)	Mean $p(i)$	5% fractiles of the bending strength		
				ranking (N/mm ²)	Central t- (N/mm ²)	Required value (N/mm ²)
D70	590	26.6	0.055	75.0	75.3	70
D50	1175	52.9	0.054	60.9	57.7	50
D30	421	19.0	0.045	52.1	36.6	30
reject	35	1.5				-

Table 6.6.B Descriptive statistics of graded data with settings from table 6.5.

Strength grade	$n=$	Yield (%)	Mean MOE	Required value (N/mm ²)	5% fractiles of the density	
					Central t- (kg/m ³)	Required value (kg/m ³)
D70	590	26.6	24500	20000	950	900
D50	1175	52.9	19300	14000	740	620
D30	421	19.0	-	11000	580	530
reject	35	1.5				-

It can be seen that the predicted yields are very well in line with the real yields. The mean $p(i)$ -values are very close to the expected 0.05. The 5% fractiles are higher than the required values. In figure 6.8, the theoretical cumulative distributions, calculated with the theory of section 3.4, are plotted with the distributions of the destructive measurements. Figure 6.9 zooms in at the lower tail of figure 6.8.

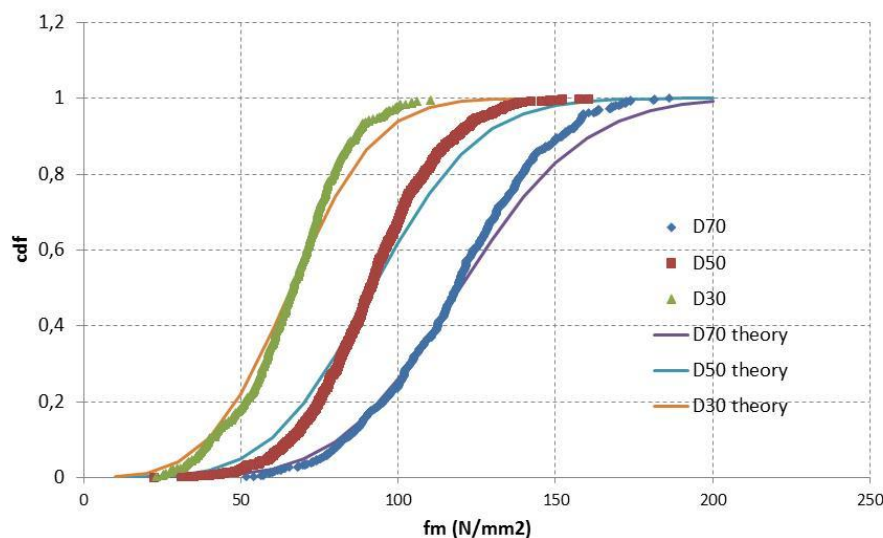


Figure 6.8. Cumulative distributions of bending strength of graded specimens. Observed distributions and theoretical distributions.

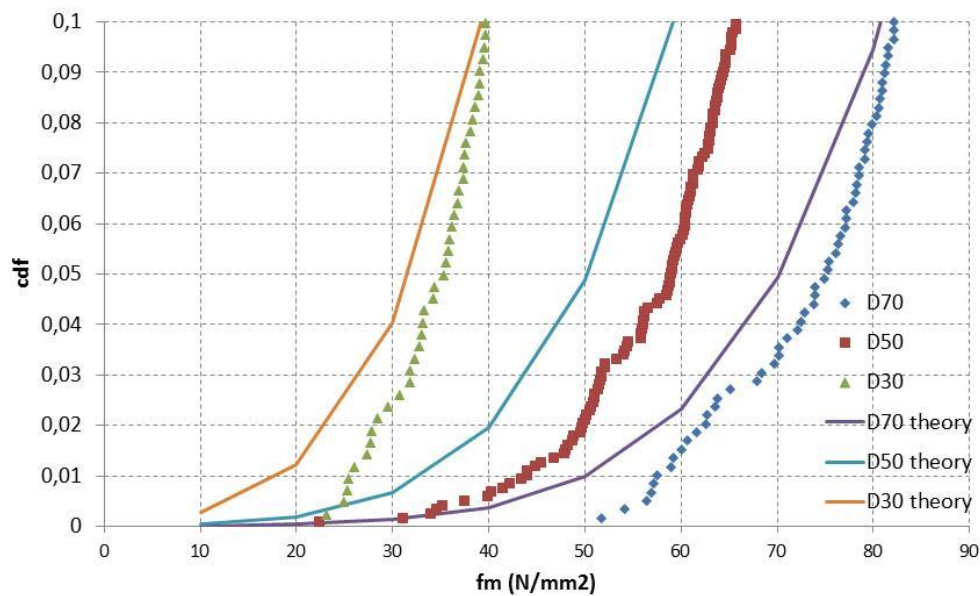


Figure 6.9. Plot 6.8 zoomed in to the lower tail.

It can be observed that both the distribution from the destructive tests and the theoretical distributions can be very well approximated by normal distributions, and that the distributions of the destructive tests and the theoretical distributions are very similar. Slight differences might be caused by the chosen standard deviation of the error or the fact that the model value distribution is not as smooth as a normal distribution. The graph of the lower tail shows that the observed destructive distributions are safe compared with the theoretical ones.

The previous example shows that when tropical hardwood is regarded as one population, species independent strength grading can be safely applied. To ensure extra safety the ratio of the standard error with the model value could be increased.

To make a comparison in terms of yield between visual and machine strength grading the trade name timber samples are assigned to the visual grades D70-D50-D30-reject based on the calculated characteristic values of table 4.20. Again, it is mentioned that this is only based on the samples tested and not yet applies to the trade name, since then the factor $k_{s,tn}$ should be applied, which could not be determined for application to all species yet. When this would be done, the assigned strength classes for visual grading would even turn out lower. The assignments based on visual grading given in table 4.20 are the highest strength classes possible on the current available data. This is done to make a comparison between the yield of visual grading and machine grading. The results with the comparison of yields are given in table 6.7.

Table 6.7. Comparison of yields between visual grading and species independent machine grading.

Grading method	Yields strength grades (%)			
	D70	D50	D30	reject
Visual grading	4.5	52.0	33.2	10.3
Species independent machine grading	26.6	52.9	19.0	1.5

Table 6.7 shows that species independent machine grading is not only a more reliable method, but also that the output in terms of yield is much higher than with visual grading.

Verification of the dataset of massaranduba.

In chapter 4.4 it was explained that by visual grading only a characteristic value of 38.9 N/mm² could be assigned to massaranduba, with the factor $k_{s,m}$ not yet applied. This means that for the tested samples (not yet the timber from the trade name massaranduba) by visual grading all pieces would be assigned to D30, when only strength classes D70-D50-D30 are evaluated. Remark: this is only done to compare the yield of the visual grading of the tested samples of massaranduba.

Now the samples MAS1- MAS5 of massaranduba will be evaluated by machine grading. The settings from table 6.5 are used. In tables 6.8.A and 6.8.B the results are presented.

Table 6.8.A. Descriptive statistics of graded massaranduba with settings from table 6.5.

Strength grade	n=	Yield (%)	Mean $p(i)$	5% fractiles of the bending strength		
				ranking (N/mm ²)	Central t- (N/mm ²)	Required value (N/mm ²)
D70	122	49.0	0.048	87.5	86.7	70
D50	79	31.7	0.043	70.4	71.4	50
D30	40	16.1	0.072	54.3	50.4	30
reject	8	3.2				-

Table 6.8.B. Descriptive statistics of graded massaranduba with settings from table 6.5

Strength grade	n=	Yield (%)	Mean MOE	Required value (N/mm ²)	5% fractiles of the density	
					Central t- (kg/m ³)	Required value (kg/m ³)
D70	122	49.0	24600	20000	1000	900
D50	79	31.7	19700	14000	940	620
D30	40	16.1	13100	11000	930	530
reject	8	3.2				-

In figure 6.10, the bending strength values are plotted against the model values for the dataset of massaranduba. In figure 6.11, the observed and theoretical distributions for the graded pieces are shown.

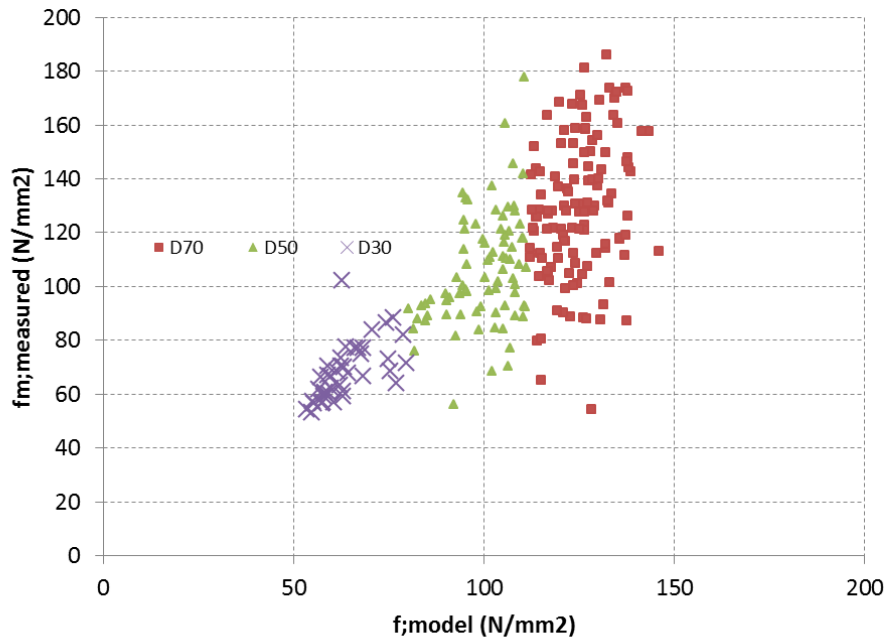


Figure 6.10. Bending strength values plotted against the model values for the dataset of massaranduba

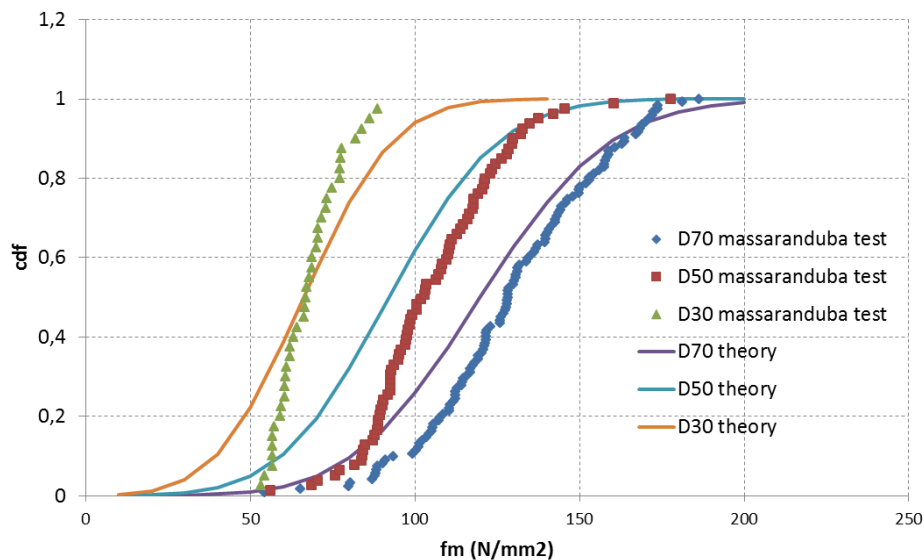


Figure 6.11. Cumulative distributions of bending strength of graded specimens of massaranduba. Observed distributions and theoretical distributions.

The figures and the tables 6.8.A. and 6.8.B. indicate that when massaranduba is machine graded, almost 50% can be graded to strength class D70. The reason for this is that the weak pieces can be distinguished from the strong pieces by machine grading (where this is not possible with visual grading). Figure 6.11 suggests that further optimisation for

massaranduba is possible when the input distribution of the model values of massaranduba would be used to derive the settings, instead of the input of the whole population of tropical hardwoods.

Verification of the dataset of cumaru

In chapter 4.4 it was explained that by visual grading the cumaru samples could be assigned to a characteristic value of 53.0 N/mm², with the factor $k_{s,tn}$ not yet applied. This means that for the tested samples (not yet the timber from the trade name cumaru) by visual grading all pieces would be assigned to D50, when only strength classes D70-D50-D30 are evaluated.

