

The bearing strength capacity perpendicular to grain of Norway spruce

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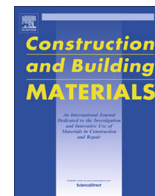
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The bearing strength capacity perpendicular to grain of norway spruce – Evaluation of three structural timber design models



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HIGHLIGHTS

- Current models to predict the compressive strength perp to grain are unreliable.
- Only one model based on yield slip-line theory is accurate.
- The V.d. Put model is the candidate for future building design code implementation.

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ABSTRACT

The perpendicular to grain compressive strength of timber is known to be much lower than the strength parallel to grain. Many timber structures, however, rely on this property especially in bearings that occur frequently in building practice. The linear elastic–plastic behaviour of structural timber loaded perpendicular to grain has been a problematic issue for decades which is reflected in the differences between the prediction models in structural design codes over the world. This article concentrates on the evaluation of the strength predictive ability of three of the latest bearing models having an empirical, semi-empirical or physical background. On the bases of a large database of over 1000 test results covering eight practical load cases, it is shown that the accuracy and consistency of the physical model is the best, which makes it a potential candidate for the new generation timber design codes.

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1. Introduction

It was Borg Madsen [1] who called Compression Perpendicular to Grain (CPG) a Cinderella property, when he complained that not enough engineering thinking was being applied to a property as the compressive strength perpendicular to grain. He referred to the ASTM-D-143 standard test method of 1926 [2], and the empirical design approach which is still in use in countries like the US, Canada, Australia/New Zealand and in Asia. He notes correctly that, with respect to perpendicular to grain, load introduction for both the “strength limit state” (ULS) as well as the “serviceability limit state” (SLS) can govern the design of timber structures. For serviceability considerations, deformations are the key issues being influenced by the initial elastic deformation and creep deformation which is driven by the wood species and moisture conditions. Thelanderson and Mårtensson [3] conclude that “design with respect to ULS need only be made when bearing failure may reduce the structural capacity of structural members or otherwise affect the safety of the structural system. In all cases

where design in ULS is not necessary, design should be made in the SLS”. This, however, presumes that the design for ULS situations is sufficiently accurate and reliable. Leijten [4] showed that the bearing capacity design models mainly used around the world do not comply, and that this assumption is far from accurate. A common and unified approach to tackle the issue seems far away. This study, however, aims to improve this situation. It also hopes to contribute to the ongoing revision of Eurocode 5 with the aim to improve code design models.

A relatively easy way out for design code regulations is to prescribe calculation methods resulting in conservative predictions. Usually, an important input parameter for the models prescribed in the design codes is the standard CPG strength. The lack of a unified approach to determine the CPG strength has led to situations like in the Scandinavian countries. In these European countries, the standard characteristic bearing strength is 2–3 times higher than the stress at proportional limit determined by tests. This is considered questionable and far from conservative, Thelanderson and Mårtensson [3]. Also Kevarinmäki [5] concludes that the short-term CPG strength value for Spruce in Finland is too high, 6.5 N/mm², and is associated with a deformation generally

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exceeding 10% of the timber member depth. He argues that 3.3 N/mm² would be more appropriate.

It will be shown that the reliability and accuracy of calculation models used for design in practice is an issue to be considered. The evaluation presented below focuses on the reliability and predictability of a three models. One of the models is currently in use for practice, while the other models have been published but not yet been accepted or had sufficient credibility. Such undertaking requires a large database of experimental test results covering most of the design situations occurring in practice. In addition, all tests must have been carried out using the same test procedure and using the same method to determine and define the CPG strength.

2. Load cases

In order to support and distinguish the best predicting model, a sufficient number of test load configuration cases should be evaluated. The load configurations should, to a large extent, reflect building practice situations. In Fig. 1 an overview is presented of these load configuration cases categorized as B–F. These categories were introduced in Leijten et al. [6] and incorporate fully and partially loaded cases. The arrow indicates the force applied, and a steel plate underneath takes care of uniform equal load introduction. The area with the highest CPG stress fails. In cases where the loaded area is as big as the support area, as in load cases D and E, both areas fail due to CPG simultaneously. Cases G and H are load cases without a direct support, so-called discrete supports. Load case H was later added by Lathuilliere [7]. Obviously one can vertically flip these load cases. It is assumed, however, that the timber at the load introduction fails in CPG. Load case J is added to check for the interaction between nearby loaded areas.

To enable comparison between the experimental test results carried out and reported by different researchers, all the experiments should use as a starting point a common standard test procedure and evaluation method to determine the CPG strength. The specimen used by the standard test method is shown in Fig. 1 as load case A. This standardized specimen of clear wood is loaded over the full upper surface of 45 × 70 mm with a depth of 90 mm. The specimen depth equals the distance between the loaded surface and the bearing support. The deformation used for the load–deformation curves is the change of this distance. The latter is not fully in agreement with the test standard CEN/EN 408 [8]. However, Le Clevé [9] has shown that taking the deformation as the change in depth of the specimen is the preferred measuring method and provides more consistent results than using the CEN/EN 408 method, Fig. 2. In this study, all evaluations were done by following the principles of CEN/EN408 but having a gauge length equal to the specimen depth.

