4 CONNECTIONS

CONTENT

4.1 BLOCK SHEAR

ESSAY 4.1 H J Larsen
Block Shear

When a member is loaded by a group of mechanical fasteners close to the end there is a risk for failure because a plug or a block is torn out, see Figure 1.

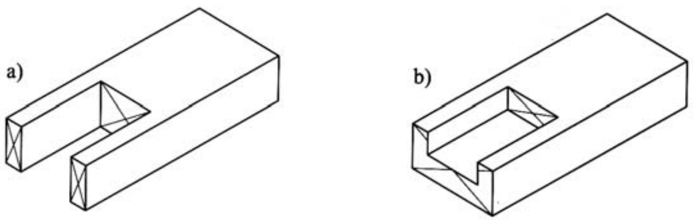


Figure 1. a) Plug shear. b) Block shear

There are two contributions to the load-carrying capacity: tension failure in end cross-section of the plug/block and shear failure in the rest of the failure surface. The deformations at failure are very different for te two failures. The strain-stress curve for tension is short and brittle and failure will take place long before a substantial part of the shear strength has been developed. Two failures are, therefore, investigated:

Tensile failure
Eurocode 5 recommends that the tensile strength is taken as $1{,}5f_{t0}$, i.e. the tension load-carrying capacity of the cross-section through the end nail line is ($_{bs}$ = block shear):

$$R_{bs} = 1{,}5\,f_{t,0}t_{pen}l_{net} \text{ where, see Figure 1, } l_{net} = \sum_i l_{t,i}$$

Often this capacity will be the bigger, but in some cases, the shear capacity is the.bigger.

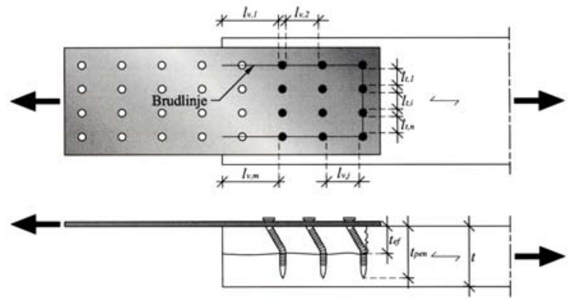


Figure 2. Block shear with tear out along the perimeter of the fasteners

Shear failure
Eurocode 5 recommends that the shear strength to take into account uneven stress distribution is taken as $0{,}75f_v$, i.e. the load-carrying for plug shear is:

$$R_{ps} = 0{,}7\,f_v t l_{net,v} \quad \text{where, see Figure 1,} \quad l_{net,v} = \sum_j l_{v,j}$$

For block shear the shear contribution is:

$$R_{bs} = 0{,}7\,f_{v,k}\,0{,}5 l_{net,v}\left(l_{net} + 2t_{ef}\right)$$

The effective thickness for thin steel plate is determined as

$$t_{ef} = \begin{cases} 0{,}4t_{pen} \\ 1{,}4\sqrt{\dfrac{M_y}{f_h d}} \end{cases}$$

The effective thickness for thick steel plate is determined as

$$t_{ef} = \begin{cases} 2\sqrt{\dfrac{M_y}{f_h d}} \\ t_{pen}\left[\sqrt{2 + \dfrac{M_y}{f_h d t_{pen}^2}} - 1\right] \end{cases}$$

30-7-2 J Kangas, A Kevarinmäki
Modelling of the block tearing failure in nailed steel-to-timber joints

Abstract
Block tearing failure mode has been found in the testing of nailed steel-to-timber joints with small nail spacings. The capacity of the nailed connection with given spacings depends on the nail capacity up to a certain joint length, after which the block tearing failure may occur, i.e. a block of timber will be cut out from the joint. The increase in the joint capacity due to additional nails will be less than the full capacity of nails.

The length of the connection is determined by this turning point, where the limiting factor of capacity changes from the capacity of nails to that of timber.

A common way to model this kind of behaviour has been to multiply the capacity of a single nail by a reduction factor after this changing point.

This paper presents a method for modelling the block tearing failure and for designing the capacity of the connection. The model depends on the block tearing mode and on the dimensions of the timber torn off from the timber member. The capacity of the joint is limited by the sum of the nail capacities and by the sum of the tensile and shear capacities of the timber in the joint area.

Nail spacings and distances can be chosen to achieve the optimum capacity of the joint, without the unfavourable block tearing failure mode occurring. The maximum capacity of the nails can also be utilized without any reduction in the effective number of nails. A suitable reduction of the nails will increase the capacity of the connection.

Concluding Remarks
In nailed timber connections, nail spacings are mainly given to avoid splitting during nailing. In the case of predrilled holes and steel-to-timber connections increased nail density is allowed by design codes. This leads to the exceeding of the tensile and shear capacity of the timber member in the joint area, and the unfavourable brittle mode of failure with reduced capacity of the joint.

The block tearing failure can be avoided by using the proposed method to model nailed steel-to-timber joints in tension. This method takes into account the dimensions of the torn off piece of timber member, and the tensile and shear capacity of timber in the joint area. Thus, the model can be used for optimizing nail spacings and distances of nailed joints made of different timber materials. The allowable nail spacings of Eurocode 5 are, therefore, found too small for nailed steel-to-timber connections, since the reduction in nail spacings due to both predrilling and steel connectors is not acceptable. In long timber joints, no reduction should be allowed at all. The reduction in the effective number of nails should also be replaced by increasing the joint area with maximum nail capacity or by determining the block tearing failure capacity of the joint area.

31-7-4 J Kangas
Design on timber capacity in nailed steel-to-timber joints

Introduction
In design codes, nail spacings are mainly used to avoid splitting during nailing. In the case of predrilled holes and steel-to-timber connections increased nail density is allowed. This may lead to the exceeding of the timber capacity in the joint area, and to the unfavourable brittle block tearing failure with reduced capacity of the joint.