Samples CUM1-CUM5 of cumaru will be evaluated by machine grading. The settings from table 6.5 are used. In tables 6.9.A and 6.9.B the results are presented.

Table 6.9.A. Descriptive statistics of graded cumaru with settings from table 6.5.

Strength grade	n=	Yield (%)	Mean $p(i)$	5% fractiles of the bending strength		
				ranking (N/mm ²)	Central t- (N/mm ²)	Required value (N/mm ²)
D70	60	27,3	0.057	81.3	89.1	70
D50	139	63.2	0.046	55.9	61.2	50
D30	21	9.5	0.038	54.1	39.1	30
reject	1	0,0				-

Table 6.9.B. Descriptive statistics of graded cumaru with settings from table 6.5

Streng th grade	n=	Yield (%)	Mean MOE	Required value (N/mm ²)	5% fractiles of the density	
					Central t- (kg/m ³)	Required value (kg/m ³)
D70	60	27,3	23700	20000	950	900
D50	139	63.2	19200	14000	850	620
D30	21	9.5	14600	11000	790	530
reject	1	0,0				-

In figure 6.12, the bending strength values are plotted against the model values for the dataset of cumaru. In figure 6.13, the observed and theoretical distributions for the graded pieces are shown.

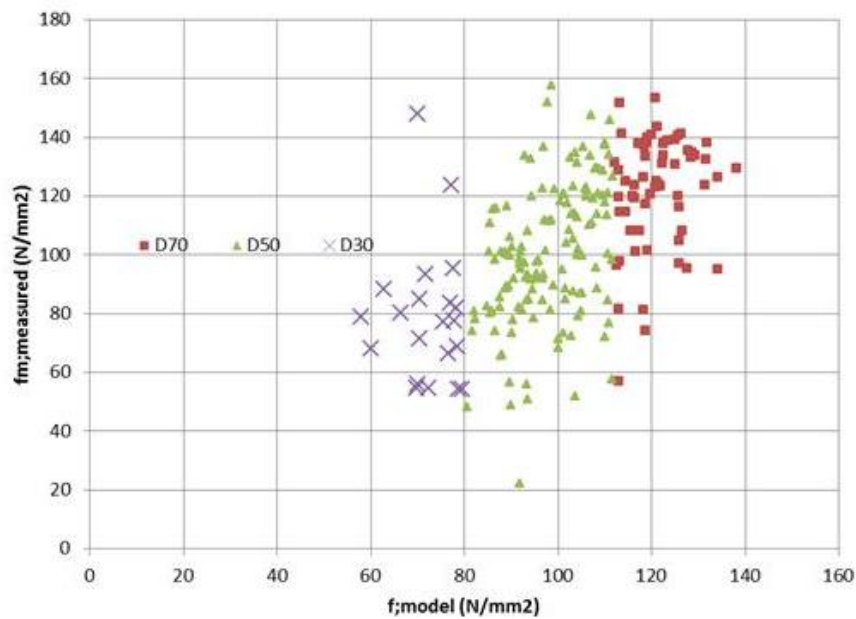


Figure 6.12. Bending strength values plotted against the model values for the dataset of cumaru

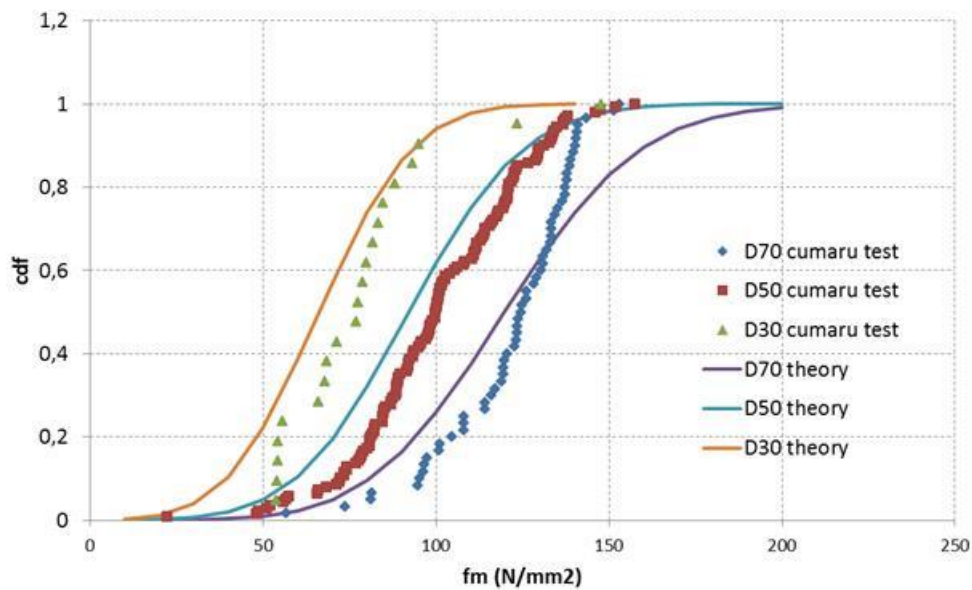


Figure 6.13. Cumulative distributions of bending strength of graded specimens of cumaru. Observed distributions and theoretical distributions.

Table 6.9 shows that when cumaru is machine graded, more than 25 % can be graded in D70, and that the specimens of weak samples are detected, in contrast with the method of visual grading.

Verification of the dataset of bilinga, okan, tali and evuess

The samples of the four wood species Bilinga, Tali, Okan and Eveuss, for which large grain angle deviations were measured after destructive tests, are now evaluated to investigate the prediction capability of the model for low strengths.

Samples BIL1, BIL2, TA1, TA2, OK2, OK3, EV1 and EV2 will be evaluated by machine grading. The settings from table 6.5 are used. In tables 6.10.A and 6.10. B the results are presented.

Table 6.10.A. Descriptive statistics of graded bilinga, okan, tali and evuess with settings from table 6.5.

Strength grade	n=	Yield (%)	Mean $p(i)$	5% fractiles of the bending strength		
				ranking (N/mm ²)	Central t- (N/mm ²)	Required value (N/mm ²)
D70	102	26.9	0.054	77.3	74.3	70
D50	136	35.9	0.065	45.4	51.2	50
D30	121	31.9	0.051	27.5	25.6	30
reject	20	5.3				-

Table 6.10.B. Descriptive statistics of graded bilinga, okan, tali and evuess with settings from table 6.5

Strength grade	n=	Yield (%)	Mean MOE	Required value (N/mm ²)	5% fractiles of the density	
					Central t- (kg/m ³)	Required value (kg/m ³)
D70	102	26.9	24400	20000	970	900
D50	136	35.9	18100	14000	800	620
D30	121	31.9	14000	11000	620	530
reject	20	5.3				-

In figure 6.14, the bending strength values are plotted against the model values for the dataset of bilinga, okan, tali and evuess. In figure 6.15 the observed and theoretical distributions for the graded pieces are shown.

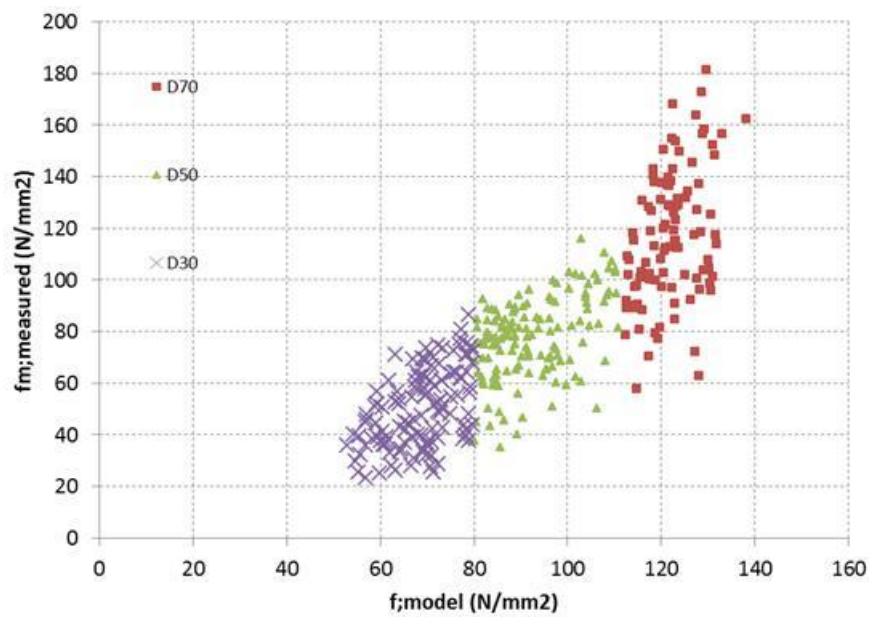


Figure 6.14. Bending strength values plotted against the model values for the dataset of bilinga, okan, tali and evuess

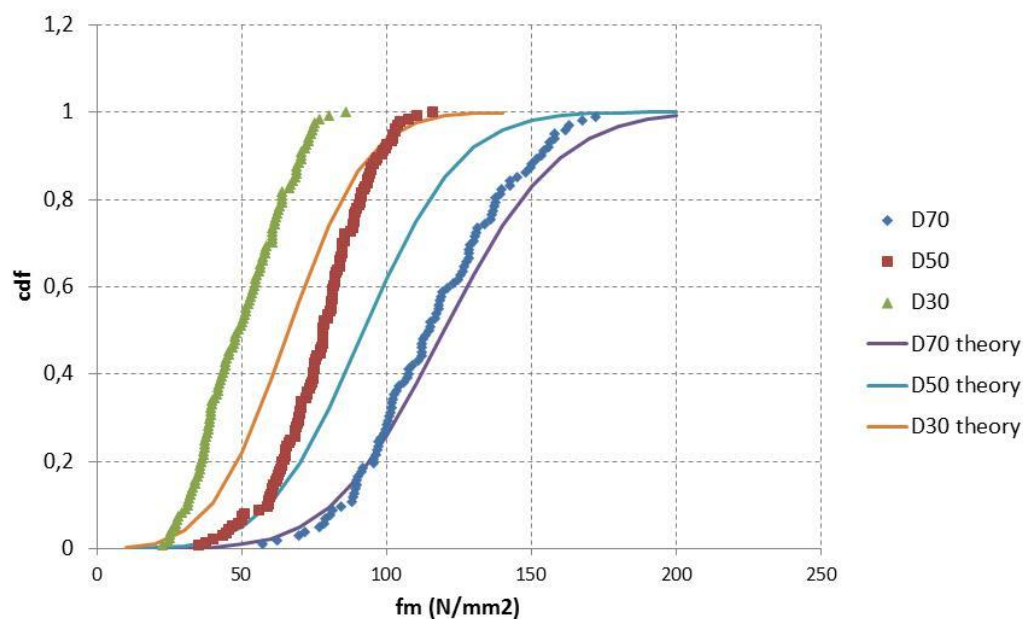


Figure 6.15. Cumulative distributions of bending strength of graded specimens of bilinga, okan, tali and evuess. Observed distributions and theoretical distributions.

Table 6.10.A suggests that the 5% fractiles of the strength grade D30 might not comply with the required values. The distributions for D30 and D50 also do not match with the theoretical distributions of the strength classes, because the input distribution of the model values for all data of tropical hardwood (on which the theoretical distributions are based) is different from that of the investigated samples. However, as explained in section 3.4, the used method is developed to stay away from trial and error on a limited amount of data. To evaluate the grading results at the 5%-level, figure 6.16 zooms in on the lower

tail of D50 of figure 6.15 and figure 6.17 zooms in on the lower tail of D30 of figure 6.15. In figures 6.16 and 6.17 the 90% confidence intervals of the theoretical distributions are also plotted. Figures 6.16 and 6.17 show that the 5% fractile of the observed distributions are within the theoretically expected 90% confidence interval. Another thing that can be observed from figure 6.15 is that the standard deviation of the distribution is much lower than that of the theoretical distribution. This means that in terms of reliability this will have a positive effect.

It can be concluded that machine grading also works for weaker samples where the yield is much higher than for visual grading.