The CPG strength in CEN/EN408 [8] is defined as the intersection of a line (2) parallel to the linear part of the load–displacement

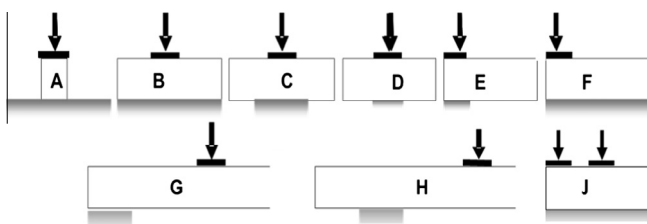


Fig. 1. Overview of load cases or categories (after [6] and [7]). (A) standard specimen EN408; (B) center load, full support; (C and D) opposite load, local support; (E) end load, local support; (F) end load, full support; (G and H) discrete load; (J) two spaced loads, full support.

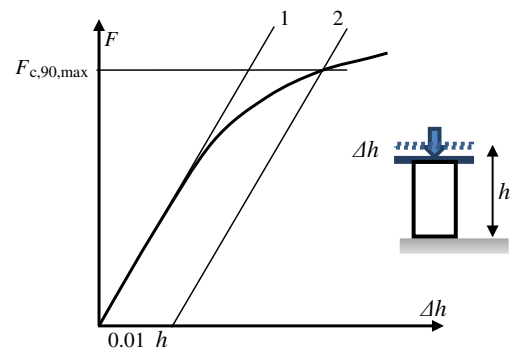


Fig. 2. Definition of CPG strength.

curve, line (1) that is off-set by 1% of the standardized specimen depth, Fig. 2. In cases B to G, where the test specimen dimensions deviate from the standard specimen, the same method is employed to determine the CPG strength. The deformations are plotted in [mm] and not in percentages of the specimen depth, the reason being that in the loading categories G and H it is not the whole specimen depth which is affected by the CPG stresses, Leijten et al. [10]. Furthermore, when the depth of the test specimen differs from the standard 90 mm, the 1% off-set line (2) is off-set 1% of the actual specimen depth. Only for category D and E the 1% off-set refers to half the specimen depth.

3. The experimental data

A literature search results in many reports dealing with CPG. Besides strength and stiffness data, there is also information about factors that influence these properties. These are the wood species, load case, moisture content, specimen shape, annual ring orientation, etc. All of these have drawn attention and have been investigated. The literature listing that follows is not exhaustive. Kollmann and Coté [11] reports pre-WWII test results by Graf [12] and Suenson [13]. These investigations counts only one test per load case which is considered insufficient and therefore these results are discarded from the analyses. However, since load case C is not covered by any researcher besides Graf [12], and considering the load deformation curves of his study are available, including one for the cube test specimen, it is decided to add these results to this study. Gehri [14] and Hübner [15] p.13 make reference to additional studies by researchers like Föppl [16], Staudacher [17,18], Gaber [19], Frey-Wesseling and Stüssi [20] and Rothmund [21]. They do not report, however, if these researchers made use of a reference standard test specimen, a standard test method or have a common definition of the CPG strength. This is why their research results have been omitted from this study. Although Kühne [22] p.42 accurately defined how to derive the CPG strength values, he did not apply any off-set and, without reporting the load deformation curves, his elaborate test results also cannot be taken into account. This is why many sources mentioned in [6] are not used for this more accurate analysis. Many other sources originating from the US like Basta [23], report tests that were carried out in accordance with ASTM-D-143 [2]. In [23] an elaborate literature review is provided about CPG tests using this ASTM standard. The focus is not exclusively on Spruce (*Picea Abies*) but on many different wood species, dealing with effects of moisture, annual ring orientation, etc. This standard test procedure does not determine the CPG strength as a physical material property but is only based on load case B, Fig. 1. The definition of the CPG values obtained with this method was once based on the proportional limit, but it is now based on a 1 mm deformation limit. As reported

by Basta [23], CPG values used by National Design Specification of the US design code are not the lower 5% values as used for most structural material properties, but they are mean values which are about 60% higher. Although many test reports in literature use this test standard, their test data were not considered for this study because of the incompatible starting points just mentioned. In [24] it is shown that using both EN and ASTM test standards not only results in considerable deviating standard CPG strength values but also that differences go up a factor of three when calculating the CPG strength capacity applying respective design codes of Europe and US. The effort to introduce correction factors to make the incompatible test results compatible by [4] are too general to be used for this detailed study.