Block tearing failure mode has been found in the testing of nailed steel-to-timber joints with small nail spacings. The capacity of the nailed connection with given spacings depends on the nail capacity up to a certain joint length, after which the block tearing failure may occur, i.e. a block of timber will be cut out from the joint.

The failure mode was first found, when MNC-connector was tested with KertoS-LVL. More test series with different timber materials, annular ringed shank nails, spacings and end distances were carried out to verify the model generally. Additional test series were conducted recently where the timber capacity was near the nail capacity of the connection.

This paper presents a simple method to design the timber capacity in nailed thick steel-to-timber connections. The capacity is limited by the sum of the tensile and shear capacities of timber in the effective timber area of the connection. It will also be proved that the proposed model explains the test results of glulam rivet connections much better than does the model of three different failure modes, which has been presented earlier.

Concluding Remarks
The allowable nail spacings of EC5 are, therefore, found too small for nailed steel-to-timber connections, since the reduction in nail spacings due

to both predrilling and steel connectors is not acceptable. In long timber joints, no reduction should be allowed at all. An increase in the end distance or in the joint length, however, could allow dense nailing patterns.

Design method of timber capacity for steel-to-timber connection with clusters of nails has been presented. The main task is to calculate the timber capacity in the joint. This method allows a simple design procedure for this type of connection. The procedure is advantageous, in that it permits to optimize the connection in the sense that premature wood failures are controlled and full use is made of the load-carrying capacity of the nails.

Failure loads predicted by the model compare well with those obtained in tests carried on with different timber materials, types and sizes of nails, sizes of the nailed area and end distances up to half a meter. Thus design recommendations presented here can be used in all nailed thick steel-to-timber connections.

34-7-10 A J M Leijten, J. Kuipers , A J M Jorissen
Interaction between splitting and block shear failure of joints

Introduction

Recently, a new failure mode for connections was introduced in the draft Eurocode 5. It refers to timber failure along the circumference of the fastener pattern, so-called block-shear. This failure type mainly governs steel-to-timber connections with close spaced fasteners. The draft also contains a splitting failure criterion for connections loaded perpendicular to the grain as presented in **Paper 33-7-7**. It is implicitly assumed that both criteria apply independently and therefore there is no interaction equation needed. Some experimental evidence is given in this paper to justify this approach.

Eurocode 5 (EC5) was revised during the last years based on comments of the member states. As a result of the discussion based on these comments, it was decided to add and modify a few criteria related to the strength capacity of connections. Particular when connections are loaded near end grain face and subjected to a force at an angle to the grain, the force components parallel and perpendicular to the grain can force separate failure modes. For the force component parallel to grain a failure criterion, known as the block shear, is introduced in EC5. Block shear is a brittle type of failure recognised by cracks along the circumference of the fastener area. A related phenomenon is shear plug failure. Another criterion is the splitting strength of beams caused by a force component perpendicular to grain based on **Paper 33-7-7**. No interaction is assumed between block shear and splitting criterion. The only experimental evidence as far as the author knows that supports and justifies this approach dates back to investigations by Kuipers and Schippers from 1960 and 1962, which deals with split-ring connections loaded at various angles to the grain. The investigations were never published other than in a Dutch Stevin Report. The paper contains a summary of their work.

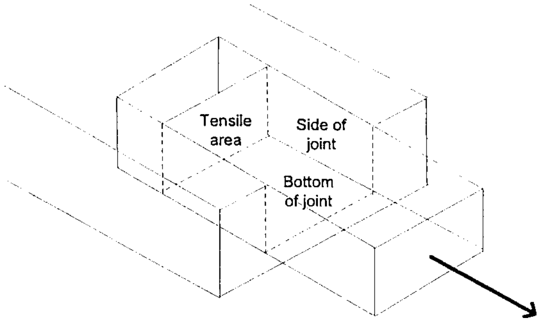
Conclusion

It can be concluded that on the basis of the test results with split-ring connectors outlined above block shear and splitting failure can be considered as two independent failure mechanisms.

36-7-2 H Johnsson
Plug shear failure in nailed timber connections: Experimental studies

Abstract

Brittle failures in mechanical timber joints should be avoided, because this often results in low resistance and brittle failure of the structure. Nailed joints experience three ultimate failure modes: embedding, splitting or plug shear failure. Plug shear failure is limiting for large nailed connections loaded in tension parallel to the grain. To avoid plug shear failure, short and wide joints are preferred, minimising the number of fasteners in line with the load and grain direction. The aim of the study is to evaluate existing prediction formulas. Furthermore, knowledge of the parameters governing the plug shear failure is sought. Plug shear failure was examined in short-term experiments on nailed steel-to-timber joints in glulam loaded in tension parallel to the grain with varying joint geometries. Test results from four different test series are presented. Using hypothesis testing, a suitable prediction formula is derived based on more than 70 experiments. It is concluded that for engineering purposes the existing modeling approach is sufficient and plug shear resistant, for the shear failure mode is modeled by the shear resistance of the bottom area of the joint.



Schematic plug shear failure

Conclusions

64 experiments on plug shear failure in nailed steel-to-wood connections were conducted. Based on experimental observations and the assumptions presented regarding material parameters, the model for predicting the characteristic plug shear resistance in the shear failure mode the following expression is suggested

$$R_{v,k} = 0,8blf_{v,k}$$

This model differs from the one suggested in Eurocode 5 (2003), Annex A, but leads to a safer design.

37-7-6 H Johnsson
Plug shear failure

Abstract

Plug shear failure in nailed timber connections is a brittle failure mode, which limits the capacity for nailed joints loaded in tension parallel to the grain. The limiting strength parameter for plug shear failure is for most cases the shear strength of timber. However, for short joints a tensile failure mode is possible. The occurrence of plug shear failure is closely linked to the spacing between the nails and in particular to the spacing perpendicular to the grain.