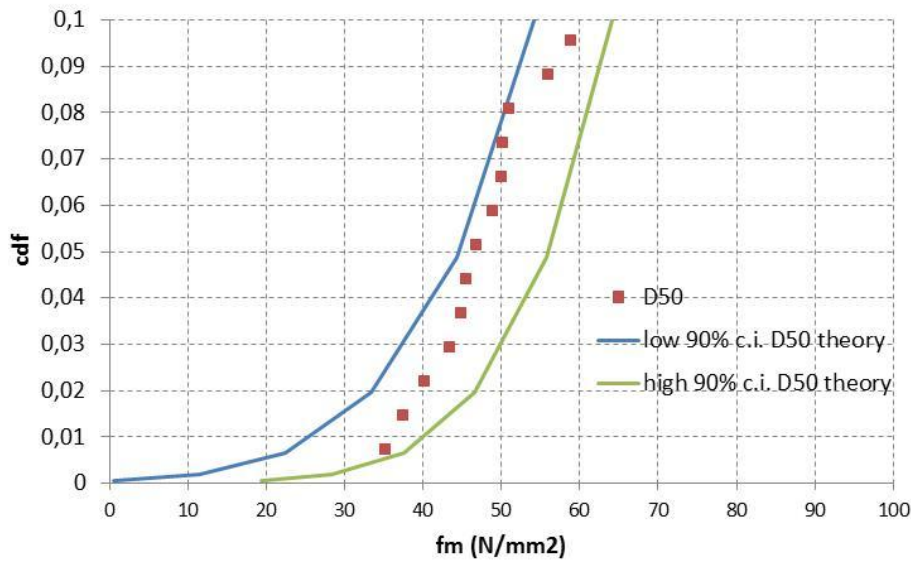


Figure 6.16. Cumulative observed distribution of the bending strength of grade D50 for bilinga, okan, tali and evuess and the 90% confidence interval for the theoretical distribution.

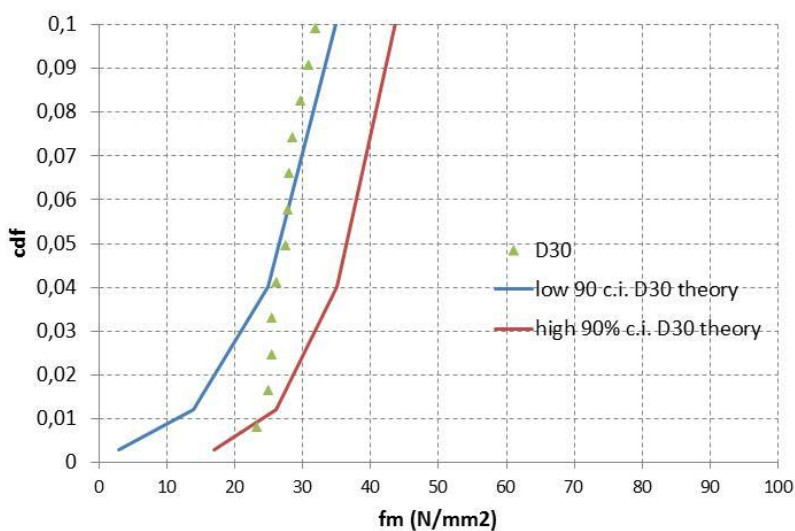


Figure 6.17. Cumulative observed distributions of bending strength of grade D30 for bilinga, okan, tali and evuess and the 90% confidence interval for the theoretical distribution.

Verification of the dataset of azobé

The 3 samples AZ1, AZ2 and AZ3 are analysed and graded with the species independent settings according to table 6.5.

The results are presented in tables 6.11.A and 6.11.B.

Table 6.11.A. Descriptive statistics of graded azobé with settings from table 6.5.

Strength grade	n=	Yield (%)	Mean $p(i)$	5% fractiles of the bending strength		
				ranking (N/mm ²)	Central t- (N/mm ²)	Required value (N/mm ²)
D70	52	33.5	0.02	95.7	90.2	70
D50	97	62.6	0.02	68.5	67.7	50
D30	6	3.9	0.003	-	65.0	30
reject	0	0.0				-

Table 6.11.B. Descriptive statistics of graded azobé with settings from table 6.5

Strength grade	n=	Yield (%)	Mean MOE	Required value (N/mm ²)	5% fractiles of the density	
					Central t- (kg/m ³)	Required value (kg/m ³)
D70	52	33.5	23900	20000	990	900
D50	97	62.6	18800	14000	930	620
D30	6	3.9	15500	11000	810	530
reject	0	0.0				-

The tables show that when the species independent settings are used, 33% of azobé can be graded in D70. It is remarkable that also the observed 5% fractile of strength grade D50 also almost complies with D70, and the 5% fractile of D70 is even 95.7 N/mm², where 70.0 N/mm² is required. The explanation for this is that the variation in the distribution for the model values is much lower for azobé than for the entire dataset, but also for species such as cumaru and massaranduba. The reason for this could be that cumaru and massaranduba consist of a number of botanical tree species, where azobé consists of only one species.

Optimisation for one species can be done by using the distribution of the model values specifically for that species. In that case one must be sure that the available data is representative for the species (sufficient low strength pieces), which is not necessary for the species independent settings.

The distribution properties for batches AZ1, AZ2 and AZ3 are:

- $f_{m,a,model} : \mu = 104.1 \text{ N/mm}^2, \sigma = 14.7 \text{ N/mm}^2$
- $f_{m,measured} : \mu = 104.1 \text{ N/mm}^2, \sigma = 21.8 \text{ N/mm}^2$

However, with the assumed standard error, with these distribution properties it is not possible to lower the limit value for D70. The mean standard error of the azobé batches is

15.6 N/mm², where the calculated mean standard error for the whole dataset at the mean model value for azobé is 26.2 N/mm².

Optimisation for azobé is therefore possible when enough data for azobé is available to define a model with a lower standard error over the model line.

The fact that for azobé the standard error around the model line is lower can be explained from the fact that in the natural errors ε_f and ε_M also the errors in the factor C_1 and C_2 are included for the species independent strength models. When the model is optimized for one species, the errors in factors C_1 and C_2 will not be in the model and the standard error might decrease for a homogenous species.

For azobé, the standard deviation of the error found is:

$$\sigma_{m,\alpha,model} = 0.15f_{m,\alpha,model}$$

In table 6.12, the settings derived with the optimized parameters are listed.

Table 6.12. Settings for grade combination D70-D50-D30-reject optimized for azobé.

Strength class	$f_{m,stat}^{required}$ (N/mm ²)	$IP_{f_{m,low}}$ (N/mm ²)	Expected Yield (%)	$p(i),low$	$p(i),high$	p_{char}
D70	70.0	85.0	90.0	0.00	0.18	0.034
D50	50.0	63.0	10.0	0.005	0.15	0.020
D30	30.0	-	-	-	-	-
reject			0.0			

In table 6.13.A and 6.13.B, the grading results are listed. With the optimized settings more than 90% of the azobé pieces can be graded in D70 and further optimisation seems possible.

Table 6.13.A. Descriptive statistics of graded azobé with settings from table 6.12.

Strength grade	n=	Yield (%)	Mean $p(i)$	5% fractiles of the bending strength		
				ranking (N/mm ²)	Central t- (N/mm ²)	Required value (N/mm ²)
D70	139	90.0	0.04	75.6	71.7	70
D50	16	10.0	0.014	-	59.1	50
D30	-	-	-	-	-	-
reject	0	0.0				-

Table 6.13.B. Descriptive statistics of graded azobé with settings from table 6.12.

Strength grade	n=	Yield (%)	Mean MOE	Required value (N/mm ²)	5% fractiles of the density	
					Central t- (kg/m ³)	Required value (kg/m ³)
D70	139	90.0	20900	20000	950	900
D50	16	10.0	16100	14000	860	620
D30	-	-	-	-	-	-
reject	0	0.0				-

Further optimisation to D80 is possible. In table 6.14, the settings for grade combination D80-D60-D40 is given.

Table 6.14. Settings for grade combination D80-D60-D40-reject optimized for azobé

Strength class	$f_{m,stat}^{required}$ (N/mm ²)	$IP_{f_{m,low}}$ (N/mm ²)	Expected Yield (%)	$p(i),low$	$p(i),high$	p_{char}
D80	80.0	100.0	61.1	0.00	0.12	0.049
D60	60.0	74.0	37.1	0.006	0.17	0.029
D40	40.0	52.0	2.2	0.002	0.14	0.009
reject			0.0			

In table 6.15.A and 6.15.B the grading results are listed. With the optimised settings, more than 50% of the azobé pieces can be graded in D80.

Table 6.15.A. Descriptive statistics of graded azobé with settings from table 6.13.

Strength grade	$n=$	Yield (%)	Mean $p(i)$	5% fractiles of the bending strength		
				ranking (N/mm ²)	Central t- (N/mm ²)	Required value (N/mm ²)
D80	83	53.5	0.044	86.6	84.6	80
D60	71	45.8	0.031	66.8	65.4	60
D40	1	0.7	0.0034	-	-	40
reject	0	0.0				-

Table 6.15.B. Descriptive statistics of graded azobé with settings from table 6.12

Strength grade	$n=$	Yield (%)	Mean MOE	Required value (N/mm ²)	5% fractiles of the density	
					Central t- (kg/m ³)	Required value (kg/m ³)
D80	83	53.5	22600	22000	990	900
D60	71	45.8	17800	17000	910	700
D40	1	0.7	14800	13000	-	550
reject	0	0.0				-

In figure 6.18, the bending strength values are plotted against the model values for the dataset azobé. In figure 6.19, the observed and theoretical distributions for the graded pieces for grades D80 and D60 are shown.

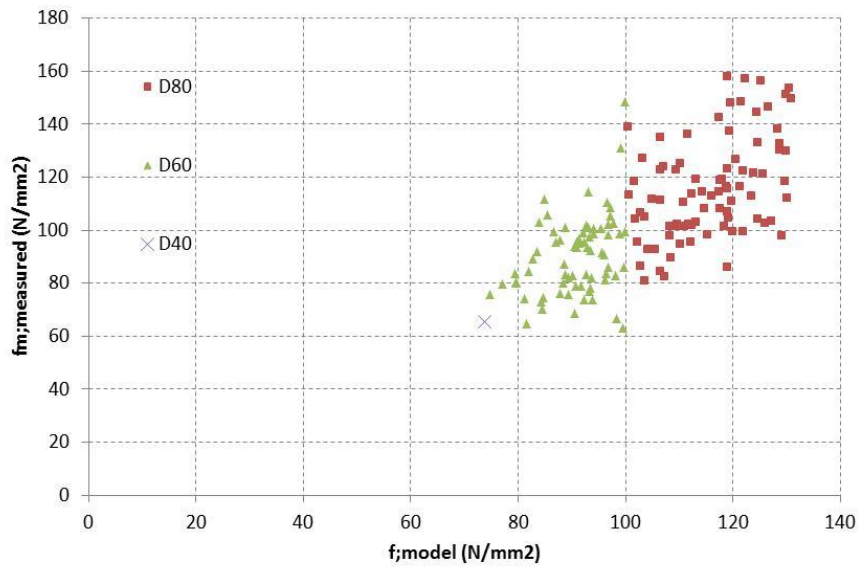


Figure 6.18. Bending strength values plotted against the model values for the dataset of azobé.

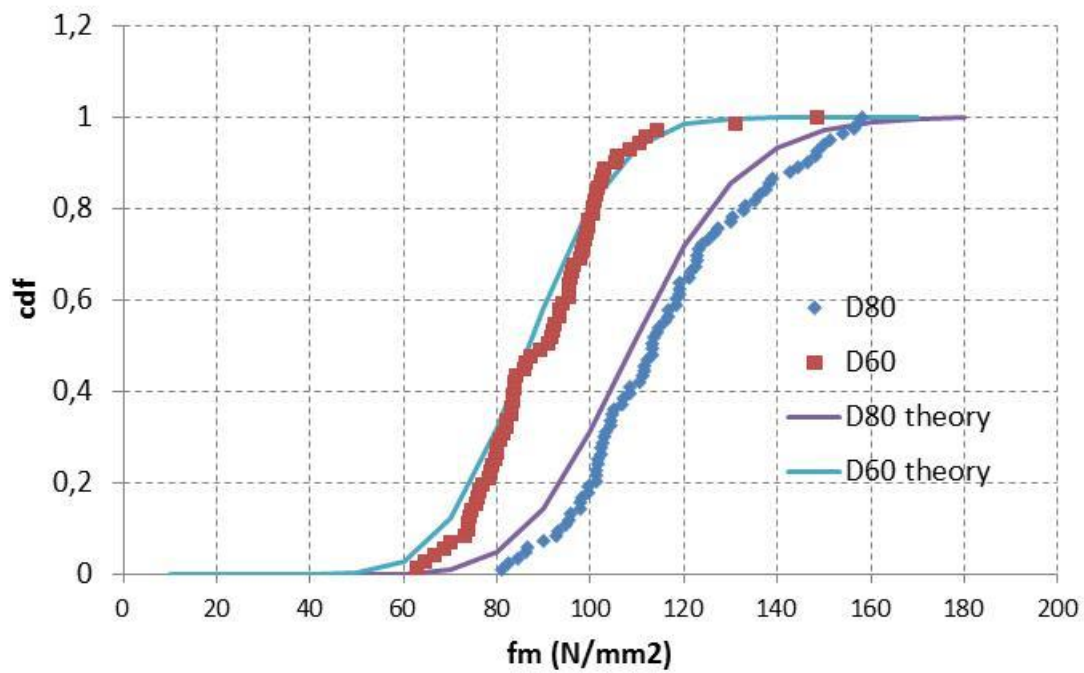


Figure 6.19. Cumulative distributions of bending strength of graded specimens of azobé. Observed distributions and theoretical distributions.