The test data adopted for this study originate from the sources given in Table 1 col.(1). The wood species of the test specimens is Spruce (*Picea Abies*). This table provides an overview of the number of samples, col.(2), one or more load cases tested, col.(3), the use of sawn timber specimens or glued laminated test specimens, col.(4) and the number of tests per sample. The range of dimensions of the loaded area at the top of the test specimen is provided in the remaining cols.(6) to (8). In all cases the loaded area width corresponds with the width of the test specimen.

On the load-deformation curve of each test two points are of interest. The first value related to the onset of yielding determined with the off-set line as shown in Fig. 1 and the second the CPG stress at 10% deformation. The total number of samples is 104 with 1017 test results for the on-set of yielding deformation and 59 samples with 524 test results for 10% deformation. The samples are very unevenly distributed over the load cases. For instance, for the on-set of yielding, 39 samples with 329 test results (one third of the total) deal with load case B while a few tests have been reported for load case C. Table 2 shows the number of samples as well as the total number of test results per load case. For the two load cases G and H, the distance between the support and the load is at least 2.5 the specimen depth. Not all the sources presented in Table 1, however, allowed the assessment of the CPG stresses at 10% deformation and for that reason the number of test results in the last column of Table 2 are different from the third column. Again, load case B is studied most at 10% deformation having still 30 test samples with 220 test results (42% of a total of 524).

All the test data mentioned above used test specimens of European Spruce (*Picea Abies*) conditioned at 20 ± 2 °C and $65 \pm 5\%$ RH

Table 2

Overview of samples and test data per load case.

Load cases	Onset of yielding <i>n</i> samples	Number of tests #	Deformation 10% <i>n</i> samples	Number of tests #
B	39	329	30	220
C	4	4	0	0
D	15	153	2	37
E	3	51	1	14
F	14	240	6	70
G	21	180	17	153
H	8	30	0	0
J	3	30	3	30
Total	104	1017	59	524

which results in an equilibrium moisture content of about 12%. The standard mean CPG strength, $f_{c,90}$, for load case A varies among the reported investigations as the Spruce specimens originate from different growth areas and exhibit a natural variation in strength, Table 3. Ignoring standard CPG strength result by Graf [12] of 1.46 N/mm² which is way off what all other researchers reported, the average standard CPG strength for Spruce is found to be 3.15 N/mm², Table 3. Hoffmeyer et al. [36] reports tests on 74 sawn timber specimens and 120 glued laminated specimens having a mean CPG strength of 2.9 N/mm² which corresponds with Table 3 overall average taking into account the number of ST and GLT specimens. Nevertheless their results are not included in Table 3 as they didn't perform the tests according to EN408 with the deviation mentioned under Load Cases. One of the conclusions in [36] was that the standard CPG strength does not change significantly with the specimen dimensions. This was later confirmed by Augustine et al. [26] for glued laminated Spruce specimens of 300 and 600 mm depth.

4. Design models for CPG strength capacity

For structural calculations the design engineer needs specifications how to determine the CPG lower 5% strength capacity. An overview of the models used by the building design codes in the last decades show an abundance of methods indicating a difficult to tackle problem. To mention a few models, reference is made to [37–47] and Eq. (1). Most of the strength capacity models are

Table 1

Overview of data sources and test info.

(1) Source	(2) Sample #	(3) Load case Fig. 1	(4) Type ¹ wood	(5) Tests # per sample	(6) Loaded ² length [mm]	(7) Loaded ³ width [mm]	(8) Specim depth <i>h</i> [mm]
Graf [12]	4	C	ST	1	55-79-120-180	180	179
Riberholt [25]	9	B	ST	3-6	15-40-80-145	40	40-80-145
Augustine et al. [26]	6	B-F	ST	15	150	160	200-480
Poussa et al. [27]	2	D-E	ST	27	70	45	90
Hansen [28]	9	B	ST	5	45-95-145	45	95-145-220
Bleron et al. [29]	5	B-D	GLT	9-10	100	78	150-300
Leijten et al. [10]	12	G	ST/GLT	4-8-9	100	40-80	145-220-400-600
Moseng and Hagle.[30]	14	B-D-E-F-G	GLT	10-12-14-15	48-90	45-90	48-90-198-405
Hardeng [31]	5	B-F-G	ST/GLT	8-9-12	90-201	48-89	90-198-405
Lathuilliere et al. [7]	22	B-D-E-G-H	GLT	2 to10	50 to 240	78 to 210	100 to 810
Ed and Hasselqvist [32]	3	G	ST	12	60-90-120	90	270
Lantinga et al. [33]	4	F-B	GLT	7-16	50	75-100	280
Goeij [34]	2	F-B	ST	21-25	50-90	35-50	45-50
Levé et al. [9]	3	F-B	ST	40	70	45	90
Mahangoe [35]	7	B-F-J	ST	9-11	45	45-150	45-90-145
TOTAL	104			1017			

¹ Type ST = sawn timber; GLT = glued laminated timber.² Loaded length parallel with the grain.³ Loaded width perpendicular to grain.