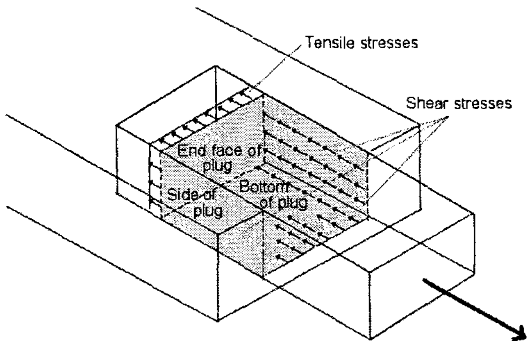
Plug shear failure was studied in short-term experiments on nailed steel-to-timber joints. The occurrence of the tensile failure mode was shown as well as the effect of increased spacing, which increases the resistance and alters the course of failure. A prediction model is proposed for the tensile failure mode. A complete prediction model for plug shear failure in nailed joints is presented together with recommendations on nail spacing.

Introduction

When timber is stressed by a group of fasteners loaded in tension parallel to the grain it results in both tension and shear stresses parallel to the grain, see the figure, where the bottom and side faces of the plug are in shear. The resistance of the joint is the lowest value of the nail embedding and the plug shear resistance, which involves tensile (R_t) and shear (R_v)capacities. In **Paper 36-7-2** the earlier models of Kangas and Vesa and Foschi and Longworth were evaluated and a model was proposed for predicting the plug shear resistance based on experimental evidence, Eqn. 1.

$$R = blf_v \quad \text{where} \quad f_v = K\left(bl\right)^{-0,25} \tag{1}$$

In (1) b is the width of the plug, l is the length of the plug including the end distance and f_v, is the shear strength of timber. The parameter Kdescribes the volume effect of shear strength and was experimentally determined to $K = 64,5$.



Schematic Plug Shear Failure

Eqn. (1) describes the resistance of the shear failure mode of plug shear failure and in **Paper 36-7-2** the course of plug shear failure was described:

1. A crack develops internally along one side of the plug. The failure is initiated at the nail farthest from the free end.
2. The crack reaches the free end and is visible on the edge. The same development goes for the other side of the plug. This occurs in two-thirds of the experiments before the ultimate load is reached.
3. The end face of the plug fails in tension.

4. The final failure occurs when a shear crack along the bottom face of the plug joins the two side cracks.

For short (in the direction parallel to the grain) joints, a tensile failure mode should be possible. In short joints, the area loaded in shear is small and the capacity according to (1) is lower than the tensile capacity of the end face of the plug.

The first aim with this paper is to investigate the occurrence of the tensile failure mode in nailed timber joints.

The second aim is to study the effect of spacing on the occurrence of plug shear failure. This study is limited to comparing cases where spacing is either $7d/3.5d$ or $10d/5d$, where d is the diameter of the nail. In the expression $7d/3.5d$, $7d$ is the spacing parallel to the grain and 3.5d is the spacing perpendicular to the grain.

37-7-7 M Kairi
Block shear failure tests with dowel-type connection in diagonal LVL structure

Introduction

The low transverse tensile strength of wood in contrast to the tensile strength in grain direction is a common problem in dowel-type connections. Several alternative solutions have been presented, mainly based on the strengthening of the connection area with special plywood, fibre reinforcement and densified timber or nail plates. However, the transverse tensile strength of the basic material could be strengthened already in the production stage of the Laminated Veneer Lumber (LVL) by bedding the veneers diagonally.

Two new versions, LVL-DA and LVL-DB, has been developed to diagonal LVL. Their bending and tensile capacity as beam system are essentially higher than those of previously presented diagonal LVL.

The failure modes of wood with dowel-type connection, when wood is loaded with tensile force, are presented in Figure 1. The failure mode is dependent on:

– edge and end distances of the dowels
– the slenderness of the dowels in relation to the thickness of wood
– the number of the dowel next to each other and in a row and also on distances between them

a) Embedment failure (plastic displacement > 0,02·d)
b) Splitting failure in grain direction
c) Plug shear failure
d) Block shear failure
e) Tension failure

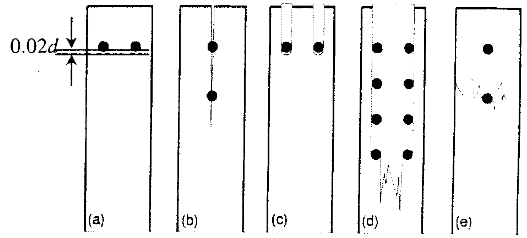


Figure 1. Failure modes in wood for connection with dowel-type fasteners according.

The primary aim is to avoid the occurrence of block shear failure, which is considered critical by using the new LVL structure. In this study the failure mode of a jointing area with unbending bolts is analysed for two different LVL-structure. The results are compared with LVL standard structure.

Because solid wood and also standard LVL are brittle materials, the joint will behave brittle as well. The only way in this situation is to provide ductility with mechanical fasteners forming plastic hinges. The connections are design on terms of wood so that the slenderness and the number of the dowels are optimised. The aim is to reach the yield point of the steel before failure of wood. Analysing the ductility of the joint Gehri has developed a concept for ratio of ductility $D = W_u/W_y$, figure 2. It is the ratio between the plastic and elastic displacement. For a good connection the ratio of $D > 3$ is proposed:

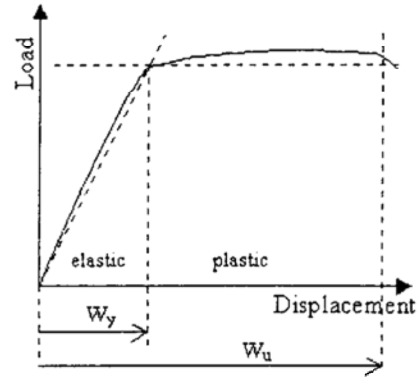


Figure 2. Principal drawing of ductility ratio according to Gehri (1996).

