The Bearing Strength Capacity of Continuous Supported Timber Beams

- a unified approach for test methods and structural design codes

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Summary

Bearing or compressive strength capacity perpendicular to the grain of timber beams is a troublesome issue. Not only do many different load cases occur in practice that are not covered by structural timber design codes but also these codes provide only a basic provision and vary throughout continents. Code design rules require the standardized compressive or bearing strength to be determined by test standards. An assessment of the results of standard test methods of the European Union, North America and Australia/New Zealand show incompatibility. It is demonstrated how previously incompatible results can be made compatible by using a physical model and some calibration tests. The model proposed offers a consistent and simple way to bridge the differences between both test standards and structural design codes. When the model is implemented in structural design codes, the designer will simply be able to calculate more accurately the bearing capacity, using any standard compressive strength, derived with whatever standard test method.

Keywords: Timber, bearing, perpendicular to grain, capacity, radiate pine, compression strength.

Introduction

Timber beams, used as structural elements like floor joists or studs, always require support at the beam ends to transfer the forces. Joists usually find support on either timber beams or other structural materials. This type of support is known as local or discreet. Studs in timber frames find end supports in a top and bottom rail. When the bottom rail is support over its full length the support is called continuous. In this article the focus is on a fully supported beam locally loaded perpendicular to grain, Fig.1. Both support conditions are just two examples where the compressive strength perpendicular to grain, also referred to in many countries as bearing strength, play an important role. Nevertheless, many feel that compressive (bearing) failure does not pose a threat to the structure because the type of failure is plastic and does not lead to structural collapse. However, high deformations can impair the use of a structure just as well as brittle failure. It is wrong to mix serviceability and ultimate limit considerations. It is generally accepted that both are completely separate situations that require a different approach. Deformation criteria should, in principal, not be incorporated into ultimate limit state design because deformations at the moment of structural collapse, are irrelevant. A rough sketch of the load-deformation behaviour of timber loaded perpendicular to grain is given in Figure 2.

Comparing the respective test standards, to determine the standard compressive strength as well as structural design codes of Europe, North America, Australia/New Zealand and Asia is revealing. In the test standards, the dimensions of the test specimen, load configuration and definition of the standard compressive strength are different, all leading to incompatible results. This would not be a problem if applying the regulations in the respective structural design code would result in the same bearing capacity for a given situation. For the situations in Figure 1, bearing capacity differences of 30% and more can be observed. In other words, there is a need to update and unify both standard test procedures as well as design code rules. It is realistic to assume that code writers are reluctant to modify drastically respective test and design standards that have been in use for decades. If this is so, this article demonstrates the ability of a recently published physically based model that explains the differences and shows how to unify test and structural design standards with a minimum of effort.

Differences in test piece dimensions and load configuration

Since the early 1926, the ASTM-D143 test method to determine the standard compressive strength perpendicular to grain has been used. The test setup reflects the situation of a beam fully supported on a bearing wall or foundation and loaded by a square stud, Bodig and Jayne (1982), Figure 3, specimen B. The timber specimen itself is 51x51x152mm of clear wood and is loaded in the centre by a 51x51mm square steel plate. The radial direction of the annual growth rings corresponds with the load direction. For structural size timber with knots, the compressive strength is higher because knots act like reinforcement, Madsen et al. (1982). For this reason, the compressive perpendicular to grain test is absent in ASTM-D198 (2008), which deals with static tests for structural lumber. Over the years, the ASTM-D143 test setup has been taken over by many other countries in America and Asia. Roughly the same specimen dimension and loading configuration is prescribed by the Australian and New Zealand Standard AS/NZS 4063: 1992 (Part 1) and by the latest version of September 2009. The test specimen has the following dimensions: 45 to 50mm depth, a minimum width of 35mm and a length 200mm, which is 48mm longer than the ASTM-D143 test piece.

Since the early nineties of the last century, the unification of the European market forced the European Committee of Standardisation (CEN) to draft European Standards. For structural timber the test methods are given by CEN EN408: 2003. In contrast to ASTM test method, the CEN test method takes a completely different starting point in which the test piece is loaded over its entire surface, Figure 3, specimen A. The test piece dimensions are 45x70x90mm and 45x70x180mm for sawn timber and glued laminated test specimens respectively. It reflects the choice of CEN to aim at well-defined physical material properties instead of properties related to typical uses or applications. It assumes that scientific models implemented in the European structural design code, use these material properties to determine the bearing capacity for any practical situation in contrast to other countries that have chosen the technological ASTM-D143 approach. For this reason only it is not surprising that differences between the ASTM and CEN test setups cause incompatible test results.