6.3.2 Species independent strength grading of the dataset of temperate hardwoods and softwoods

With the method derived in section 3.4, settings for machine grading for temperate hardwoods and softwoods can now be derived based on the model parameters derived in chapter 6.2:

$$f_{KR,mod} = 0.025\rho + 0.0031MOE_{KR} - 7.3$$

For the standard deviation of the error:

$$\sigma_{m,KR,model} = 0.12f_{m,KR,model} + 6.0$$

The intercept is rounded to 6.0.

The distribution properties are:

- $f_{KR,mod} : \mu = 45.3 \text{ N/mm}^2, \sigma = 10.1 \text{ N/mm}^2$
- $f_{m,measured} : \mu = 45.3 \text{ N/mm}^2, \sigma = 14.9 \text{ N/mm}^2$
- Relation between $f_{m,a,model}$ and $f_{m,measured}$:

$$f_{m,measured} = A f_{KR,mod} + B, \text{ where } A = 1.0 \text{ and } B = 0.0.$$

(in this case $f_{KR,mod} = IP_{fm}$ as defined in section 3.4).

The settings for the grade combination C40-C30-C18 according to the method, described in section 3.4, are determined. In table 6.16 the derived settings are listed.

Table 6.16. Settings for grade combination C40-C30-C18-reject for the whole sample of softwoods and temperate hardwoods

Strength class	$f_{m,stat}^{stat}$ required (N/mm ²)	IP_{fm} low (N/mm ²)	Expected Yield (%)	$p(i),low$	$p(i),high$	p_{char}
C40	40.0	59.0	8.8	0.00	0.073	0.046
C30	30.0	45.0	42.6	0.014	0.094	0.047
C18	18.0	27.0	37.3	0.03	0.165	0.045
reject			3.5			

In figure 6.20 the $p(i)$ -values for the range of the strength class ranges are given together with the distribution of the model values.

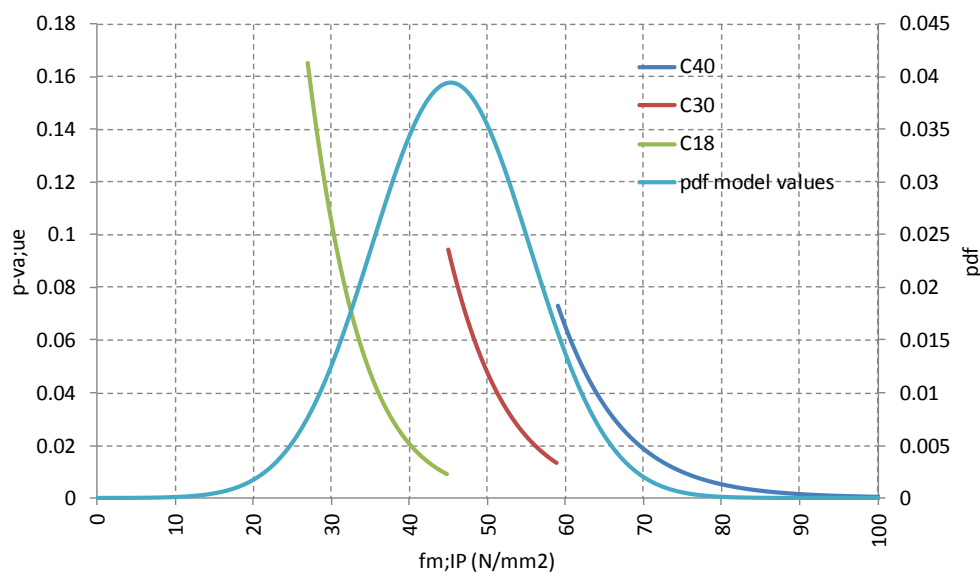


Figure 6.20. $p(i)$ -values for the settings from table 6.15.

In tables 6.17.A and 6.17.B the grading results are listed.

Table 6.17.A. Descriptive statistics of the graded dataset of temperate hardwoods and softwoods with settings from table 6.15.

Strength grade	$n=$	Yield (%)	Mean $p(i)$	5% fractiles of the bending strength		
				ranking (N/mm ²)	Central t- (N/mm ²)	Required value (N/mm ²)
C40	262	11.5	0.043	38.0	41.8	40
C30	883	38.9	0.047	31.0	30.6	30
C18	1104	48.6	0.038	19.4	19.3	18
reject	22	1,0				-

Table 6.17.B. Descriptive statistics of the graded dataset of temperate hardwoods and softwoods with settings from table 6.15.

Strength grade	$n=$	Yield (%)	Mean MOE	Required value (N/mm ²)	5% fractiles of the density	
					Central t- (kg/m ³)	Required value (kg/m ³)
C40	262	11.5	16700	14000	500	420
C30	883	38.9	13300	12000	410	380
C18	1104	48.6	10000	9000	330	320
reject	22	1,0				-

The mean $p(i)$ -values are below 0.05. The 5% fractiles for the bending strength are higher than the required values, except for the 5% fractile of C40 with the method of ranking. In figure 6.21, the theoretical distributions, calculated with the theory of section 3.4, are

plotted with the distributions of the destructive measurements. Figure 6.22 zooms in to the lower tail of figure 6.21.

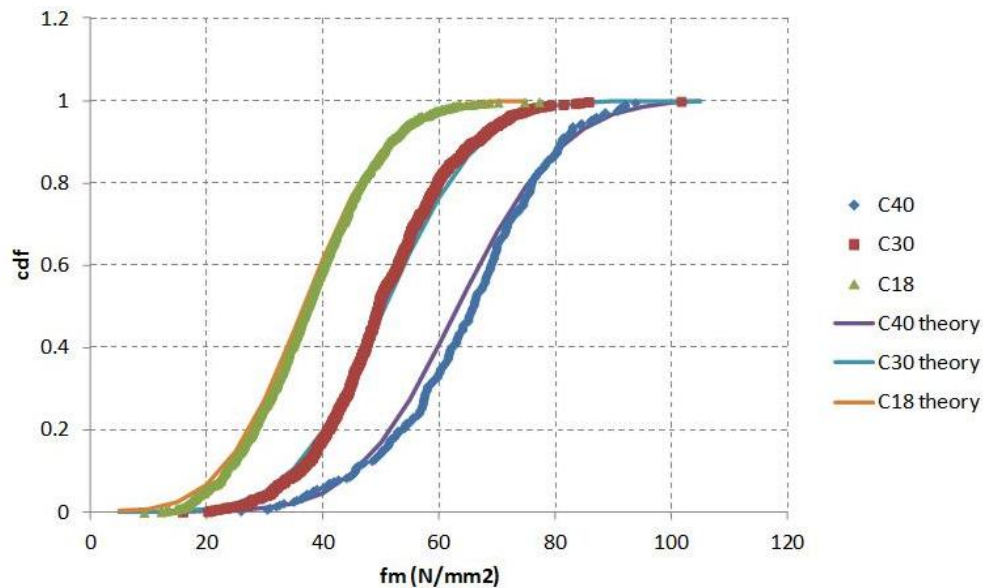


Figure 6.21. Cumulative distributions of bending strength of graded specimens of the datasets of softwoods and temperate hardwoods. Observed distributions and theoretical distributions.

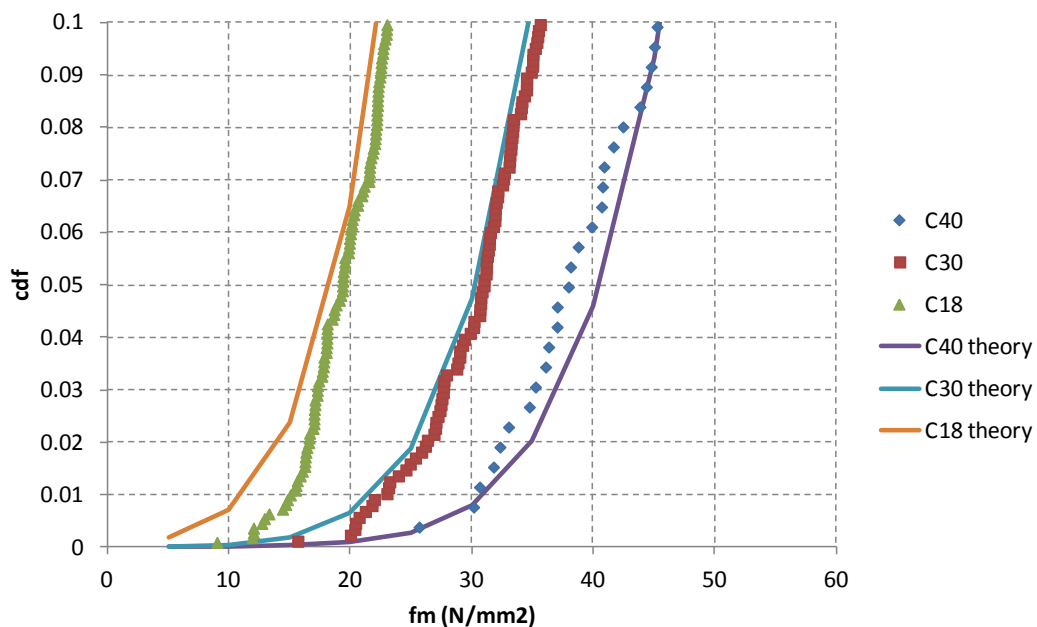


Figure 6.22. Cumulative distributions of bending strength of graded specimens of the datasets of softwoods and temperate hardwoods. Observed distributions and theoretical distributions. The figure zooms in on the lower tail of figure 6.21.

Both the destructive and the theoretical distributions can be very well approximated by normal distributions, and they practically overlap. For C40, slight differences with the theoretical distribution can be observed.

For C40, the 90% expected confidence interval is calculated and plotted in figure 6.23.

This figure shows that the destructive data practically falls into a 90% confidence interval, but the 5% fractile just outside this confidence interval. This can be explained by figure 6.24. In figure 6.24, the regression plot between the test values and the model values is given. From this plot the difference with the theoretical distribution can be explained by a “hole” in the data for C40. If this data would be present the observed distribution would shift to the right at the 5% fractile. It is exactly this influence of individual datapoints on the general trend that is intended to be avoided by the proposed method of section 3.4.

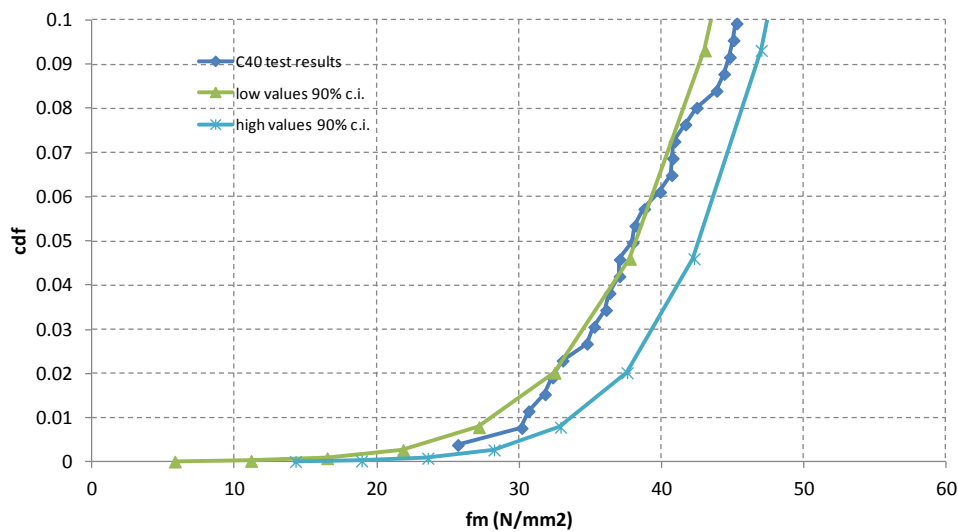


Figure 6.23. Cumulative distributions of bending strength of grade C40 for softwoods and temperate hardwoods. Observed distribution and 90% confidence interval for the theoretical distribution.