Conclusion
With the new LVL-D structure the block shear failure can be avoided even when using stiff fasteners in dowel-type connections with wood. Also the ductility ratio of 4 can be reached. The new structure can reach up to 40 % higher characteristic failure load in tension than LVL standard.

39-7-4 A Hanhijärvi, A Kevarinmäki, R Y Koski
Block shear failure at dowelled steel-to-timber connections

Introduction
The design of dowel type joints of timber is well established in the Eurocode 5 by the use of the Johansen theory. It has shown to perform well when the fastener diameter is small and consequently the fasteners are slender. With the increase of span length in large timber structures, the use of high capacity dowel type joints is necessary. The high capacity dowelled connections are often implemented by slotted in steel plates and large diameter dowels or bolts. With the increase of the diameter the rigidity of the dowel increases more than the embedment capacity. Therefore with large diameter rigid dowels, also the failure of timber at the joint area becomes more easily critical for the capacity of the connection – not only as embedment failure but through failure of the whole joint area by tension or shear. The failure mechanisms in this manner at the connection area are known as block shear or plug shear.

The lack of design against timber failure as consequence of shear and tension at the connection area (block shear) was found to be the partial reason for a recent failure of a large roof structure in Finland. Although the primary reason for the failure was a manufacturing fault in one connection (missing dowels) of a large glulam truss, the failure would not have proceeded to a catastrophic one, unless the true capacity of the properly manufactured joint had not been much lower than the capacity assumed in the design, which did not include any consideration of timber failure at the joint area. The true capacity of the connection was tested later after the collapse in a full-scale test, in which it was found that the true capacity was only app. 50% of the design value. The tests showed also clearly the importance of the block shear failure mechanism as the critical one. At the time of the design of the roof, the ENV-version of the Eurocode 5 did not contain any mention of this type of failure mechanisms.

The aim of the present work is to improve the grounds for design of heavy-duty dowelled connections by experimental investigation of the timber failure at the joint area in joints implemented by steel plates loaded in tension. Altogether more than 150 tension tests were made with glulam and Kerto-LVL specimens. The experimental program contained tests of both double-shear and multiple shear (4- and 6- shear) plane connections. In double shear, both timber-steel-timber and steel-timber-steel specimens were tested. In multiple shear the outermost parts were always timber.

Conclusions
Based on the large experimental data that has been gathered by the loading of more than 150 specimens and 300 connections, the following recommendations ran be given for the development of the design of dowelled timber-to-steel connections:

- The plug shear failure mechanism does not occur in the connection area contrarily to what the design equations in EC5 suggest. This is due to the fact that the relatively rigid dowels remain straight or bend very little before failure, which is then mostly due to the block shear mechanism. Failure by plug shear would probably require the development of fully developed plastic hinges, which does not occur at the failure level of block shear. The design equations assume the fully developed plastic hinges in order to determine the failure mechanism based on the Johansen theory.
- If the mean material property values are substituted in the design equations of EC5, the value of F_{sn}, and F_{Bm}, representing splitting or row shear failure and block shear or plug shear failure, respectively, are often very close to each other. In many cases the value of F_{sn}, is lower than F_{Bm}. However, in these experimental tests no pure splitting failure was observed and row shear in only few series. This indicates that the equation for the reduction effect of the number of dowels in a row is too conservative for these connections, because it does not take into account the slenderness of the dowels.
- Connections with cross-veneered Kerto-Q-LVL showed much higher experimental capacities than could be anticipated from the design calculations using the characteristic values in EC5. This is due to the fact that the plug shear failure does not occur because of rigid dowels and because the low flat wise shear strength reduces the capacity dramatically in case of plug shear.

– When the calculated failure mode of EC5 is block shear, it can be concluded that the formulas result usually in clearly conservative design for glulam, but they are approximately on the right level for Kerto-S-LVL. However, the following additional condition should be given for the failure mode block-shear with shear capacity higher than tension: it works only if the edge distance is sufficiently large so that the tensile capacity of the outermost timber strips is enough to carry the whole failure load. It is apparent that, in most block shear failure cases, a simultaneous combination of tensile and shear stress is acting.

40-7-3 A Hanhijärvi, A Kevarinmäki
Design methods against timber failure mechanisms of dowelled steel-to-timber connections

Introduction

The design of dowel type joints of timber is well established in the Eurocode 5 by the use of the Johansen theory. It has shown to perform well with small diameter slender fasteners. In large structures, with the increase of span length, the use of high capacity connections is necessary. High capacity dowelled connections are often implemented by slotted-in steel plates and large diameter dowels or bolts. With increasing diameter, the rigidity of the dowel increases more than the embedment capacity. Thus, the failure of timber at the connection area becomes more easily critical for the capacity of the connection – not only as embedment failure but through failure of the whole joint area by tension or shear. These shear failure mechanisms are known as block shear, plug shear or row shear. The design for these failure mechanisms is not yet sufficiently well established.

The aim of the present work was to improve the grounds for design of heavy-duty dowelled connections by a large experimental program and consequent development of design model against the timber failure mechanisms in connections implemented by steel plates and loaded in tension. The experimental program consisted of more than 150 tension tests on glulam and LVL-specimens. This paper deals with the design method that was developed based on the experimental results.

The proposed design procedure presents a method to design dowelled timber-to-steel connections against timber failure mechanisms. The method pursues to distinguish all failure modes and present a sufficiently accurate equation for each one. This methodology chops up the complex failure phenomenon to many rather simple equations which all have reasonable purpose and derivation. Although the method is suitable for hand calculation, the best way to apply it is through a calculation spreadsheet or a small computer program, for which it is extremely suitable, since the method does not require iteration or interpolation.