A third standard test method is presented in ISO 13910: 2003, which is similar to the alternative test given in the informative annex of Australian and New Zealand standard AS/NZS 4063: 2008. Although the loading condition resembles the ASTM-D143 test method, the specimen is not fully supported but mirrored, as shown in Figure 3, specimen C. Another deviation from ASTM and CEN is that structural size specimens are prescribed. The dimensions of the test piece are not yet specified but are all related to the specimen depth. The specimen length is 6 times the specimen depth. However, whatever the test piece depth, the steel plates' dimension used to introduce the load, is fixed to 90mm along the grain. Arguments for this choice given by Leicester et al. (1998) are to match in-service practices and partly because bearing is a local phenomenon that does not involve the full depth of the beam. In general, this test setup with opposite loaded steel plates results in the lowest strength values. In the annex of AS/NZ4063, it is argued that the test may form the basis for the determination of the characteristic strength values of structural size timber, although there is limited experience of application.

Although the results of these standard tests can be useful, the structural design code clauses need empirical determined correction factors to account for different loading and support conditions occurring in practice.

Difference in definition of strength

An important problem encountered in the interpretation of test results is the difference in definition of the compressive strength between test standards. The current situation is that for clear wood specimens of 50mm height, ASTM-D143 defines the compressive strength at 1.0mm (0.04”) deformation, which corresponds to 2% strain. The AS/NZ 4063:1993 sets a fixed 2mm deformation as definition, which corresponds to a strain of 4%, illustrated in Figure 4. It is obvious that this type of definition does not account for differences in wood species where density affects significantly the steepness of the stress-strain curve. Once again the CEN standard takes a different approach. The standard compressive strength is defined by the intersection between the stress strain curve and a line parallel to the elastic part of the curve with 1%h off-set, where h is the specimen height. The elastic part is taken as the intersection of a straight line and the deformation curve at 10 and 40% of the estimated standard compressive strength. This method therefore accounts for differences in elastic stiffness of wood species.

The ISO 13910 and AS/NZS 3603 take identical definitions. The ISO takes the intersection at a fixed 0,1h deformation. The AS/NZS follows the same approach as the CEN standard but now with a line 2mm off-set, irrespective of the specimen height h. The latter definition is suggested as an alternative for the proportional limit reflecting a serviceability limit according to Leicester et al. (1998).

As Poussa et al. (2007) shows for Finnish Spruce, the ASTM-D143 specimen result in 2.5 times higher strength values. i.e. 7.0 N/mm^2 compared to 2.8 N/mm^2 with CEN EN 408. Franke and Quenneville (2009) report for New Zealand Radiata Pine 5.7 N/mm^2 using the CEN EN408 test method and 11.1 N/mm^2 using the 2mm offset ISO 13910 and AS/NZ 4063:1993. The strength values are very different and incompatible.

Differences in structural design codes

If test standards of the continents result in different standard strength values, it is interesting to analyse what respective structural design codes stipulate in the bearing capacity design clauses. Particularly for NDS-2005 and the NZ3603:1993, which are the structural timber design codes of the US and Australia/New Zealand, only one particular clause deals with the determination of the bearing capacity. The bearing capacity is calculated by multiplying the loaded area times the standard compressive strength times a factor k_c. Both standards refer only to one specific design situation given in Figure 5 where two beam overlap. One is the continuously supported bearing beam locally loaded by the top beam. No guidance is provided which of the two beams actually fails in bearing. Experiments in the past must have shown the influence of the overlap length as the factor k_c depends on the overlap length up to 150mm. The smaller the overlap the higher is the factor k_c, Table 1. For both NDS-2005 and the NZ3603:1993 this factor is almost the same. The EN1995-1-1/A1 (Eurocode 5) takes a completely different approach covering more load cases. Similarly, the bearing capacity results from a factor k_c times the loaded area times the standard compressive strength. The k_c factor consists of two contributions. The first is an empirical factor of 1.25 and 1.5 accounting for solid and glued laminated timber, respectively. The second is a ratio that incorporates the influence of fibres near the edges of the loaded area that contribute in bearing. Particularly the fibres that run close underneath the loaded area will be squeezed into a S-shape when the deformation increases. Consequently, fibres that run close to the surface but parallel with the loaded edges hardly contribute. The S-shape fibres are assumed to contribute by the so-called rope or chain effect. This is accounted for by adding 30mm to the loaded length parallel to the grain of the loaded area. If on both edges fibres are squeezed into the S-shape the total length of the loaded area parallel to the grain is taken as two times 30mm, Table 1. This approach is based on empirical models by Madsen et al. (2000) and Blass et al. (2004). For discrete or local supports EN1995-1-1/A1 (Eurocode 5) specifies, $k_c = 1$. What is clear from Table 1 is that k_c values used by the standards are very different.