Table 3
Overview of standard CPG strength.

Source	Mean $f_{c,90}$ [N/mm ²]	Specimen Number of tests	Spruce ¹
Graf [12]	1.46	1	ST
Riberholt [25]	3.3	24	ST
Augustine et al. [26]	3.31	62	GLT
Poussa et al. [27]	2.80	200	ST
Hansen [28]	2.70	30	ST
Bleron et al. [29]	3.01	22	GLT
Hardeng [31]	3.69	8	GLT
Lathuilliere et al. [7]	3.26	42	GLT
Ed et al. [32]	3.12	6	ST
Lantinga et al. [33]	2.65	10	GLT
Goeij [34]	3.13	24	ST
Levé et al. [9]	2.51	48	ST
Mahangoe [35]	2.80	10	ST
Mean	2.81	342	ST
Mean	3.22	144	GLT
Overall mean	2.93	Total 487	

¹ Type ST = sawn timber; GLT = glued laminated timber

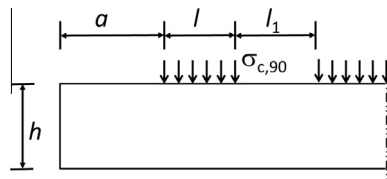


Fig. 3. Conditions for $k_{c,90}$.

empirical in nature and are very simple. Models that were published after 1983 followed the guidance given in the CIB-W18 [38] with as starting point Eq. (1)

$$\sigma_{c,90} = \frac{F_{c,90}}{b \cdot l} \leq k_{c,90} \cdot f_{c,90} \quad (1)$$

where: $\sigma_{c,90}$ is the actual CPG stress based on the load $F_{c,90}$ divided by the loaded surface area (b is the width and l is the parallel to grain loaded length); $k_{c,90}$ is a parameter that accounts for influencing factors like, moisture content, wood species, and load case; $f_{c,90}$ is the standard CPG strength value. The $k_{c,90}$ values proposed by [38] are as follows:

$k_{c,90}$ is equal to 1.0 unless $a \geq 100$ mm and $l \geq 150$ mm, Fig. 3. Where a is the distance from the end grain to the loaded area, l the length of the loaded area and l_1 the spacing between adjacent loaded areas. When the conditions for a , l and l_1 are satisfied $k_{c,90}$ may be determined according to:

$$k_{c,90} = \sqrt[4]{150/l} \text{ with } 1 \leq k_{c,90} \leq 1.8$$

Although Fig. 3 as presented in [38] does not show how the beam is supported it apparently is not an issue to consider. Note that the $k_{c,90}$ in the equation is restricted up to a value of 1.8. Later models propose values up to 4.0. Design models of later date, such as [39] in 1987 and [42] in 1999, though very similar, provide more detail for $k_{c,90}$ values as shown in Tables 4 and 5. The tables will not result in smooth transitions in the CPG design capacity but in jumps when boundaries are crossed.

Table 4
 $k_{c,90}$ values according to Eurocode 5 (1987) [38].

	$l_1 < 150$ mm		$l_1 \geq 150$ mm	
			$a \geq 100$ mm	$a < 100$ mm
$l \geq 150$ mm	1		1	1
$150 \text{ mm} > l \geq 15$ mm			$\sqrt[4]{150/l}$	$1 + (\sqrt[4]{150/l} - 1)(a/100)$
$15 \text{ mm} > l$			1.8	$1 + (a/125)$

The design model now in use in the European design code [46], based on research by Madsen [1] and modified by Görlacher and Blass [48], is presented as Eq. (2) with $k_{c,90}$ values given in Table 6. The background for the introduction of an effective length, l_{ef} instead of the actual

$$\sigma_{c,90} = \frac{F_{c,90}}{b \cdot l_{ef}} \leq k_{c,90} \cdot f_{c,90} \rightarrow \sigma_{c,90} = \frac{F_{c,90}}{b \cdot l} \leq \frac{l_{ef}}{l} k_{c,90} \cdot f_{c,90} \quad (2)$$

loaded length l is to account for the contribution by the rope effect of wood fibers adjacent to the loaded area, Fig. 4. This rope effect proposed by Görlacher and Blass as determined with Madsen’s test is 30 mm at maximum. However, this effect is questioned by Moseng and Hagle [30], as they didn’t find any influence between the use of steel plates’ rounded and sharp edges. The latter supposed to cut the surface fibers. Again the nature of the tabulated $k_{c,90}$ values result in unrealistic jumps in the design capacity, Table 6. Especially if the loaded length of a support is close or slightly more than 400 mm, the $k_{c,90}$ value drops from 1.75 to 1.0 applicable to glued laminated beams. The background for these jumps is unknown.