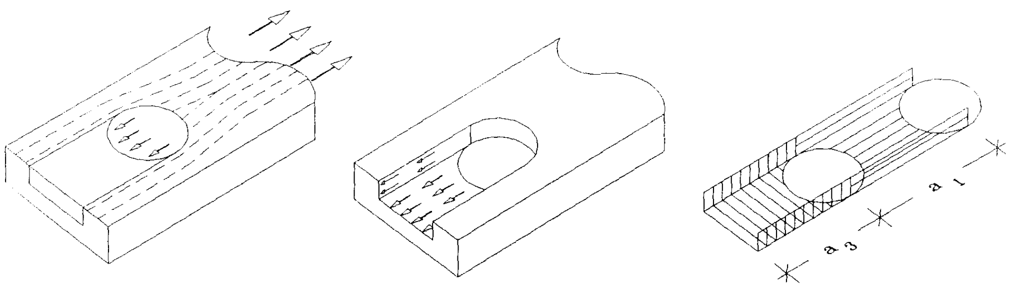
The method has so far been verified only against the test results of this project for which is fits surprisingly well. More verification calculations would be advantageous but are limited by the small amount of experimental data available. However, the model should be verified properly against situations with only one row of dowels (not included in this project), which would give more insight to its performance in modelling the splitting behaviour.

4.2 CONNECTORS

25-7-6 H J Blass, J Ehlbeck, M Schläger
Characteristic strength of split-ring and shear plate connections

Introduction
During the last CEN TC 124 WG4 meeting in Trento, Italy, the convenor of WG4 presented a calculation model describing the load-bearing behaviour of split-ring and shear plate connections subjected to tensile forces parallel to the grain. Using this model existing test data are evaluated to determine characteristic strength values of split-ring and shear-plate connections. The result of the evaluation is compared to today's allowable loads of this type of mechanical timber connections.

Calculation model
The model used to describe the failure of split-ring and shear-plate connections assumes a shear block failure of the wood in front of the connector. The embedment stresses which in reality are unevenly distributed over the half circle of the split-ring are assumed to be uniformly distributed and acting parallel to the load direction. The embedment stresses are then transferred through shear stresses into the tension member (see Figure). For tension members the capacity of the bolt is neglected, since the bolt usually is placed in oversized holes and only just starts bearing when the split-ring connection fails.



Conclusions
In order to establish characteristic strength values of split-ring and shear-plate connections old test data have been evaluated. The results of the evaluation show the suitability of a calculation model presented and discussed in Working Group 4 of CEN TC 124. This model assumes a block shear failure mode for joints loaded in tension. The influence of the bolt on the load-bearing capacity is neglected. Besides, the model provides consistent results also for joints loaded under an angle to the grain. For joints loaded in compression, the capacity of the bolt may additionally be taken into account.

An influence of number of connector units per joint could not be found within the range covered by the test data. The same applies to the influence of the angle between load and grain direction. For angles up to 70° the 5-percentile value of the strength per connector was independent of the load-grain angle.

25-7-7 H J Blass, J Ehlbeck, M Schläger
Characteristic strength of tooth-plate connector joints

Introduction
The change of code formats in European timber codes from an allowable load format towards a partial safety coefficient format requires characteristic strength values of the material and of connections. For joints with pin-type fasteners, the characteristic connection strength can be calculated using a solution based on the work of Johansen. Eurocode 5 provides those equations to calculate single and double-shear joints with nails, screws, dowels or bolts based on the joint geometry and the strength of the timber and the fastener. For other types of mechanical timber joints, characteristic strength values have still to be determined. The members of CEN TC 124 WG4 have the task to establish characteristic strength values for connector joints. During the last CEN TC 124 WG4 meeting, a calculation model describing the load-bearing capacity of tooth-plate connections was presented. Using this model existing test data are evaluated to determine characteristic strength values of tooth-plate connections. The result of the evaluation is compared to today's allowable loads of this type of mechanical timber connections.

Calculation model
The model used to describe the load-carrying capacity of tooth-plate connections is based on the assumption of a load-sharing between tooth plate connector and bolt. The connection strength can therefore be described by:

$$R_{j,k} = R_{c,k} + \eta R_{b,k}$$

with

$R_{j,k}$ characteristic load-carrying capacity of the tooth plate connection containing both tooth plates and bolt

$R_{c,k}$ characteristic load-carrying capacity of the tooth plate connector

$R_{b,k}$ characteristic load-carrying capacity of the bolt according to Eurocode 5

η factor between 0 and 1 to account for the effect of load distribution between connector and bolt.

An evaluation of the allowable loads of tooth-plate connectors according to DIN 1052 resulted in the following relation between the load-carrying capacity of a circular tooth-plate connector and its diameter:

$$R_{c,k} = A\, d^{1,5}$$

where

A factor depending on the connector type (to be determined through tests)

d_c connector diameter.

Conclusions

In order to establish characteristic strength values of Bulldog tooth-plate connections old test data have been evaluated. The results of the evaluation show the suitability of a calculation model presented and discussed in Working Group 4 of CEN TC 124. This model assumes a load-sharing between bolt and connector. The characteristic load-bearing capacity of the connector has been determined on the basis of the assumption of a complete load-sharing.

An influence of number of connector units per joint could not be found within the range covered by the test data. The same applies to the influence of the angle between load and grain direction. Hence, the design proposal is based on independency between characteristic load-bearing capacity of a Bulldog tooth-plate connection and load-grain angle or number of fasteners, respectively. This applies to connections with up to three connector units.

26-7-2 C J Mettem, A V Page, G Davis
Validatory tests and proposed design formulae for the load-carrying capacity of toothed-plate connectored joints

Abstract

An earlier paper [Davis, G. and Mettem, C.J: Design rules for the UK timber connectors, CEN TC 124/WG 4, February 1993] reviewed the basis of design data for ring connectors, plate connectors and toothed-plate, connectors in the permissible stress design code, BS 5268. It then developed formulae for calculating the characteristic load-carrying capacities of timber-to-timber joints made with these connectors. These formulae, which were intended for use with EC5, produced values which were related to the permissible long-term loads given in BS 5268, but they were not related directly to experimentally measured, ultimate loads.