Incompatible test results made compatible

For many years researchers tried to develop models that account for the influences of geometry of the bearing beam and load configuration but proposed only empirical models. Recently, Van der Put (2008) republished his stress dispersion model (1988) based on plastic theory using the equilibrium method. The equilibrium method always results in a safe approach. This model is much more flexible, reliable and accurate than the empirical models so far and possesses a greater applicability to cover situations in practice as shown by Leijten et al.(2009a), who evaluated nearly 700 test results.

This model is the only realistic candidate to make incompatible test results compatible. If proven correct, it has the potential to become globally accepted by all future structural timber design codes, while the test standards for the determination of the compressive strength perpendicular do not necessarily need to be changed. The incompatible test results can be unified by deriving correction factors based partly on this theory.

The stress dispersion theory takes the standardized compressive strength of a full surface loaded specimen (CEN EN408) as a starting point. The bearing capacity is determined by the standard compressive strength multiplied with an adjustment factor, k_c accounting for the bearing beam dimensions and loading configuration. The theoretical derivation is given in Van der Put (2008). The stress field distribution chosen is the same as the one that follows from the slip-line theory, known from mechanics of solids, which is solved by the method of characteristics. The method assumes a dispersion of the bearing stresses activating more material depending on the level of deformation. To obtain a simplified solution the first term of a power polynomial approximation appeared to suffice. If the loaded area covers the full width of the beam the stresses disperse as shown in Figure 6. One deformation level, which assumes the onset of yielding and valid for small deformations of about 3% for coniferous wood, the bearing stresses disperse at a 45^0 angle (1:1).

The other deformation level for large 10% deformations the slope of the dispersing stresses changes to 34^0 (1.5: 1). The model uses the ratio of the parallel to grain length of the loaded area and the maximum or effective length of dispersion near the bottom support. The slope of dispersion beyond which bearing stress can be neglected was predicted by Madsen et al. (1982) and Hansen (2005) based on FEM and is in agreement with the model prediction. The stress dispersion model is formulated as:

$$\frac{F_d}{bl} = \sqrt{\frac{l_{ef}}{l}} f_{c,90} \qquad (1)$$

For coniferous wood:
$l_{ef} = h + 2\ (1.5h)$ for 10% deformation
$l_{ef} = h + 2\ (h)$ for 3 to 5% deformation

where:

F_d is the failure load, in N

l is the contact length of the applied load in grain direction, in mm

h beam depth, in mm

b the width of the beam, in mm

l_{ef} is the effective length at the support, in mm

$f_{c,90}$ is the reference compressive strength perpendicular to the grain (EN408), in N/mm^2.

The stress dispersion assumes a loaded area over the full width of the beam. For situations where the loaded area width is smaller than the beam width the theory assumes the same dispersion in all directions if deformations are large enough. In that case the square root expression in (1) changes to effective bearing area divided by the loaded area (1). However, the dispersion in width direction could not be confirmed by tests as yet, Leijten (2009b). The depth to width ratio of the bearing beam, so-called aspect ratio, should be limited to 4 in order to prevent premature failure mechanism such as rolling shear, Basta (2005). A model with these capabilities may also be applied in reverse. For instance, when only test results are available for loading conditions as shown in Figure 7, the model should be able to calculate backwards to retrieve the standard compressive strength for each load case. To demonstrate the flexibility of the model to cover special bearing cases, Figure 6 also shows the assumed stress dispersion for a situation where the top and bottom loaded areas are different. This however, assumes both stress dispersion areas are not too far apart.

Evaluation of former test result

The aim of this evaluation is to demonstrate the capabilities of the model to relate incompatible test results of different load configurations and to show how they stem from just one hypothetical standard compressive strength. To check this hypothesis, tests on Australian Radiata Pine reported by Leicester et al. (1998) are evaluated. They cover three load cases A, B and C shown in Figure 8. Leicester and co-authors were attempting to discover a suitable test method for structural timber (in-grade test method), as it was felt that the test results on small size clear wood specimens were inadequate in reflecting the bearing strength accurately. Kiln dried Radiata Pine specimens were conditioned to a moisture content of 12.5% and were graded into three strength grades. The test specimen sizes were 35x90mm and 35x190mm and the number of tests were n=300 and 290, respectively. The total specimen length was 6 times the depth. The length of the steel plate for the load application was 90mm for case A and C, while for case B plates were 45, 90 and 180 mm for the cases B1, B2 and B3, respectively. The deformations recorded were taken between the top and bearing plates. The load deformation curves were not reported. Only the average bearing stress per test series at a number of discrete points on the load deformation curve were reported. This includes the bearing stress using a 2mm offset as well as the bearing stress at 5 and 10mm deformation, as in Figure 4. The specimens of each strength grade were equally represented in the load cases.