The only design model based on a physical theory is presented by Van der Put in 1990 [49] and is found to have a high potential [6]. The model is based on the assumption that the compressive stresses spread as in an isotropic material as if the effect of the relative stiff fibers parallel to grain can be ignored. These stresses distribute over the depth of the material according to the yield or slip line theory. The degree of spreading depends on the deformation as shown in Fig. 5. From theoretical considerations it follows that at the onset of yielding the compressive stresses spread by 1:1 (45°) and for large deformations of about 10% the spreading angle is 1:1.5 (34°). This is in agreement with findings by [50] who in 1982 reported the same spreading ratio for CPG stresses to die out. The theory applies generally and therefore is assumed to be wood species independent. The model is given by Eq. (3).

$$\sigma_{c,90} = \frac{F_{c,90}}{b \cdot l} \leq k_{c,90} \cdot f_{c,90} \rightarrow k_{c,90} = k \sqrt{\frac{l_{ef}}{l}} \quad (3)$$

Where l_{ef} is the effective or spreading length parallel to the grain as shown in Fig. 5; k is a correlation factor to cater for differences in model prediction and experimental results. Although it is suggested in [44] that for load case B theoretically this k -factor is approximately 1.1 for all other cases the suggestion is $k = 1.0$. The effective length is restricted by the geometric (dimensional) boundaries of the beam or by nearby spreading stresses, Fig. 6. For situations where the support conditions are not continuous but discrete as in load cases G and H of Fig. 1, previous models did not provide any guidance for the design engineer. In [10] it was shown that for load cases G and H the depth of the spreading stresses is limited to a maximum of 140 mm or 40% of the beam depth, whichever is the smallest, Eq. (4)

$$k_{c,90} = \min \left\{ \begin{array}{l} 140 \text{ mm} \\ 0.4 h \end{array} \right. \quad (4)$$

where h is in mm.

The last and most recently published model by Lathuilliere et al. [7] is actually a semi-empirical model. Although the derivation initially follows analytical principals, the introduction of arbitrarily

Table 5
 $k_{c,90}$ values according to DIN1052 (1999) [42].

	$l_1 < 150 \text{ mm}$		$l_1 \geq 150 \text{ mm}$	
	$a \geq 100 \text{ mm}$	$a < 100 \text{ mm}$	$a \geq 100 \text{ mm}$	$a < 100 \text{ mm}$
$l \geq 150 \text{ mm}$	1.3	$1 + \frac{3a}{1000}$	1.7	$1.3 + 4a/1000$
$150 \text{ mm} > l \geq 15 \text{ mm}$			$1.7 + (150 - l)/100$	$1.7 + a(150 - l)/1000$
$15 \text{ mm} > l$			3.0	$1.7 + a/77$

fixed values for certain parameters brings it down to a fitting procedure. The model is presented as:

$$\sigma_{c,90} = \frac{F_{c,90}}{b \cdot l} \leq k_{c,90} \cdot f_{c,90} \tag{5}$$

where:

$$k_{c,90} = 1 + \frac{f_v}{f_{c,90}} \cdot \frac{k_{sh} \cdot h_t}{l} \cdot \frac{2}{3} \cdot k_{sb} \cdot k_{sc} \cdot n_d \tag{6}$$

with:

$$k_{sh} = \begin{cases} \frac{1}{3} & \text{in case of bending} \\ \frac{1}{2} & \text{for all other load cases} \end{cases} \quad (\text{in [6] the values are reversed accidentally})$$

$$k_{sb} = b^{-0.325}$$

$$k_{sc} = \begin{cases} 1.51 & \text{descrete support} \\ 1.85 & \text{continuous support} \end{cases}$$

$$n_d = \begin{cases} 1 & \text{end support} \\ 2 & \text{for intermediate support} \end{cases}$$

where: f_v the shear strength, $f_{c,90}$ the standard CPG strength, h_t the beam depth, l the length parallel to grain of the loaded area, b width of the loaded area.

5. Model evaluation

Now the models and the sources of the test data have been briefly reviewed, the predictive ability of the models will now be analyzed. Although statistical analyses deliver key parameter values to quantify differences between models, a graphical representation is added to show what statistical values can be

Table 6
 $k_{c,90}$ according to Eurocode 5 (2008) [46].

	$l_1 < 2h$		$l_1 \geq 2h$	
	Sawn timber	Glued laminated timber		
		$l \leq 400 \text{ mm}$	$l_1 > 400 \text{ mm}$	
Local support	1	1.25	1.5	1
Continuous support		1.5	1.75	1

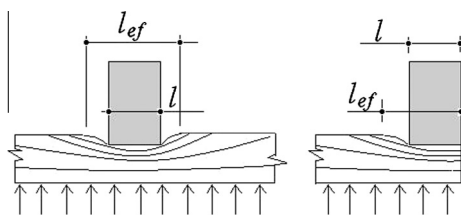


Fig. 4. The effective length.