This paper describes experimental work which was undertaken on timber joints made with toothed-plate connectors. The purpose was to provide some additional validatory tests to give a better basis to evaluate various formulae which have been proposed for calculating their load-carrying capacity. It is important that such formulae should be as simple as possible, and should predict with acceptable accuracy the measured load-carrying capacities of real joints, with safety levels similar to those associated with similar formulae for nails, bolts and other dowel-type fasteners. For the load-carrying capacity of any dowel-type joint, on accuracy of ±10% has previously been considered acceptable.

In the first stage of this research, an investigation was made of bolted joints, which were assembled in a similar configuration, and using similar bolts, to those used later for the connectors. The results of these bolt tests were compared with the now well-established EC5 formulae. There was a significant difference between the experimental load-carrying capacity and that predicted by theory, which is explained in the paper.

The second stage of the programme consisted of tests on a range of three-member, double shear joint specimens containing toothed plate connectors. The effects on load-carrying capacity of connector size, member thickness, timber species and density are being evaluated from these tests, and these factors are considered in this paper.

The measured values of the load-carrying capacities of the toothed-plate joints, and values previously reported from tests carried out in 1952 and 1961 were then compared with values produced by three formulae. The first was a formula proposed by Blass and Schläger (Trial calculations

for determination of the load-carrying capacity of joints with bulldog connectors, CEN TC 124/WG 4, April1993), the second a development of this formula based on work previously reported by the authors, and the third was a new, simplified formula.

On average all three formulae predicted ultimate loads within 3 % of the test values. However, only the second and third formulae correctly assessed the effects on joint strength of timber density and member thickness.

Since some design formulae which have been proposed involve separating the contribution of the bolt, it was decided to measure the strength of plain bolted joints made from timber of comparable density, and with bolts of the same two diameters that were used for the connectored joints. As mentioned in the Introduction, it was discovered that the ultimate loads on these bolted joints considerably exceeded the values predicted by the formulae in EC5, so two further sets of tests were conducted using bolts without nuts and washers, one with members of European redwood, the other with Southern pine in order to include a timber of higher density. Some of the bolts themselves were tested to determine their ultimate tensile strength.

A further series of tests on toothed plates manufactured was conducted by Lee and Lord (TRADA report E/TR/21,1961). These plates were double-sided round toothed plates, similar to the other plates tested in 1993.

On the basis of these tests, Formula (1) gives the most consistent and accurate results, but it must be noted that all the specimens tested had densities of around 500 kg/m^3 and were made with thin members of similar thickness; and, as previously noted, the formula as it stands does not adequately allow for variations in these parameters.

$$R_{j.k} = 18 d_c^{1,5} + R_{b,k} \qquad (1)$$

$$R_{j.k} = 0,0014 t_{\min}^{0,5} \rho_k^{1,25} d_c^{1,5} + R_{b,k} \qquad (2)$$

$$R_{j.k} = 0,0077 \rho_k d_c \left(t_{\min} + 40\text{mm}\right) + R_{b,k} \qquad (3)$$

where

$R_{j.k}$ is characteristic load-carrying capacity of one double-sided round toothed plate and bolt, in N

$R_{b.k}$ is characteristic load-carrying capacity of the bolt per shear plane, calculated from Eurocode 5, in N

d_c is diameter of toothed plate, in mm

t is minimum of outer member thickness and half the inner member thickness for 3-member joints or minimum member thickness for 3-member joints, in mm

ρ_k is characteristic density, in kg/m^3.

All three formulae over-compensated for connector diameter according to these test results.

Conclusions

For real bolted joints made with nuts and washers, the bolt design formulae given in EC5 give good predictions of the yield loads but not of the absolute ultimate loads. The predicted loads appear to be about 70 % of the experimental values. It may be considered that design formulae for joints made with toothed plate connectors should in a similar way predict load-carrying capacities which are less than the ultimate load-carrying capacity.

27-7-10 A V Page, C J Mettem
Characteristic strength of UK timber connectors

Introduction

In an official report to the meeting of CEN/TC 124 'WG4", the members were asked

(i) to describe a procedure for converting the allowable or working loads for timber connectored joints used in their country to characteristic load-carrying capacities

(ii) to compare the values obtained in this way with corresponding values calculated from formulae proposed by E. Gehri:

$$R_{c,k} = 15\, d_c^{1,5} \left(\frac{\rho_k}{380}\right)^{0,5} + R_{b,k} \quad \text{N}$$

where

d_c is the diameter of the plate in mm

ρ_k is the characteristic timber density in kg/m^3

$R_{b,k}$ is the characteristic load-carrying capacity of the bolt in single shear in N

(iii) to make proposals for obtaining characteristic values for the connectors used in their country.

The purpose of this paper is to provide an answer to these three points for the UK-

Conclusions

The permissible fastener loads parallel to the grain, which are tabulated in BS 5268: Part 2, are based on test data which relate them to the density and thickness of the timber members, as well as to the size and type of connector. The formulae proposed by E Gehri were modified to allow for these influences in line with the original test data, and the results were then compared with values from BS 5268 converted by a factor of 2.9 to nominal characteristic values. The results were, in most cases, well within 10 % of each other.

4.3 DOWEL-TYPE FASTENERS LOADED PARALLEL TO GRAIN

ESSAY 4.2 H J Larsen
Dowel type fasteners

Load-carrying capacity, general

The design of laterally loaded dowel-type fasteners in Eurocode 5 is based on the work of K. W. Johansen, first described (in Danish) in [Johansen,1941][1] and [Johansen,1949][2]. The theory often called The European Yield Model (EYM) is mentioned in **Paper 12-7-2**, but since it plays an important role for Eurocode 5 and many CIB-W18 papers it is briefly described in the following.

The very simple theory assumes that failure is ductile and caused by yielding in the fastener or by compression failure of the wood and not by brittle failure e.g. by splitting of the wood. This is normally ensured by requirements to timber sizes and to minimum spacings and distances to end and edge.