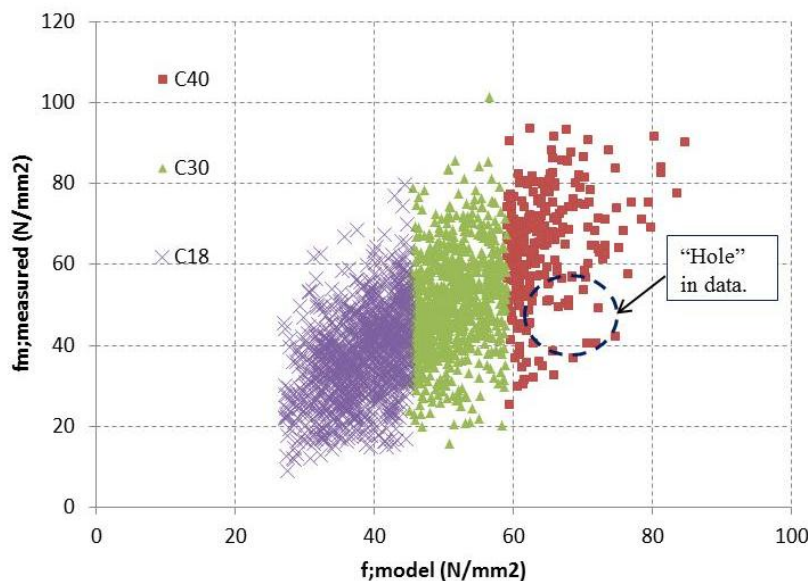


Figure 6.24. Regression plot of bending strength against model values for softwoods and temperate hardwoods. Less data is available in the circle.

The results from this section show that species independent grading is possible when the failure mode is similar for the pieces to be graded with the same prediction model. For temperate hardwoods and softwoods knots are the main failure inducing characteristics. Therefore, these can be graded with the same model in a species independent way.

Discussion and conclusions

Discussion

To be able to model the influence of strength reducing features, an assumption of the strength without these features (so called clear wood) has to be made. From literature the density was found to be the basic influencing parameter for the strength and stiffness of clear wood. This can be explained by the fact that the density is determined by the amount of cell wall material, which is related to the amount of fibers in the cell wall. The assumption is made that the strength and stiffness of a single fiber is the same for all wood species. This has not been verified by testing single fibers in this thesis, but by investigating the relationships between density with strength and with stiffness for clear wood from literature. A linear relationship was found between the density and strength and also between the density and the stiffness. The scatter around the mean regression line of strength against density is assumed to be the natural variation that cannot be explained at the clear wood level. The scatter (and the standard deviation around the mean model line) increased with increasing density, the coefficient of variation being constant. The same phenomenon was observed for the stiffness and the density. Because both the strength and the stiffness are linearly correlated with the density, the strength and stiffness for clear wood are also correlated to each other. The natural variation for the strength was found to be related to the natural variation of the stiffness, which has a positive effect on the correlation between strength and stiffness for clear wood.

Clear wood gives the maximum strength and stiffness for a piece of timber with a certain density. However, in structural timber, strength reducing features are present. These features can be divided into two groups: features that influence the reduction in strength and stiffness in such a way that the reduction can be predicted and features that reduce the strength and stiffness in an unpredictable way. The first group of features contains knots and grain angle deviation (slope of grain). The second group of features contains compression failures and large fissures. The first group of features can be used for grading structural timber depending on the level of their existence in a piece of timber. The second group of features must be limited or excluded in the timber to be graded. To be able to use the strength reducing features of the first group for visual grading, they must be

objectively measurable and repeated measurements must give the same output. It has been shown that for knots this is possible at an acceptable level, but that the visual measurement of grain angle deviation for tropical hardwoods cannot be performed accurately. To implement the fact that larger grain angle deviation might be present than can be visually determined, a very conservative factor $k_{s,tn}$ based on the number of subsamples for visual grading should be applied, which would give much lower strength values than has been experienced. Therefore, species independent strength grading is not possible for visual grading when grain angle deviation is governing. Investigating species independent grading for visual grading where knots are governing was not the objective of this thesis. Therefore, in this thesis species independent grading based only on machine grading of tropical hardwoods was investigated. For structural sizes ($h > 75$ mm) no influence of the height of the pieces on the bending strength is found. For the modulus of elasticity the influence of moisture content was quantified to be applied for both softwoods and hardwoods. For the bending strength of tropical hardwoods only the influence of moisture content was determined and quantified. The influence of the reduction on the strength and on the stiffness of the strength reducing features knots and grain angle deviation could be formulated with the use of structural mechanics. The equations to describe the reduction of the strength and of the stiffness when grain angle deviation is present are different from the equations to describe the reduction of the strength and of the stiffness reduction when knots are present. The basic reduction of strength and stiffness can be described by equations that are independent on the input distribution. These have to be determined experimentally. When machine strength grading is performed, the density and modulus of elasticity can be used as parameters to predict the bending strength of structural timber. The density and MOE take into account the influence of knots and grain angle deviation. For species independent grading prediction equations can be formulated. These equations include constants that are dependent on the input distribution of the material. These constants have been determined by performing a (non)-linear regression analysis. The prediction models for timber containing knots and for timber containing grain angle deviation are different, and also the scatter around the prediction lines differ. Thus, different prediction models are necessary for timber containing knots and for timber containing grain angle deviation. Species independent strength grading is therefore possible when a distinction is made between the occurrence of knots or of grain angle deviation as strength reducing characteristic.

To evaluate the result of the grading process the 5% fractile of the bending strength of the pieces to the assigned grades have to be determined. The problem with the current standardized methods is that they are very sensitive for the number of pieces in the assigned grades, and that they are very sensitive for the studied observation. To overcome this problem a statistical method was derived to determine the settings based on the model properties and the distribution of the prediction model values.

In practice the outcome of the research means that a division has to be made between softwood and temperate hardwoods (which normally contain knots and little grain angle

deviation) and tropical hardwoods (which normally contain grain angle deviation and only a very limited amount of knots). A visual override is therefore necessary to perform species independent machine strength grading. In cases where both knots and grain angle deviation is present a limit to the presence of knots has to be made to be able to use the prediction model based on the presence of grain angle deviation.

Conclusions:

- The influencing parameters for species independent grading are density, knot ratio and grain angle deviation from the beam axis.
- The knot ratio and the grain angle deviation from the beam axis are taken into account by machine measurements of the density and the modulus of elasticity.
- For tropical hardwoods only machine strength grading is suitable for species independent strength grading, because visual grain angle deviation measurements cannot be made accurately on timber not destructively tested.
- The basic constants in the equations describing the reduction of strength and stiffness of the characteristics knot ratio and grain angle deviation can be determined experimentally on samples with the same density and the magnitude of the characteristics evenly distributed.
- The species independent strength reducing equations can be derived from structural mechanics
- The constants in the species independent strength predicting models depend on the input distribution and can be determined by (non)-linear regression analysis.
- To be able to determine the settings (limit values) of the model values in the grading process, the scatter around the species independent strength predicting models can be estimated by determining the sliding standard deviation of the error over the range of the model values.
- The shape of the scatter of the species independent strength models for timber with knots as failure initiation characteristic and for timber with grain angle deviation as failure initiation characteristic differ. Therefore a distinction has to be made in species independent grading for timber with knots as prevalent failure initiating characteristic (normally softwoods and temperate hardwoods) and in species independent grading for timber with grain angle deviation as prevalent failure initiating characteristic (normally tropical hardwoods).
- To determine the settings a statistical method has been developed with the distribution of the model values, the strength predicting equation and the scatter predicting equation as input.
- Based on the characteristic values of the investigated samples of tropical hardwood species independent machine grading gives higher yields in the highest class than with visual grading.

- Whether visual assessment of grain angle deviation is possible, is questionable. In this research it has not been possible to predict by a visual assessment the grain angle deviation after the destructive test at an acceptable level. For machine strength grading the grain angle deviation is accurately incorporated by the modulus of elasticity.
- (species independent) machine strength grading takes into account the influence of knots and grain angle deviation, but not the influence of compression failures, fissures and other anomalies. Therefore a visual override is always necessary in combination with (species independent) machine strength grading.

The research question formulated in section 1.3 can be answered in short in the following way:

The influencing parameters for species independent grading models are the density, knots and grain angle deviation. They can be quantified by machine measurements of the density and the modulus of elasticity. To ensure safe and economical use of tropical hardwoods only machine grading is suited for species independent grading, but a visual override for strength reducing features that are not detected by the density and modulus of elasticity is necessary. Species independent machine strength grading of tropical hardwoods makes it possible to grade timber from different species coming in small amounts on the market in a safe and economical way.

Recommendations

The outcome of this research opens the possibility for species independent strength grading of structural timber. This has been verified for datasets of tropical hardwoods, temperate hardwoods and softwoods. It is recommended that the developed models are verified on other datasets by other researchers.

It should be made explicit in the standards for deriving settings for machine strength grading that species independent strength grading is allowed.

The input distribution of the strength prediction parameters determines the outcome of a regression analysis. The requirements for the input distributions for deriving settings for machine strength grading should be incorporated in the relevant standards.

The basic constants in the equations describing the reduction of strength and stiffness of the characteristics knot ratio and grain angle deviation can be investigated for species with different densities. In this case more insight in the scatter of this constants can be gained. That would provide more theoretical background for the developed theories.

A method was developed to determine the 5% fractiles of strength grades based on model properties. This method can replace the standardized methods for deriving the 5% fractiles based on pieces in a single grade, because these are very sensitive for the number of pieces in this grade and are the result of only one observation.

It is recommended to use species independent machine strength grading for tropical hardwoods in practice instead of visual grading because it gives higher yields in higher strength grades and it enlarges the reliability of the structures in which the timber is applied.