The problem encountered in comparing bearing stresses given at fixed deformations of 5mm or 10 mm is that the strain for the smaller 90mm depth specimen is approximately twice as high compared to the 190mm specimens. As no information is given about the load deformation curves itself, it makes comparison of two specimen sizes impossible. In Figure 9 and 10, the compressive stress using the 2mm offset method and at 10mm (11%) deformation are presented graphically for each load case, represented by the two left bars for the load cases, A, B and C. The right two bars of each load case represent the hypothetical standard strength determined with the model, assuming a stress dispersion of 1:1 and 1:1.5 for the 2mm offset method and the 10mm (11%) deformation. The mean hypothetical standard compressive strength for all load cases A to C is 7.2 N/mm^2 and 5.9 N/mm^2 for the 90mm and 190mm specimens with a variation of less than 5%. The model is able to bring down the differences despite the different load cases.

Franke & Quenneville (2009) studied the effect of different strength definitions using clear wood specimens of New Zealand Radiata Pine with standard dimensions of 50x50x200mm. They report an average standard strength of 5.7 N/mm^2 and 6.2 N/mm^2 using the 1% offset CEN method and the 2mm offset method (AS/NZ method). The values are close, lower than the hypothetical standard strength calculated. One cause, which cannot be ruled out, is the presence of knots in the structural size specimens. Research carried out in the framework of this study showed that knots strongly affect the compressive strength positively.

Additional confirmation

There are still a number of issues that need to be resolved before the stress dispersion model can be given full credit. Does the model perform well for other load cases, for instance when the loaded area is smaller than the beam width and/or is not fully supported? The simple design code rules outlined previously do not consider all of these cases. For this reason, additional tests were performed on solid and glued laminated Radiata Pine of New Zealand. Test by Leijten (2009b) were carried out at Auckland University, New Zealand considering a variety of load configurations and support conditions like presented in Figure 11. The specimen width and depth were 240x45mm and 270x90mm for the solid and glued laminated specimens, respectively. The specimens were conditioned to 20^0C and 65% RH. The speed of testing was such that 3% to 5% strain was obtained in about 300 s.

The prediction ability of the bearing capacity using the structural design codes of NDS and AU/NZ and Eurocode 5, with appropriate k_c values presented in Table 1, is evaluated. The respective clauses require the respective standard compressive strength as input values.

For New Zealand Radiata Pine, the standard compressive strength values are taken from Franke and Quenneville (2009), who derived 11.1N/mm^2 for the mean and 8.9N/mm^2 as lower 5% value in accordance to the ASTM/ AU/NZ method. Following the CEN method, they reported 5.7 N/mm^2 for the standard mean compressive strength and 4.4 N/mm^2 as lower 5% value.

In the evaluation of the test results there were cases where the loaded area did not cover the total specimen width as shown in the bottom right situation of Figure 11, an empirical reduction of the stress dispersion sideways was applied; 1:10 for the small deformations and 1:1 at 10% deformation, which gave the best fit. Previous investigations on Norway spruce showed the dispersion to be 1:0.4 and 1:0.7, respectively, Leijten (2009c). Figures 12 and 13 show the mean compressive strength per test series versus the stress dispersion model predictions. They also show the NDS and AU/NZ design code predictions considering both the results at 1% off set (3-5% total deformation) and at 10% deformation, respectively. The high scatter of the NDS and AU/NZ structural design code predictions is not strange as for most tested situation, k_c =1.0 (Table 1) applies. For that reason, the predictions are represented by a horizontal row of dots. For the Eurocode 5, k_c values varied more however, without much improvement to follow the trend. The variability of the prediction is shown by Figures 14 and 15 where a fitted log normalised distribution is applied to each prediction. These figures lead to the following conclusions.

For bearing strength capacity estimation at about 3% total deformation, the model of Eq.(1) is the most accurate predictor with the least variability. The code provisions of both NDS/AU/NZ6303 and Eurocode 5 are not well suited to predict the bearing capacity. At 10% deformation, the model stress dispersion is well suited too. In comparison all the structural design codes are coarse and unreliable.

Outlook for Code improvement

Having noted the inability of current structural design codes to come even close in predicting accurately the bearing capacity, being coarse and inflexible compared to the stress dispersion model, it is time to discuss how to improve this situation. For the benefit of the timber designer, there are a number of options to improve the respective standards but all depend on to how far standard committees are willing to change them.