The principal behaviour is illustrated in Figure 1 for a so-called double shear connection (two side members and one middle member). The main part of the load is transferred by contact pressure between the timber members and the dowel that is exposed to shear and bending. Part of the load may be taken by direct tension in the inclined dowels and by friction between the timber members.

It is assumed that the dowel acts as a beam laterally loaded by a constant contact pressure q per unit length. The relation between the contact pressure and the deformation may be found by the test set-up sketched in Figure 2: A stiff steel cylinder in a hole in a timber member is loaded by a force F. The figure shows a typical load-deformation curve. At the beginning there is a linear relation between load and impression, followed by a curved part after which the load falls slightly by increased deformation. In practice a stiff-ideal plastic behaviour with yield value F_y may be assumed.

---

1. Johansen, K.W.: Forsøg med Træforbindelser. Bygningsstatiske Meddelelser. Vol. XII, Nr. 2, pp. 29-86.
2. Johansen, K.W.: Theory of timber connections, International Association of Bridge and Structural Engineering (IABSE), Basel, Publication 9, 1949.

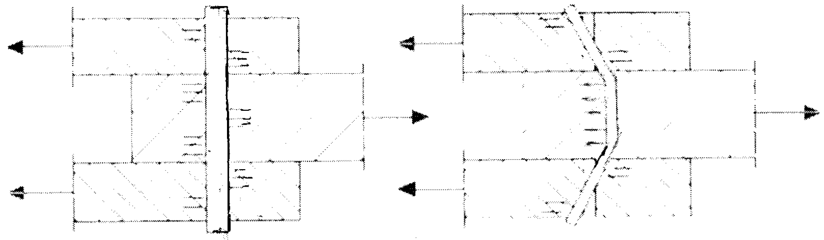


Figure 1. Double shear dowel connections. Thick dowels remain straight and the load is transferred almost solely by shear in the dowel. Slender dowels bend and part of the load may be taken by tension in the inclined dowel parts and by friction between the timber parts that are pressed together by the tension forces.

The so-called embedding is defined by:

$$f_h = \frac{F_y}{d\,t} = \frac{q_y}{d} \tag{1}$$

where F_y is the total load and q_y is the load per unit length when the wood material starts to yield (to get permanent deformation because of some type of fibre damage) and d is the dowel diameter. The embedding strength depends first and foremost on the compression strength (and thereby on the density ρ of the wood) and on the angle α between the load and the fibre direction. Also the dowel form, surface and diameter play a significant role.

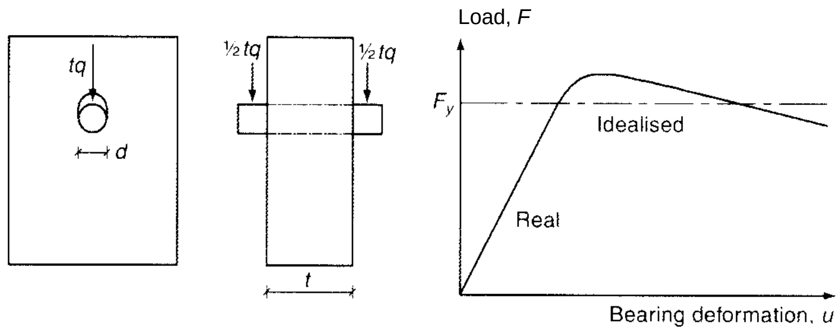


Figure 2. Dowel pressed into a timber member, and the real and ideal-plastic load-deformation curve.

The load-carrying capacity also depends on the yield moment M_y of the dowel. Assuming ideal-plastic behaviour of both wood and dowel, the dowel behaves either as a stiff unit without bending deformation or as stiff dowel parts that are joined by yield hinges. The possible failure modes for single and double shear joints are shown in Figure 3.

In the single shear joint, the dowel will either remain straight (failure modes 1) or bend in one or two yield hinges (failure modes 2 or 3 respectively). For the double shear joint failure, modes 1 correspond to a translation of the dowel either in the side members or the middle member. In failure mode 2, two yield hinges occur in the middle member while the dowel remains straight in the side members. In failure mode 3 four yield hinges are formed within the dowel, two in the middle member and one in each outer part.

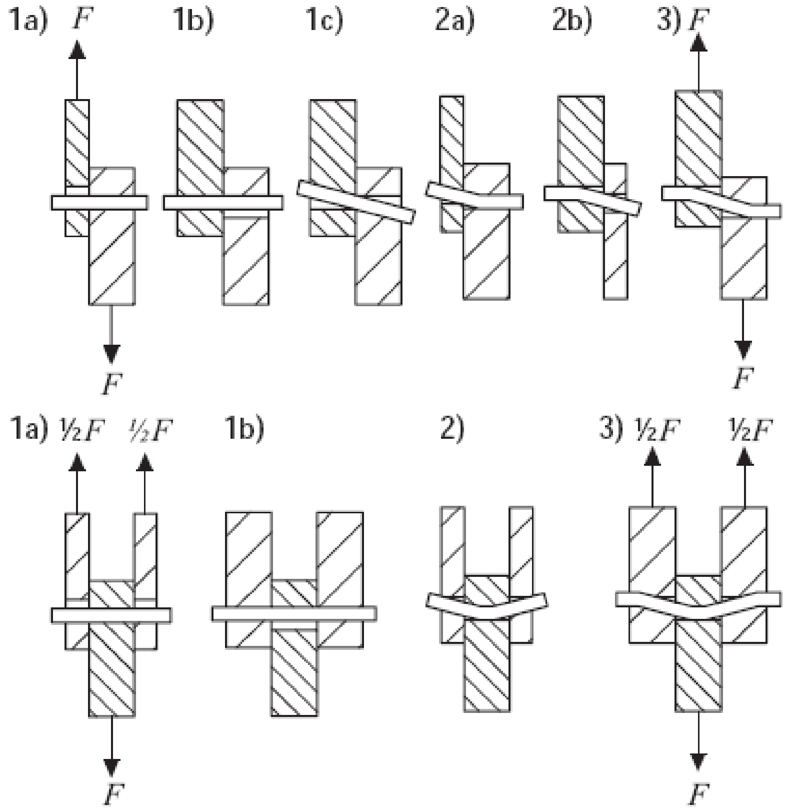
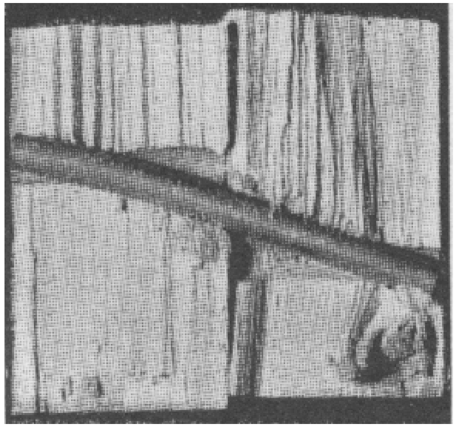
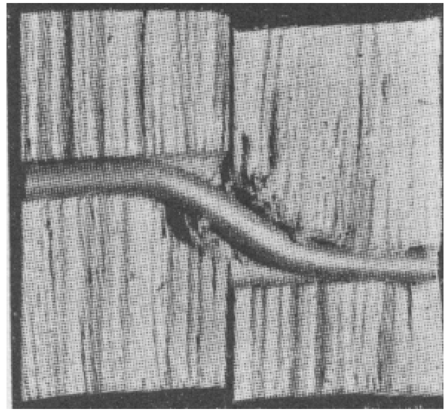