References

- Agnew, D.C., Constable C. (2008) 'Chapter 4. Multivariate random variables, correlation, and error propagation' Course notes SIO 223, [online], available: <http://igpphome.ucsd.edu/~cathy/Classes/SIO223A/sio223a.chap4.pdf> [accessed 9 July 2014].
- Anonymous (2006), '*JCSS Probabilistic model code.Part3: Resistance models-3.5 Timber*', JCCS, [online], available: http://www.jcss.byg.dtu.dk/~media/Subsites/jcss/english/publications/probabilistic_model_code/timber.ashx [accessed 9 July 2014].
- Armstrong, J. P., Skaar, C. and Zeeuw, d. C. (1984) 'The effect of specific gravity on several mechanical properties of some world woods', *Wood Science and Technology*, 18, 137-146.
- Bano, V., Arriaga, F. Soilan, A and Guaita, M. (2010). 'F.E.M. analysis of the strength loss in timber due to the presence of knots. ' *World Conference of Timber Engineering 2010*, June 20-24 2010, Riva del Garda, Italy.
- Barrett, J. D. and Hong, J.-P. (2009) 'Moisture content adjustments for dynamic modulus of elasticity of wood members', *Wood Science and Technology*, 44(3), 485-495.
- Blass, H.J. et al. (1995) *Timber Engineering STEP 1*, Almere: Centrum Hout.
- Bohannon, B. (1966) 'Effect of size on bending strength of wood members', *research paper FPL 56* [online], available: www.fpl.fs.fed.us/documnts/fplrp/fplrp56.pdf [accessed 9 July 2014].
- Box, G.E.P., Hunter, W.G. and Hunter, J.S. (1978), *Statistics for Experimenters* (New York: John Wiley & Sons, Inc.)
- Chui, Y.H., Turner, R. and Smith, I. (1991), 'Effect of sampling size on accuracy of characteristic values of machine grades'. *CIB W18, proceedings paper 24-17-3*, 1991, Oxford, United Kingdom.
- Denzler, J. K. (2007) *Modellierung des Grosseneffekt bei biegebeanspruchtem Fichtenschnittholz*, PhD-thesis, Technische Universität München.
- Dinwoodie, J. M. (2000) *Timber, its nature and behavior*, (London: E & FN Spon)

- Fewell, A. R. and Glos, P. (1988) 'The determination of characteristic strength values for stress grades of structural timber. Part 1.', in *CIB-W18 no. 21-6-2*, Parksville.
- Flink, P. (2012) *Een onderzoek naar de invloed van lengte, vochtgehalte en andere aspecten op de sterkte van naald- en loofhout*. Bachelor Thesis, Delft University of Technology (in Dutch).
- Freund, R.J. and Wilson, W.J.(1998). *Regression analysis: Statistical modeling of a response variable* (San Diego:Academic Press).
- Gerhards, C.C. (1975) 'Stress wave speed and MOE of sweetgum ranging from 150-15 percent MC', *Forest Products Journal*, (25), 4,51-57.
- Görlacher, R. (1990) *Klassifizierung von Brettschichtholzlamellen durch Messung von Longitudinalschwingungen*, PhD-thesis, Universität Karlsruhe.
- Govers, A. (1966) *Working stresses for a number of wood species*, Delft: Houtinstituut TNO.
- Green, D.W. and Evans, J.W. (1992) 'Moisture content adjustment procedures for engineering standards', in *CIB-W18/25-17-3*, Ahus.
- Green, D.W. and Evans, J.W. (2001) 'Evaluation of standardized procedures for adjusting lumber properties for change in moisture content', *report no. FPL-GTR-127* [online], available: www.fpl.fs.fed.us/documnts/fplgtr/fplgtr127.pdf [accessed 9 july 2014].
- Guttman, I. (1970) *Statistical tolerance regions: classical and bayesian*, London: Charles Griffin & Co.
- Gustavsson,L., Pingoud,K. and Sathre,R. (2006) 'Carbon dioxide balance of wood substitution: comparing concrete- and wood-framed buildings', *Mitigation and Adaptation Strategies for Global Change*, (11), 667-691.
- Hajela,D. (2008) 'Scientists to capture DNA of trees worldwide for database', *USA Today* [online], available: http://usatoday30.usatoday.com/tech/science/2008-05-02-890171068_x.htm [accessed 9 july 2014].
- Hanhijarvi, A., Ranta-Maunus, A. and Turk, G. (2005) *Potential of strength grading of timber with combined measurement techniques - report of the Combigrade project phase 1.VTT publication no. 568* Espoo: VTT.
- Hankinson.,R.L., (1921) 'Investigation of crushing strength of spruce at varying angles of grain.', *U.S. Air Service* , Inform. Cir.3 no.259.
- Hek, C. (2014) *De druksterkte afhankelijk van de vezelrichting en de druksterkte loodrecht op de vezelrichting in verhouding tot de dichtheid van hardhout*. Bachelor Thesis, Delft University of Technology (in Dutch).

- Hoffmeyer, P. (1987) 'The role of grain angle, knots, tension wood, compression wood and other anomalies on the mechanical properties of wood', Technical University of Denmark, *Technical report no. 183/87*[online], available: http://orbit.dtu.dk/fedora/objects/orbit:83427/datastreams/file_5288446/content [accessed 9 july 2014].
- Houtinstituut TNO (1921) *Investigation of crushing strength of spruce at varying angles of grain*, vol 3. no. 259, Engineering Division, Air serice, Dayton, Ohio.
- Houtinstituut TNO (1961) *Buig- en drukproeven met tola branca van handelskwaliteit en afmetingen en bijbehorend foutvrij hout*. Rapport H-62-145. Houtinstituut TNO. Delft. (in Dutch)
- Houtinstituut TNO (1961) *Buig- en drukproeven bij iroko van handelskwaliteit en afmetingen en bijbehorend foutvrij hout*. Rapport H-61-147. Houtinstituut TNO. Delft. (in Dutch)
- Houtinstituut TNO (1961) *Buig- en drukproeven bij yang van handelskwaliteit en afmetingen en bijbehorend foutvrij hout*. Rapport H-61-176. Houtinstituut TNO. Delft. (in Dutch)
- Houtinstituut TNO (1962) *Buig- en drukproeven peroba de campos van handelskwaliteit en afmetingen en bijbehorend foutvrij hout*. Rapport H-62-146. Houtinstituut TNO. Delft. (in Dutch)
- Houtinstituut TNO (1963) *Buig- en drukproeven met basralocus van handelskwaliteit en afmetingen en bijbehorend foutvrij hout*. Rapport H-63-3. Houtinstituut TNO. Delft. (in Dutch)
- Houtinstituut TNO (1963) *Toelaatbare spanningen voor zeventien houtsoorten*. Rapport H-63-51. Houtinstituut TNO. Delft. (in Dutch)
- Kollmann, F. F. P. and Cote, W. A. (1968) *Principles of Wood Science and Technology*, Heidelberg: Springer-Verlag.
- Lambrechts, C., Wilkie, M.L., Rucevska, I. and Sen, M. (2009) 'Vital Forest Graphics', *UNEP,FAO,UNFF report no. DEW/1032/NA* [online], available: http://www.grida.no/files/publications/vital_forest_graphics.pdf [accessed 9 july 2014].
- Leknitskij, S.G. (1981) *Theory of Elasticity of an Anisotropic Body*. (Moscow. Mir publishers) .
- Mohler, K. (1977) 'Consideration of combined stresses for lumber and glued laminated timber', in *CIB-W18 paper no. 9-6-4*, Stockholm,

- Mohler, K. (1979) 'Consideration of combined stresses for lumber and glued laminated timber (addition to paper CIB-W18/9-6-4)', in *CIB-W18 paper no. 11-6-3*, Vienna.
- Norris, C. B. (1962) 'Strength of orthotropic materials subjected to combined stresses', report no. 1816 [online], available: www.fpl.fs.fed.us/documnts/fplmisc/rpt1816.pdf [accessed 9 July 2014].
- Pellicane, P.J. and Bodig, J. (1981) Sampling error in the bending strength distribution of dimension lumber. *Wood Science and Technology*, 15, 211-225.
- Pope, D.J., Marcroft, J.P. and Whale, L.R.J. (2005) 'The effect of global slope of grain on the bending strength of scaffold boards', *Holz als Roh- und Werkstoff*, (63), 321-326.
- Ranta-Maunus A, Turk G (2010). 'Approach of dynamic production settings for machine strength grading.' *World Conference of Timber Engineering 2010*, June 20-24 2010, Riva del Garda, Italy.
- Ranta-Maunus, A. (2012) 'Determination of settings in combined strength, stiffness and density grading of timber.' *European Journal of Wood and Wood Products*, 70:883-891.
- Ravenshorst, G.J.P. et al.. (2004) 'An economic method to determine the strength class of wood species', *HERON*, vol. 49, No.4,297-326.
- Ravenshorst, G.J.P. and Van de Kuilen, J.W.G (2008) A method for quality control of strength grading machines using non-destructive measurements. *Cost Action E53 conference*, October 29-30 2008 Delft, The Netherlands.
- Ravenshorst, G.J.P. and Van de Kuilen, J.W.G (2009). Relationships between local, global and dynamic modulus of elasticity for soft- and hardwoods *CIB W18, proceedings paper 42-10-1*, Dubendorf, Switzerland.
- Ravenshorst, G.J.P. and Van de Kuilen, J.W.G. (2010) Strategies for quality control of strength graded timber. *Cost Action E53 conference*, October 4-7 May 2010 Edinburgh, UK.
- Ravenshorst ,G.J.P., de Vries,P.A. and Van de Kuilen, J.W.G. 'Aspects of the difference between the local and global modulus of elasticity of structural (hardwood) timber.' in *Materials and Joints in Timber Structures, Rilem Bookseries vol. 9 (773-783)*, Springer

- Ravenshorst, G.J.P. and Van de Kuilen, J.W.G. (2014) 'A new approach for the derivation of settings for machine graded timber', *Wood Material Science & Engineering*, published online 29 September 2014, DOI:10.1080/17480272.2014.950329.
- Rey, A. (2012) *Deinvloed van vochtgelate en visuele kenmerken op de buigsterkte van hardhout*. Bachelor Thesis, Delft University of Technology (in Dutch).
- Riberholt, H. (1979) 'Tapered and curved glulam members', in *CIB W18 paper no. 11-10-1*, Vienna,
- Riberholt, H. and Madsen (1979) '*Strength of timber structures, measured variations of the cross sectional strength of structural lumber*'. Structural Research Laboratory, Technical University of Denmark.
- Rijsdijk, J. F. and Laming, P.B. (1994) *Physical and Related Properties of 145 timbers* (Dordrecht: Springer Science + Business Media)
- Ross, R. J. et al. (2010) 'Wood Handbook. Wood as an engineering material', *FPL-GTR-190* [online], available: www.fpl.fs.fed.us/documnts/fplgtr/fpl_gtr190.pdf [accessed 9 July 2014].
- Rouger, F. (1995) , 'Volume and stress distribution effects, STEP lecture B1' in *Timber Engineering, STEP 1*, Almere: Centrum Hout.
- Schajer, G.S. (2001) 'Lumber strength grading using X-ray scanning', *Forest Products Journal*, (51), 1,43-50.
- Scheffé, H. and Tukey, J. W. (1944) 'A formula for sample sizes for population tolerance limits', *The Annals of Mathematical Statistics*, (15), 217.
- Sclove, S.L.C., (2005) 'Concerning the sample standard deviation', University of Illinois at Chicago,[online], available: <http://www.uic.edu/classes/idsc/ids571/samplvar.pdf> [accessed 9 July 2014].
- Smith, I., Landis, E. and Gong, M. (2003) *Fracture and fatigue in wood*, Chichester: J.Wiley.

- Sørensen, J.D. and Hoffmeyer, P. (2001) Statistical analysis of data of timber strengths. Paper No.206, Aalborg University. [online], available at <http://vbn.aau.dk/ws/files/36371485/Statistical%20Analysis%20of%20Data%20for%20Timber%20Strengths.pdf> [Accessed 10 April 2013].
- Stapel, P., Van de Kuilen, J.W.G. and Ravenshorst, G.J.P. (2011). Influence of sample size on assigned characteristic strength values. *CIB W18, proceedings paper 44-17-1*, 2011, Alghero, Italy.
- Stapel, P. and Van de Kuilen, J.-W. G. (2013) 'Influence of cross-section and knot assessment on the strength of visually graded Norway spruce', *European Journal of Wood and Wood Products*, 72(2), 213-227.
- Thelandersson, S. and Larsen, H. J. (2003) *Timber Engineering*, Chichester: J. Wiley.
- Unterwieser, H. and Schickhofer, G. (2011) 'Influence of moisture content on sound velocity and dynamic MOE of natural frequency- and ultrasonic runtime measurement', *European Journal of Wood and Wood Products*, 69, 171-181.
- Van Benthem, M and Bakker, B. (2011) 'Lesser known species are part of it too', *Probos bosberichten* (7) [online], available: www.europeansttc.com/wp-content/uploads/2014/03/bosberichten2011-07Engels.pdf [accessed 9 July 2014].
- Van de Kuilen, J.W.G. and Leijten, A. (2002) *Schuifsterkte bepaling van zeven houtsoorten voor de toepassing in verkeersbruggen*. Report 2001-4/HE43. Delft University of Technology (in Dutch).
- Van de Kuilen, J.W.G. and Blass, H.J. (2005). Mechanical properties of azobé (*Lophira alata*). *Holz als Roh- und Werkstoff*, 63.1-10.
- Van de Have, R. (2013) *De invloed van de vezelrichting op de mechanische eigenschappen van hout*. Bachelor Thesis, Delft University of Technology (in Dutch).
- Van der Poel, D. (2012). *Het voorspellen van de E-local, E-global en E-dynamisch voor hout met verschillende visuele kenmerken*. Bachelor Thesis, Delft University of Technology (in Dutch).
- Van Otterloo, J. (2010). *De invloed van vochtgehalte, afmetingen en defecten op de sterkte van naald- en loofhout*. Bachelor Thesis, Delft University of Technology (in Dutch).