First to be considered is the difference in the types of test specimens. Test piece A, Figure 8, always produces the lowest strength values compared to types B and C for any strength definition. It is demonstrated by using modification factors derived with the stress dispersion model, to transform the results with test pieces B and C to type A (CEN) equivalent values. The model modification factors are 0.58 and 0.71 for B and C, respectively. These modification factors can also be derived experimentally. For Radiate Pine, Franke and Quenneville (2010) reported 0.61 and 0.67 based on a total of 150 tests, column 3, Table 2. The difference between both experimental and the stress dispersion method is small, approximately 5%. It actually means that having a 1% off set strength definition the experimentally determined strength of for instance piece C can be transformed to the

CEN specimen value by multiplying with 0.67.

The influence of differences in strength definition is resolved experimentally. To change the test results of the ISO and AS/NZ 2mm off set to the CEN 1% off set strength definition, reduction factors of 0.92, 0.84 and 0.86 apply to test pieces A, B and C, as in Franke and Quenneville (2010), column 4, Table 2. To transform the ISO/AS/NZ standard compression test values directly to equivalent type A (CEN) values, factors in column (5) can be used, which follow from multiplying columns (3) and (4).

The mean strength values obtained for the ASTM-D143 test piece B, with the 1mm (0.04") off set strength definition can be transformed to the 1% off set definition by applying a factor of 0.66 based on evaluation of tests by Ranta-Manus (2007), who reported 200 tests of Spruce-Pine, column (7), Table 2. Applying the ASTM-D143 strength definition of 0,01" deformation to test piece A mean test results of Hansen (2005) for Spruce results in a factor of 0,90. So horizontally one finds the change in strength definition and vertically the change in test specimen. In column (8) the modification factor is given to transform the ASTM standard values to type A (CEN) equivalent which results from multiplication of columns 3 and 7. Summarizing, column (5) and (8) contain modification factors to transform the standard test results of ISO 13910, AS/NZ 4063 and ASTM-D143 to equivalent CEN values. With these transformation factors in mind one is able to apply the the stress dispersion model in respective design standards.

This leaves standard committees with two options. The first is to modify the standard strength values with Table 2 modification factors and to introduce the stress dispersion model in the structural design code. The second is to move Table 2 modification factors to the structural design code and combine them with the k_c factor of the model. For the latter option, only the structural design code provisions need to be changed.

Conclusions

The model presented, enables accurate prediction of the compressive strength capacity. This model, in combination with the experimental analyses of Franke and Quenneville (2010), resolves the differences between the standard test methods of ASTM-D143, ISO13910 and AUS/NZS 4063 for which modification factors are derived. The inability of the major structural timber design code like EN1995-1-1, NDS and AU/NZ3603 to predict the compressive strength capacity for continuous supported beams accurately is demonstrated. It is argued that the adoption of the modification factors derived, in combination with the model presented, the bearing strength capacity can be predicted much more accurately than ever before.

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Table 1: Adjustment factors for bearing

ASTM-D2555							
Bearing length [mm]	13	25	38	51	76	102	152
k_c =	1.75	1.38	1.25	1.19	1.13	1.10	1.0
AS/NZ3603:1993							
bearing length [mm]	10	25		50	75	100	150
k_c =	1.90	1.60		1.30	1.15	1.06	1.0

EN 1995-1-1/A1:2008 Type	solid wood	glued laminated
	k_c	k_c
Not fully supported	1	1
Full supported	$1.25(l+2*30)/l$	$1.5(l+2*30)/l$
		with l= loaded length

Table 2: Modification factors

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
mean 50%	Num of Tests n*	CEN 1% off set	ISO/NZ 2mm off set	Transformation AS/NZ to CEN	Num of Tests n**	ASTM 0,04” off set	Transformation ASTM to CEN
A	90	1.00	0.92	0.92	30	0,90	0,90
B	30	0.61	0.84	0.51	200	0.66	0,40
C	30	0.67	0.86	0.58	-	-	-

n* Radiata Pine; Franke and Quenneville (2010); n** Spruce; Hansen (2005), Ranta Manus (2007)

Fig. 1**:** Examples of practice

Fig. 2: **S**tress-strain curve perp. to grain of wood.

Fig. 3: Test specimens

Fig. 4: Strength definitions

Fig. 5: Beam on continuous support

Fig. 6: Assumed dispersion of stresses for two load cases

Fig. 7: Stress dispersion for special situation

Fig. 8: Configuration A, B and C

Fig. 9: Test by Leicester (1998) and Model results

Fig 10: Test by Leicester (1998) and Model results

Fig. 11: Overview of tested load configurations.