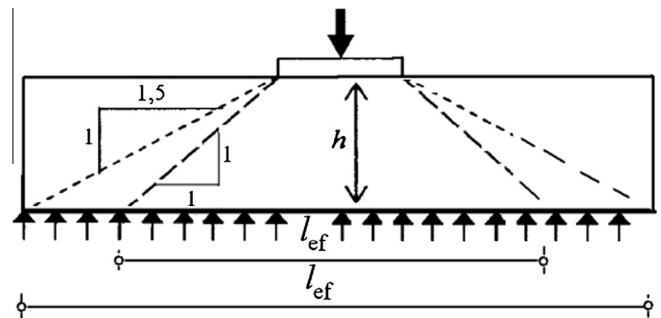


Fig. 5. Spreading of compressive stresses.

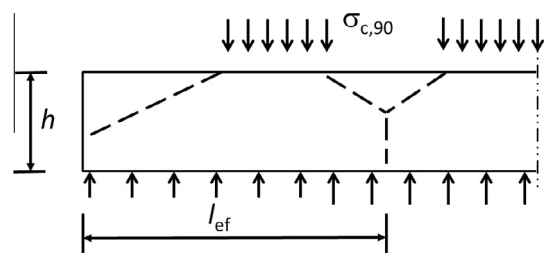


Fig. 6. Restrictions for the effective length.

imagined. For that reason graphs are produced that set the sample mean CPG values on one of the graph axes against the mean model prediction, Fig. 7. If the data is on the diagonal line, an ideal fit is obtained. A histogram of the ratio of test value and the model prediction is also given. The histogram is complimented with a fitted normal distribution curve, and the key statistical parameter values for the mean and standard deviation are given in the legend.

As shown the Van der Put model scores a mean of 0.99 which is close to the most ideal value of 1.0 while the standard deviation is

0.166. A similar evaluation is now presented for the model by Lathuilliere and the Eurocode5/A1 [43] in Figs. 8 and 9.

In Table 7 an overview is given about the statistical mean and standard deviation of the fitted normal distributions of the histograms for the on-set of yielding as well as for 10% deformation, although the graphs of the histograms of the latter are not presented here.

In the above analysis all test data has been considered irrespective of the load case. To check if the models perform differently per load case and deformation the same evaluation is repeated but for each load case separately, Table 8. From this Table it follows that again the Van der Put model is the most consistent for the onset of deformation. The frequently in building practice occurring load

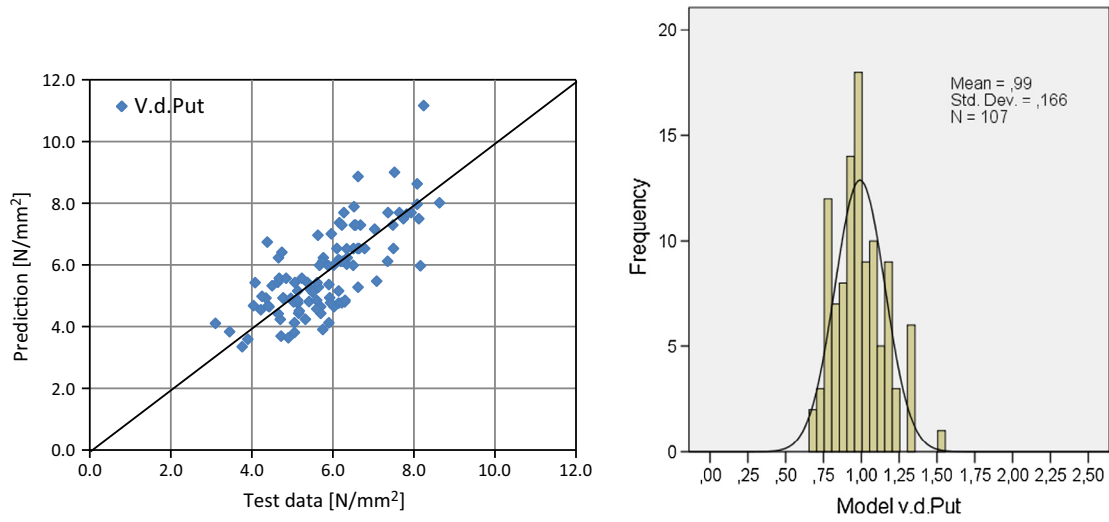


Fig. 7. Prediction by Van der Put model (left) with histogram (right).