- Vocht, A. De (2000). *Basishandboek SPSS 10* (Utrecht: Bijleveld press, in Dutch).
- Vrouwenvelde, A.V. (2008). 'EN 1990. Basis of Design. Annex B Reliability Differentiation. Annex C. Reliability theory.' Presentation. *Workshop Eurocodes*, February 18-20 2008, Brussels, Belgium.
- Wagenfuhr, R. (2007) *Holzatlas*, Leipzig: Fachbuchverlag Leipzig.
- Weibull, W. (1939) *A statistical theory of the strength of materials*, Royal Technical University, Stockholm.
- Wilks (1941) 'Determination of sample sizes for settings tolerance limits', *The Annals of Mathematical Statistics*, (12), 91-96.
- Winterink, A. (2013) 'Resultaten VVNH monitoring rapportageformulier eerste helft 2013', *Probos* [online], available: [accessed 9 July 2014].
- Wiselius, S. I. (2010) *Houtvademecum*, The Hague: Sdu Uitgevers.
- Ziethen, R., Bengtsson, C. (2010). 'Machine strength grading-prediction limits-evaluation of a new method for derivation of settings. ' *World Conference of Timber Engineering 2010*, June 20-24 2010, Riva del Garda, Italy.
- Ziethen, R. Bengtsson, C. (2011), Initial settings for machine graded structural timber. *CIB W18, proceedings paper 44-5-3*, 2011, Alghero, Italy.

Standards

- ASTM D143 (2009): *Standard test methods for small clear specimens of timber*. West Conshohocken. ASTM
- ASTM D2915 (2003): *Standard practice for evaluating allowable properties for grades of structural lumber*. West Conshohocken. ASTM
- BS 5756 (2007). *Visual grading of hardwood-Specification- Determination of characteristic values of mechanical properties and density*. BSI..
- prEN 338 (2014). *Structural Timber- Strength classes*. Brussels. CEN.

- EN 384 (2010). *Structural Timber- Determination of characteristic values of mechanical properties and density*. Brussels. CEN.
- EN 408+A1 (2012). *Timber Structures-Structural Timber and Glued Laminated timber- Determination of some physical and mechanical properties*. Brussels. CEN.
- EN 1912 (2012). *Structural Timber - Strength classes - Assignment of visual grades and species*. Brussels. CEN.
- EN 13183-1. (2002). *Moisture content of a piece of sawn timber -Part 1. Determination by oven dry method*. Brussels. CEN.
- EN 14081-1+A1:. (2011). *Timber Structures-Strength graded structural timber with rectangular cross section-Part 1. General requirements*. Brussels. CEN.
- EN 14081-2 +A1:. (2012). *Timber Structures-Strength graded structural timber with rectangular cross section-Part 2:Machine grading-additional requirements for initial type testing*. Brussels. CEN.
- EN 14081-4:. (2009). *Timber Structures-Strength graded structural timber with rectangular cross section-Part 4: Grading machine settings for machine controlled systems*. Brussels. CEN.
- EN 14358 (2007). *Timber Structures- Calculation of characteristic values and acceptance criteria of a sample*. Brussels. CEN.
- INSTA 142 (2009). *Nordic visual grading rules for timber*. NS.
- N 1012 (1927). *Keuringsvoorschriften voor hout als bouw materiaal en voorschriften voor houtbereiding*. (In Dutch)
- N 1012 (1932). *Keuringsvoorschriften voor hout als bouw materiaal en voorschriften voor houtbereiding*. (In Dutch)
- N 1012 (1940). *Keuringsvoorschriften voor hout als bouw materiaal en voorschriften voor houtbereiding*. (In Dutch)
- N 1055 (1955). *Technische grondslagen voor bouwvoorschriften TGB 1955*. Den Haag, CNB (In Dutch)

NEN 3852 (1973). *Technische grondslagen voor bouwvoorschriften TGB 1972*. Delft, NEN (in Dutch)

NEN 5493 (2010). *Quality requirements for hardwoods in civil engineering works and other structural applications*. Delft. NEN.

NEN 5498 (1991). *De bepaling van de karakteristieke waarde van de buigsterkte en de elasticiteitsmodulus evenwijdig aan de vezelrichting en de volumieke massa van gezaagd hout*. Delft. NEN.

NEN 5499 (2007). *Requirements for visually graded softwoods for structural applications*. Delft. NEN

NEN 6760 (1991). *Technische grondslagen voor bouwvoorschriften TGB 1990*. Delft, NEN (in Dutch)

NEN-EN 1990 (2002) . *Eurocode. Basis of structural design*. Delft. NEN.

NEN-EN 1995-1-1 (2005) . *Eurocode 5. Design of timber structures - Part 1-1: General-Common rules and rules for buildings*. Delft. NEN.

ISO/CD 12122-1 (2012). *Structural timber products-Determination of characteristic values- part 1. Basic requirements*. Geneva. ISO

Websites

www.fsc.org Forest Stewardship Council.

www.pefc.org Program for the Endorsement of Forest Certification

www.tpac.smk.nl Timber Procurement Assessment Committee

www.europeansttc.com European Sustainable Tropical Timber Coalition

Appendix A.

Regression analysis

There is a basic difference between physical prediction models and prediction models based on regression analysis. The main difference is the causal relationship between model parameters and measured data. In physical models, relationships are built based on theoretical relationships for which it is clear which causes changes in another property. For instance, the presence of knots reduces the section modulus and therefore the strength. In regression models, an empirical relationship is found between the predicting parameters where there does not have to be a causal relation. An example is the moisture content that could be used as predicting parameter. We could find a reduction of strength between 12% m.c. and 30 % m.c. So we could conclude that more water in the timber causes a reduction of strength. However, the presence of the amount of water in the timber does not cause the reduction, but the water absorption of the fibers does. We know that above 30% m.c. no further reduction of strength takes place. This is because the fibers are then fully saturated. The cause of the strength reduction is the amount of water taken up by the fibers, not by the wooden piece. For the fibers, there is an ongoing reduction of strength from dry to saturated. Of course one could go a step further and ask what the cause of strength reduction of the fibers by water absorption is. That means that there is a level where a basic empirical relationship between properties is assumed as the start for the physical modelling.

In this thesis, the start for the physical model is the basic empirical relationship that the cell wall thickness, and related to that the density at macroscopic level, is the basis of the strength and stiffness of timber. Based on literature and own experiments, a linearly increasing relationship between the density and strength and stiffness is assumed with an increasing standard error over the range of the density. The value of the factor describing this relationship can then be determined with a regression analysis.

The approach in this thesis is to describe the strength and stiffness properties based on physical models and determine the factors in the relationships by regression analysis. To perform the regression analysis, the method of ordinary least squares (OLS) and weighted least squares will be used (WLS).

Ordinary Least Squares (OLS)

Consider a linear regression between x and y :

$$y = \beta_0 + \beta_1 x + \varepsilon$$

This line predicts a mean value $\bar{y} = \beta_0 + \beta_1 x$ with a normally distributed error term $N(0, \sigma_\varepsilon)$. There are n observations for x and y . y is the dependent parameter and x the independent parameter

σ_ε is calculated as

$$\sigma_\varepsilon = \sqrt{\frac{\sum(\bar{y}_i - y_i)^2}{n - 1}}$$

When an ordinary least squares analysis is performed, the coefficient values are found in the following way:

Matrices X , Y , B and E can be written as:

$$X = \begin{bmatrix} 1 & x_1 \\ 1 & x_i \\ 1 & x_n \end{bmatrix} \quad Y = \begin{bmatrix} y_1 \\ y_i \\ y_n \end{bmatrix} \quad B = [\beta_0 \quad \beta_1] \quad E = \begin{bmatrix} \varepsilon_1 \\ \varepsilon_i \\ \varepsilon_n \end{bmatrix}$$

Where:

- Y is the response variable (matrix $1 \times n$)
- X is the matrix with independent variables (matrix $n \times m$), X^T is the transposed matrix of X and $(XX^T)^{-1}$ is the inverse matrix of XX^T
- B is the matrix with the coefficients (matrix $m \times 1$)
- E is the matrix with error terms (matrix $1 \times n$). E is assumed to be normally distributed with equal variance over the range of Y .

The linear regression can be written in matrix notation: $Y=BX+E$ and the error term as $E=Y-BX$

A closed form solution for matrix B is found under the condition that the sum of EE^T is minimized:

$$\min = \sum (\bar{y}_i - y_i)^2$$

The closed form solution for B in matrix notation is:

$$B = (XX^T)^{-1}XY$$

The solution for the variances of B is:

$$\text{var}(B) = (XX^T)^{-1}\sigma_\varepsilon$$

Which can be written as:

$$\text{var}(\beta_0) = \frac{\sigma_\varepsilon^2 * \sum X_i^2}{n \sum (X_i - \bar{X})^2} = \sigma_{\beta_0}^2$$

$$\text{var}(\beta_1) = \frac{\sigma_\varepsilon^2}{\sum (X_i - \bar{X})^2} = \sigma_{\beta_1}^2$$

The regression coefficients β_0 and β_1 are randomly normally distributed variables $N(\beta_0, \sigma_{\beta_0})$ and $N(\beta_1, \sigma_{\beta_1})$. These values for β and σ_β are given by programs like SPSS.

When in this thesis correlation graphs are shown, without further comment, they are made with the spreadsheet program Microsoft Excel. The trendlines and coefficients of determination (r^2 -values) are determined by Excel by using the least squares technique.

For non-linear models there is no closed form solution for matrix B under the condition that EE^T is minimized. An iterative approach is necessary where the optimization target can also be the least squares estimate. In that case, the matrix B is iteratively updated until it has converged to a solution. In Excel the built-in solver is equipped for this purpose. In Fylstra et al. (1998) the backgrounds for Excel solver are described. With Excels' solver, constraints can be formulated that are integrated in the optimization process. In optimizing non-linear models the variances of B cannot be estimated as with linear models. A solution for this is for instance bootstrapping, where the variance can be estimated. This is integrated in the statistical program SPSS. The formulas for error propagation can also be used for non-linear models.

Weighted Least Squares (WLS)

When the error terms are uncorrelated as in the OLS, but there is no equal variance of the error, then a weighted regression might be suitable to estimate the variances of the matrix B . This is useful when the error shows a linearly increasing variance over the range of the prediction values. In that case, the matrix V is used, which is the variance-covariance matrix:

$$V = \sigma_\varepsilon^2 I_n$$

When the errors are independent with equal variance I_n is an $n \times n$ identity matrix. When the errors are independent but have unequal variance then the off diagonal elements of I_n will still be zero, but the diagonal elements will not have the same value. The solution for matrix B then becomes:

$$B = (XV^{-1}X^T)^{-1}XV^{-1}Y$$

The inverse of V contains the reciprocal of the variances $1/\sigma_\varepsilon^2$. This means that the products of the other matrices are weighted by the reciprocals of the variances of the individual observations, that is why the method is described as weighted regression.

The variances for B are then

$$\text{var}(B) = (XV^{-1}X^T)^{-1}\sigma_{\varepsilon,w}$$

Where $\sigma_{\varepsilon,w}$ is the error term (now with equal variance) of the weighted regression. Of course the variances in the error terms are not known before the weighted regression takes place.

Appendix B.

Visual examination of tropical hardwoods

Visual grading according to standard NEN 5493.

For visual grading of tropical hardwoods the Dutch standard NEN 5493 gives requirements. Other countries have their own standards like the BS 5756 in the United Kingdom. The requirements however are very similar. In this thesis the visual grade C3 STH according to NEN 5493 is used to visually grade the pieces of tropical hardwoods.

In contrary to softwoods and temperate hardwoods, there is only one visual grade defined for tropical hardwoods. This is caused by the fact that for softwoods and temperate hardwoods knots are an important grading parameter, which is very suited to be quantified in such a way that different limits can be defined. Because for tropical hardwood coming presently on the market knots are very rare this feature cannot be used to distinguish different classes, there is only a requirement formulated to limit the occurrence of this feature.

In principle it would be possible to define different classes for the grain angle deviation (slope of grain) but as explained in chapter 4, the measurement of this feature is that difficult that this is not possible. As a result only one visual grade can be defined for tropical hardwoods. For timber with a certain trade name that meets the requirements for the visual grade the characteristic values have to be determined. Based on these characteristic values the strength class can be determined to which the pieces with that trade name that are visually graded may be assigned to.