Fig. 12: Test series at 1% off versus model prediction

Fig. 13: Test data at 10% deformation versus model predictions

Fig. 14: Variability of model prediction for 1% off set

Fig. 15: Variability of model deformations at 10% deformation

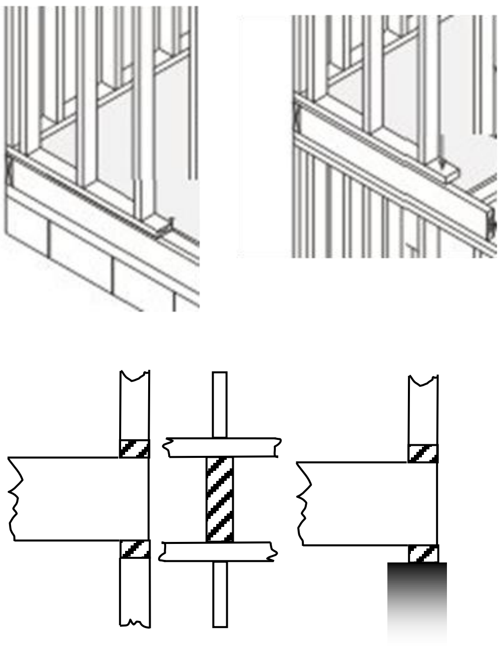

Fig. 1: Examples of practice

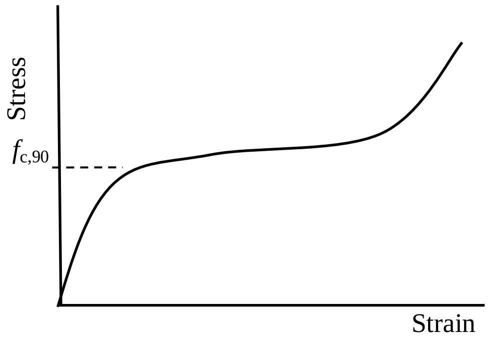

Fig. 2: Stress-strain curve perp. to grain of wood.