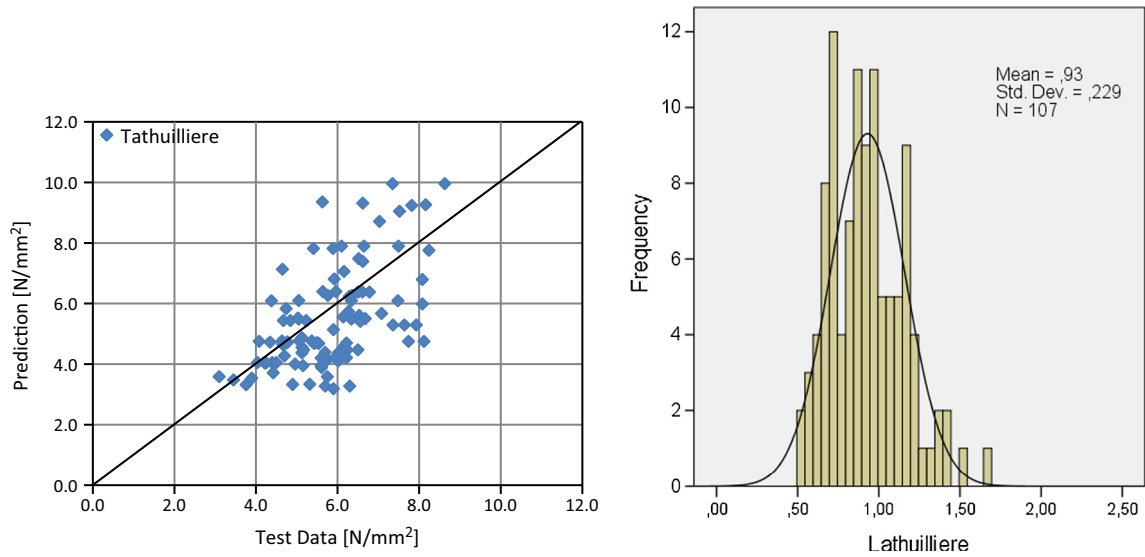


Fig. 8. Prediction by the Lathuilliere model with histogram.

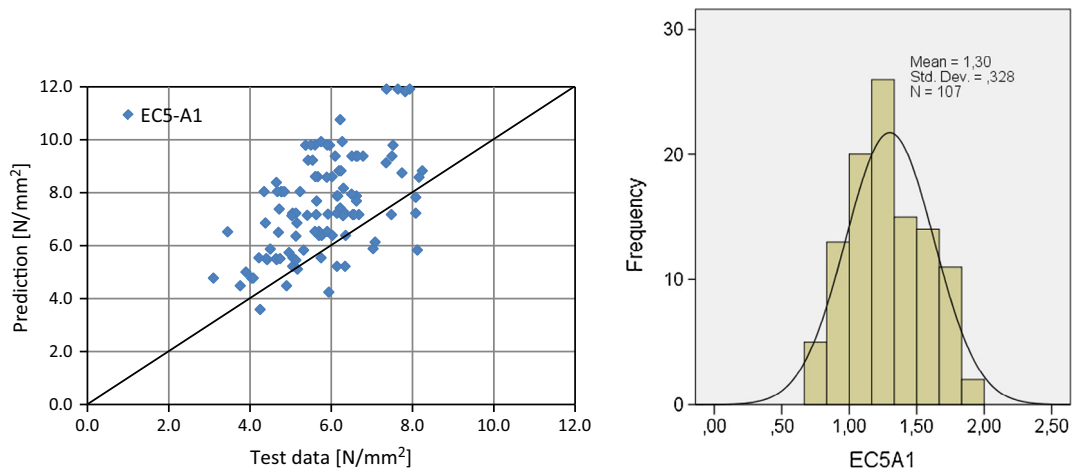


Fig. 9. Prediction by the Eurocode 5/A1 model with histogram.

Table 7
Overview of the statistical parameters of the histogram.

Model	On-set of yielding			10% deformation		
	Mean	Stand. dev	Coef. of var (%)	Mean	Stand. dev	Coef. of var (%)
Van der Put	0.99	0.166	17	0.83	0.132	16
Lathuilliere	0.93	0.229	25	0.65	0.170	26
EC5/A1	1.30	0.328	25	0.99	0.237	24

Table 8
Overview of model performance per load case.