The visual grade C3 STH gives requirements that are not only strength related but also application related. In this thesis only the requirements related to strength will be discussed.

The requirements according to visual grade C3 STH of NEN 5493 related to strength are:

- The knot ratio must be lower than 0.2.
- The grain angle deviation (defined as slope of grain) must lower than 1:10.
- No compression failures are allowed.
- Fissures: surface checks are allowed. Only small (100 mm long) face shakes and end shakes are allowed (no fissure through the thickness, see EN 14081-1)
- No wane is allowed
- No holes in the pieces.
- No big other anomalies.

In section 3.2.2. the methods for measuring the visual characteristics are explained.

Pieces rejected because of visual override in combination with species independent machine grading.

In this thesis species independent strength grading is developed. Grain angle deviation (slope of grain) was identified as the main strength reducing parameter for tropical hardwoods, and it was found that this parameter could be integrated in machine strength grading by measuring the MOE. In combination with species independent strength grading grain angle deviation (slope of grain) does not have to be examined visually.

Because it was found that when knots are the feature that causes failure a different predicting model has to be used for machine strength grading, this feature has to be limited in combination with machine strength grading. When the knot ratio is lower than 0.2 the influence of knots is lower than the influence of grain angle deviation, therefore this limit is kept as a requirement in the visual override.

A visual override means that every piece that is machine graded has to be examined visually on certain characteristic that cannot be detected by the machine measurements.

The requirements for the visual override in combination with machine grading for tropical hardwoods are:

- The knot ratio must be lower than 0.2.
- No compression failures are allowed.
- No large fissures are allowed: The requirements according to EN 14081-1 for the visual override are used. Fissures not going through the thickness may be not greater than 1 m or $\frac{1}{4}$ the length of the piece, whichever is lesser. Fissures going through the thickness are only permitted at the ends with a length not greater than the width of the piece.
- No wane is allowed
- No holes in the pieces.
- No big other anomalies.

Pieces that do not meet the requirements for these visual examinations should be removed from the grading process.

43 pieces from the entire dataset of $2218 + 43 = 2261$ pieces of the database of tropical hardwoods (1.9 %) had to be removed from the grading process because of the visual override. In table B1 a number of these pieces are listed for which photographs are available. The reason for removal because of the visual override is given. Compression failures, fissures going through the thickness and large knots are the main reasons for removal.

The pieces removed from the grading process were still tested to obtain insight in their influence. In figures B.1 and B.2 the effect of the influence of the visual override is given

for the model according to equation (6.4). The figures show that when the pieces would not have been removed the scatter from the model line would increase considerable due to these pieces. The developed models cannot accurately predict the scatter from the model line for this pieces. Figures B.1 and B.2 show that with the current used machine measuring techniques a visual override is necessary.

Table B1. Selection of pieces removed from the grading process due to the visual override.

Nr.	Trade name	Reason for removal based on visual override	Photograph
1	angelim vermelho	Compression failure	
2	angelim vermelho	Knot ratio >0.2	
3	angelim vermelho	Knot ratio >0.2 in combination with compression failure	
4	greenheart	Big hole in the piece	

5 greenheart Compression failure



6 cumaru Compression failure



7 cumaru Compression failure



8 cumaru Fissures going through the thickness.



9 cumaru Compression failure



10 tauro vermelho Compression failure



11 tauro vermelho Compression failure



12 tauro vermelho Resin pockets and fissures



13 tauro vermelho Compression failure



14 tauro vermelho Compression failure



15 bangkirai Knot ratio >0.2



16 bangkirai Fissure through the thickness at the beam end



17 karri Knot ratio >0.2



18 karri Knot ratio >0.2



19 basralocus KR >0.2



20 basralocus Compression failure



21 basralocus fissures through the thickness



22 basralocus Knot ratio >0.2



23 nargusta Compression failures



24 tali Knot ratio >0.2



25 tali Knot ratio >0.2



26 tali Knot ratio >0.2



27 bilinga Knot ratio >0.2



28 bilinga Knot ratio >0.2



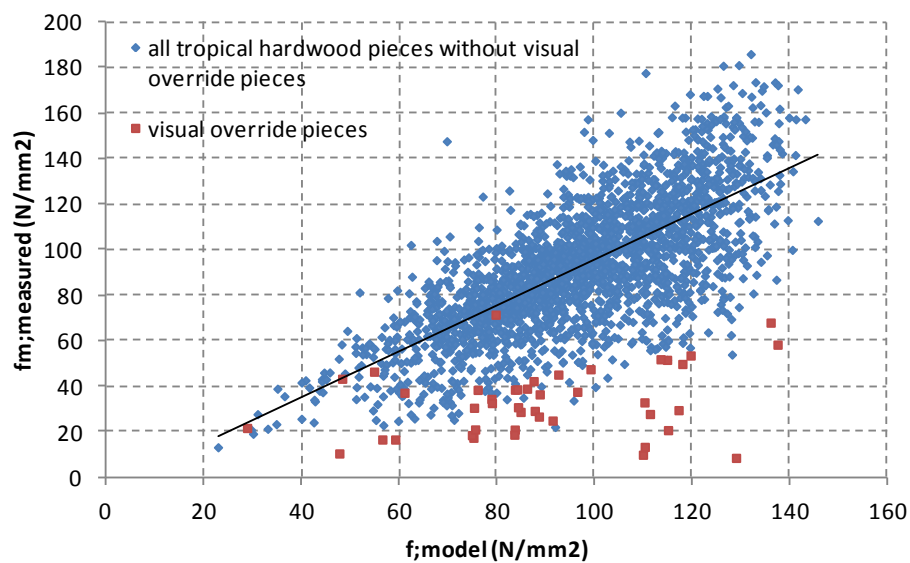


Figure B.1. Regression plot of bending strength against model values for tropical hardwoods without visual override pieces and for the tropical hardwood visual override pieces.

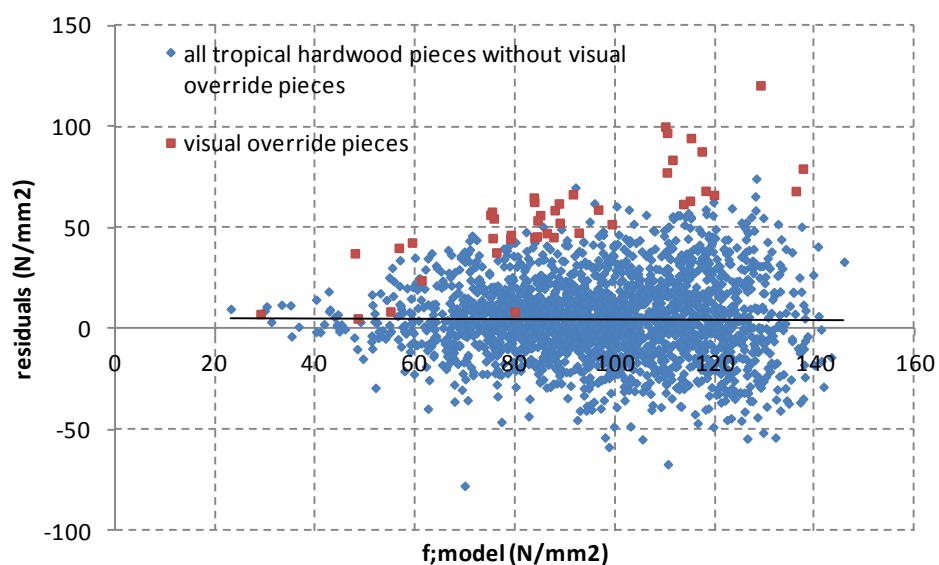


Figure B.2. Regression plot of residuals against model values for tropical hardwoods without visual override pieces and for the tropical hardwood visual override pieces.

Appendix C.

Strength class profiles

according to prEN 338

(2013)

Table C.1. Characteristic values for softwood strength classes at 12 % m.c. according to prEN 338.

C-class	$f_{m,k}$	MOE_{mean}	$density_k$
16	16	8000	370
18	18	9000	320
24	24	11000	350
30	30	12000	380
35	35	13000	390
40	40	14000	400
45	45	15000	410
50	50	16000	430

Table C.2. Characteristic values for hardwood strength classes at 12 % m.c. according to prEN 338.

D-class	$f_{m,k}$	MOE_{mean}	$density_k$
18	18	9500	475
24	24	10000	485
27	27	10500	510
30	30	11000	530
35	35	12000	540
40	40	13000	550
45	45	13500	580
50	50	14000	620
55	55	15500	660
60	60	17000	700
65	65	18500	800
70	70	20000	900
75	75	22000	900
80	80	24000	900

Appendix D.

Test programs

The properties of the datasets listed in tables 3.1, 3.2 and 3.3 were determined in several test programs. The commissioners and the laboratories where the samples were tested are:

The tropical hardwood species were tested under the commission of:

- The Netherlands Timber Trade Organisation (VVNH)/ Rijkswaterstaat (part of the Dutch Ministry of Infrastructure and the Environment): CUM1, MAS1, AZ1, AZ2, AZ3, GR1, GR2, GR3, GR4, OK1, OK2, OK3, KA1, NA1, PI1, BAS1, BAN1, BIL1, BIL2, EV1, EV2, TA1, TA2.
- Precious Woods Europe: AV2, AV3, AV4, AV5, CUM2, CUM3, CUM4, CUM5, MAS2, MAS3, MAS4, MAS5, SV1, CR1, LA1, LF1, PU1, TV1, FA1, SA1, FP1.

The tropical hardwood species were tested at the laboratories of:

- Delft University of Technology: MAS2, MAS3, MAS4, MAS5, CUM2, CUM3, CUM4, CUM5, GR2, GR3, GR4, OK2, OK3, BIL1, BIL2, EV1, EV2, TA1, TA2.
- TNO: CUM1, MAS1, AZ1, GR1, OK1, KA1, PI1, VI1, BAS1, BAN1, AV2, AV3, FA1, SA1, FP1.
- CNR Ivalsa, Italy: MAS3, CR1, LF1, LA1.

The temperate hardwood species were tested under the commission of:

- The Netherlands Timber Trade Organisation (VVNH)/ Rijkswaterstaat (part of the Dutch Ministry of Infrastructure and the Environment): O1, O2, R1.
- Brookhuis Micro Electronics: C1.

The temperate hardwood species were tested at the laboratories of:

- TNO: O1, O2, O3.
- CNR Ivalsa, Italy: C1.

The softwood species were tested under the commission of:

- The Netherlands Timber Trade Organisation (VVNH)/ Rijkswaterstaat (part of the Dutch Ministry of Infrastructure and the Environment): D1, L1, L2.
- Brookhuis Micro Electronics: S1, S2, S3, S4, S5, S6, S8.

The softwood species were tested at the laboratories of:

- Delft University of Technology: S1, S2, S3, S7, S8.
- CNR Ivalsa, Italy: S4.
- MeKa, Latvia: S5, S6.

A number of students at the Delft University have cooperated in performing a part of the experiments and analysing them as part of their Bachelor thesis. These experiments focused on answering sub questions concerning the influence of size, moisture content, grain angle deviation and knots. The results of their work are incorporated in this thesis. The individual researches are reported in Van Otterloo (2010), Flink (2012), Van der Poel (2012), Rey (2012), Van den Have (2013) and Hek (2014).

Curriculum Vitae

Gerard Johannes Pieter Ravenshorst was born on 23 March 1966 in Stad Delden.

He followed his secondary education at the Christelijk Lyceum in Almelo where he obtained his diploma for Atheneum B.

He studied building engineering at the Technical University in Eindhoven and retrieved his diploma as structural engineer in 1995.

After that he worked as a structural engineer in practice for different engineering firms. In that period he worked as an all-round structural engineer in the design and verification of load-bearing structures of houses, offices and public buildings.

From 2000 he worked as a researcher for TNO Building and Construction Research in Rijswijk and Delft, where he focused on timber structures.

In 2007 he started working at the section Structural and Building Engineering of the Delft University of Technology. He works as a lecturer and researcher. In addition to that he has been working on this PhD-thesis.

He is a member of national and international standardization committees in the field of timber engineering and timber material properties. Since 2013 he is chairman of the Dutch standardisation committee for timber structures.

He is member of the committee of the Timber Centre from the Netherlands Timber Trade Organization that gives information about the application of timber in for instance buildings and hydraulic applications