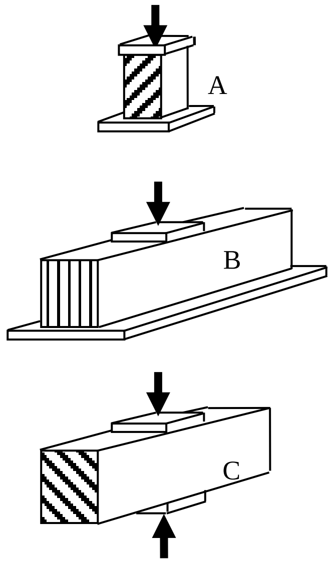

Fig.3: Test specimens

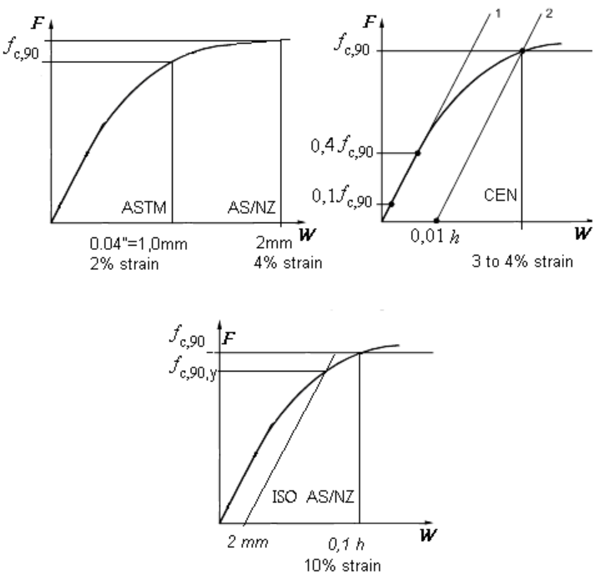

Fig. 4: Strength definitions

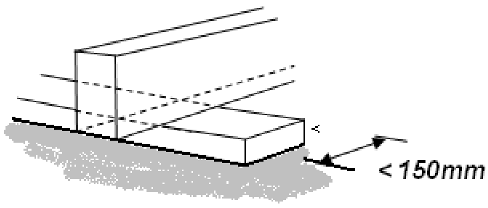

Fig. 5: Bearing on continuous support

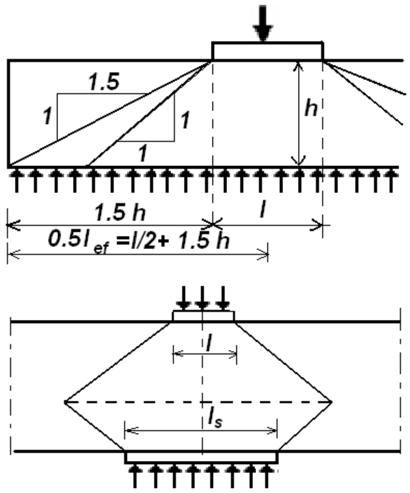

Fig. 6: Assumed dispersion of stresses for two load cases

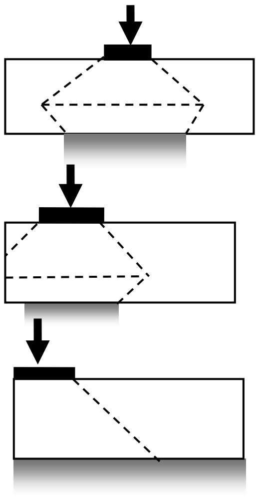

Fig. 7: Stress dispersion for special situation

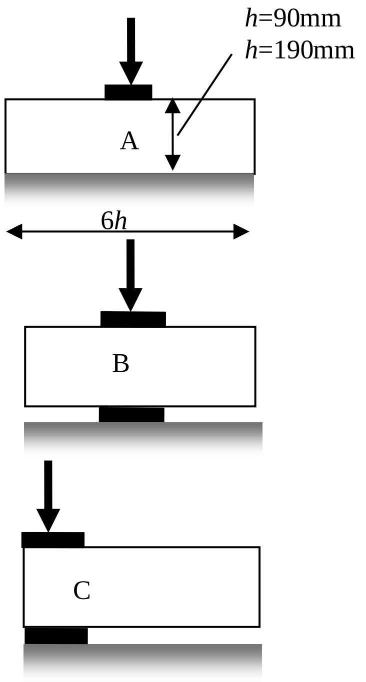

Fig. 8: Configuration A, B and C

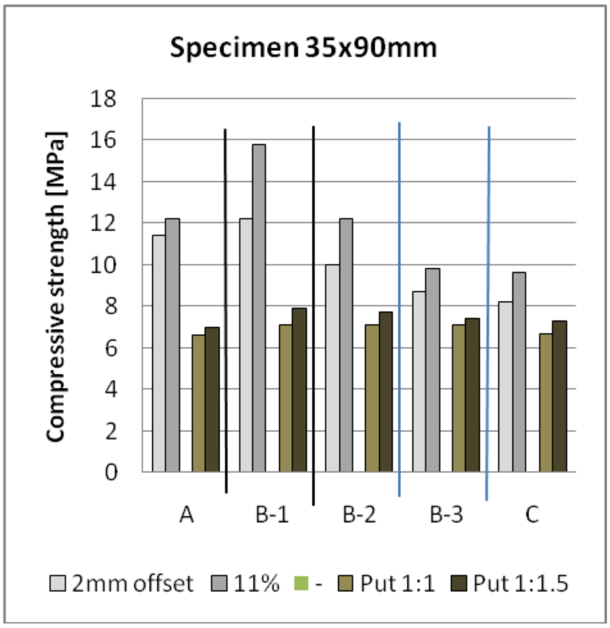

Fig 9: Test by Leicester (1998) and Model results

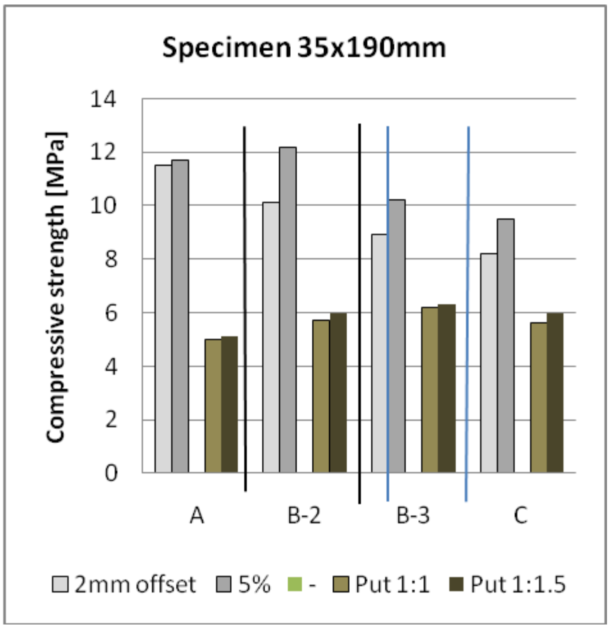

Fig 10: Test by Leicester (1998) and Model results

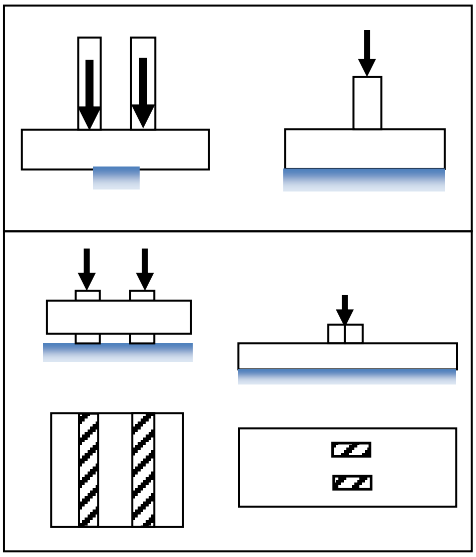

Fig. 11: Overview of some tested load configurations.