Load case	Samples #	Tests n	Mean ratio prediction/test result		
			V d Put	Lathuilliere	EC5/A1
<i>Onset of yielding</i>					
B	39	329	1.09	0.90	1.37
C	4	4	0.88	0.65	0.83
D	15	153	0.97	0.89	1.16
E	3	51	0.97	0.86	1.23
F	14	240	0.92	0.81	1.18
G	21	180	0.99	1.13	1.39
H	8	30	0.89	1.14	1.32
J	3	30	0.85	0.73	1.14
Mean			0.96	0.91	1.21
St. dev			0.08	0.17	0.18
C. of var			8%	19%	15%
Total	104	1017			
<i>10% deformation</i>					
B	30	220	0.81	0.59	0.93
C	0	0			
D	2	37	0.88	0.66	1.21
E	1	14	0.94	0.76	0.98
F	6	70	0.87	0.63	1.01
G	17	153	0.91	0.83	1.07
H	0	0			
J	3	30	0.85	0.69	1.14
Mean			0.88	0.70	1.06
St. dev			0.04	0.09	0.10
C. of var			8%	13%	9%
Total	59	524			

case B is on average + 10% to low. This is in contrast to the Lathuilliere and EC5/A1 model in which predictions are respectively +9% and +37% too high. Even for load cases H and J the EC5/A1 model is out by more than +30%. An effort was made to improve the performance of the Van der Put model by making use of the parameter k in Eq. (3) and the deviations from the ideal ratio of 1 to apply a $k = 0.9$ for load case B and $k = 1.15$ for load cases C, H and J for instance. However, this didn't significantly improve the overall performance of the model nor did the standard deviation decrease much.

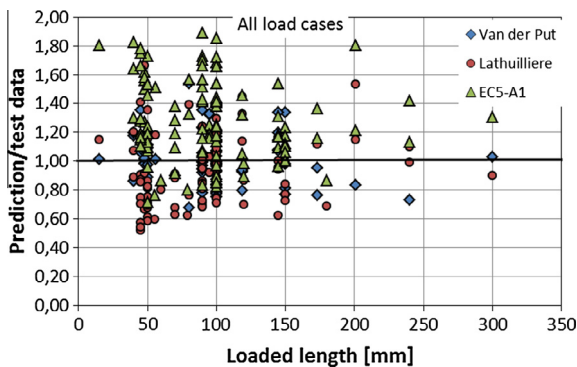


Fig. 10. Model prediction ability versus the loaded length.

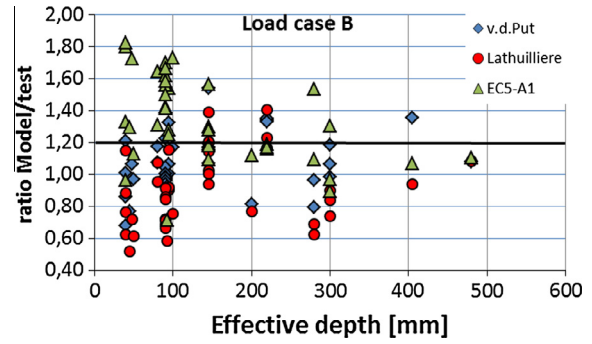


Fig. 11. Model prediction ability versus the (effective) depth.

The model currently in use by building practice, as it is incorporated in the European design standard Eurocode 5, performs badly. The main cause of this is the inability to account for the differences in the depth of the beam and the jumps in the $k_{c,90}$ values, Table 6.

Because at 10% deformation, the CPG strength increases by about 15% as compared to the onset of yielding. Since neither the EC5 model nor the model of Lathuilliere take into account the level of deformation, these models automatically give a lower ratio of model prediction to test results at 10% deformation. Furthermore, since the EC5 model substantially overestimates the CPG strength at 1% deformation, by change it gives a good prediction at 10% deformation. In contrast the Van der Put model is the only model that acknowledges the increased CPG strength at 10% deformation; although apparently not to the extent of the test results (see Table 7). Nevertheless being the only of the three models accounting for this increase, the Van der Put model is indeed the most appealing one.

There are obviously many more variables to check with the models. One of them is the length of the loaded area or the (effective) depth of the test specimen versus the strength prediction/test data ratio. In particular the (effective) depth might be of interest as for instance a 10% deformation of a 40 mm depth specimen is very different from a 400 mm specimen. How the models cope for the onset of yielding deformation with these differences is presented for all load cases and for load case B as the most frequently tested, in Figs. 10 and 11 respectively. In both figures the EC5/A1 model tend to be well represented in the non-conservative part (>1.0). On average in both figures the EC5/A1 model results in a non-conservative approach.

6. Conclusion

The main aim of this study is to give a state of the art of the available test data of compressive perpendicular to grain (CPG) strength for the wood species Spruce (*Picea Abies*) and to test the predictive ability of three models. Test data of a great number of sources is collected which all had a common test method and definition of the CPG strength. Eight load cases are distinguished and the predictive ability of three models is compared at the onset of yielding as well at 10% deformation. The three models selected are the latest published empirical, semi-empirical and physical models as given by the Eurocode 5/A1 [43], Lathuilliere et al. [7] and Van der Put [44], respectively. Considering all eight load cases it can be concluded that the best and most consistent and accurate model is the physical Van der Put model [44]. Compared to the Eurocode 5/A1 model it requires hardly more calculation effort. The Eurocode 5/A1 model currently applied by practice is the least of the three models evaluated.

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