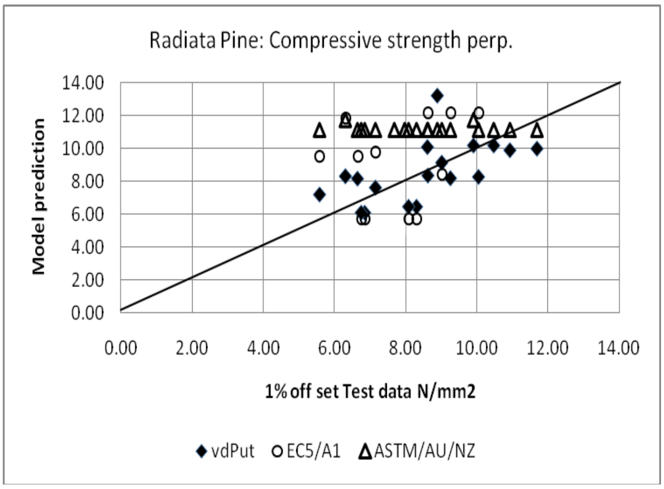

Fig. 12: Test series at 1% off set versus model prediction

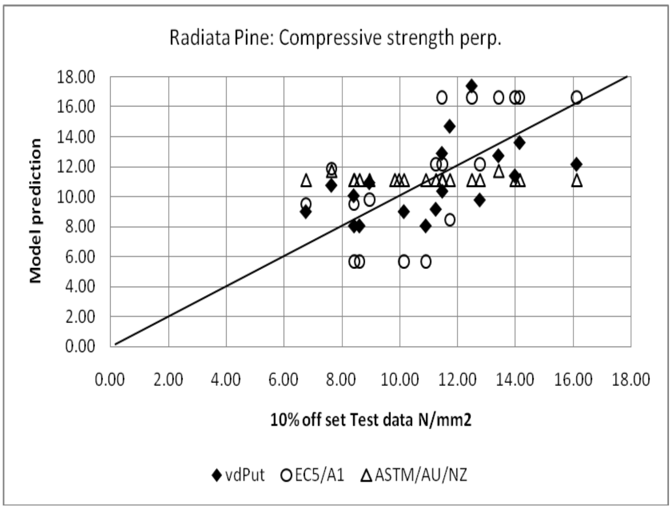

Fig. 13: Test data at 10% deformation versus model predictions

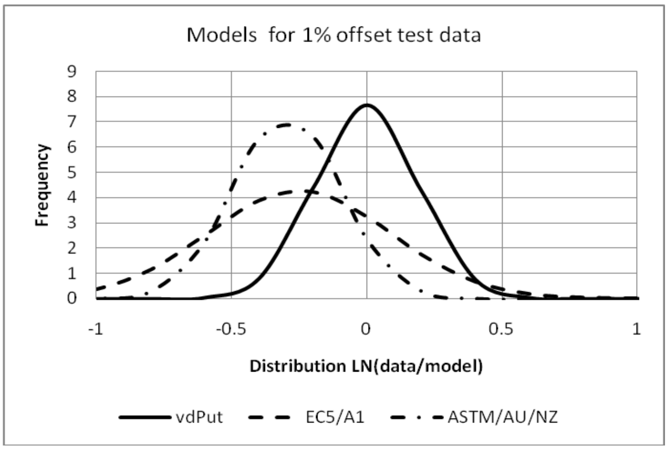

Fig. 14: Variability of model prediction for 1% off set

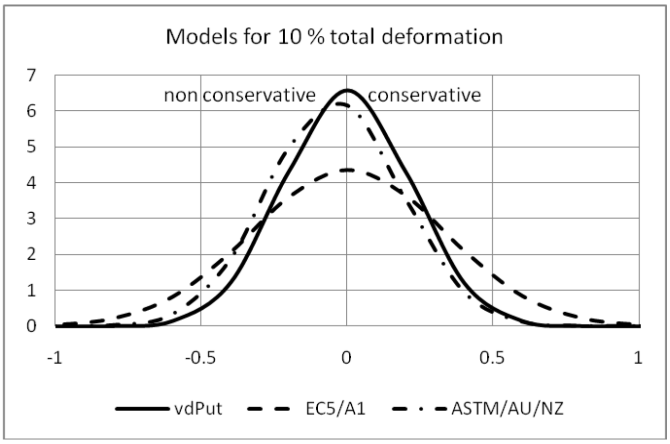

Fig. 15: Variability of model deformations at 10% deformation