Figure 3. Failure modes for single (upper row) and double shear joints (lower row).

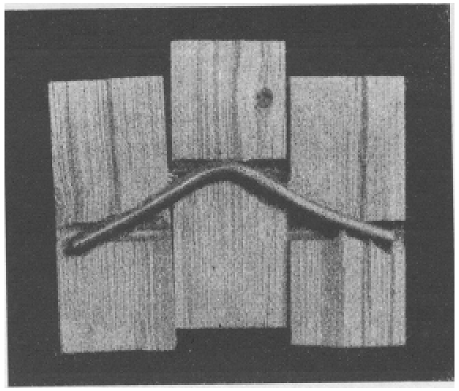
Figures 4 and 5 show examples of K.W. Johansen's test specimens opened after failure. The difference in failure modes in figure 3 is due to the embedding strength being considerably higher for oak than for spruce.



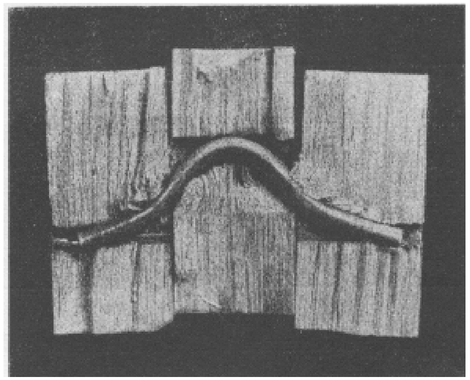
a) Single shear joint, 10 mm dowel in spruce



b) Single shear joint,10 mm dowel in oak



c) Double shear joint, 10 mm dowel in spruce



d) Double shear joint,10 mm dowel in oak

Figure J.2.4. Photos from tests with dowels in different wood species. From [Johansen, 1941].

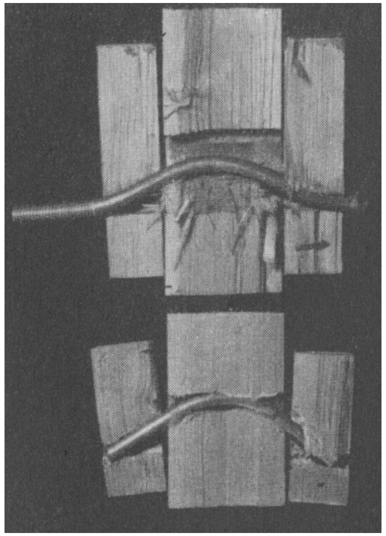


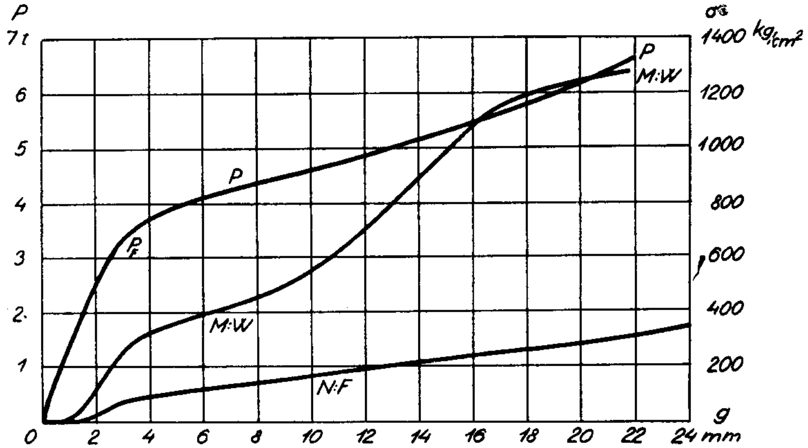
Figure 5. Failure modes for bolt (top) and dowel (bottom). From [Johansen, 1941].

Figure 5 illustrates the difference between a bolt and a dowel. For bolts, head and nut with washers reduce the deformation at the surface. This may change the failure mode to one with a higher load-carrying capacity. This is however not taken into consideration in Eurocode 5.
With increasing load first elastic deformation develops creating tension in the fastener due to an inclination of the fastener axis. A prerequisite for significant tensile forces in the fastener is the anchorage of the fastener in the member, e.g. by head and nut in bolts or by thread and head in nails or screws. This tensile force in the fastener presses the members together and causes friction between the members.

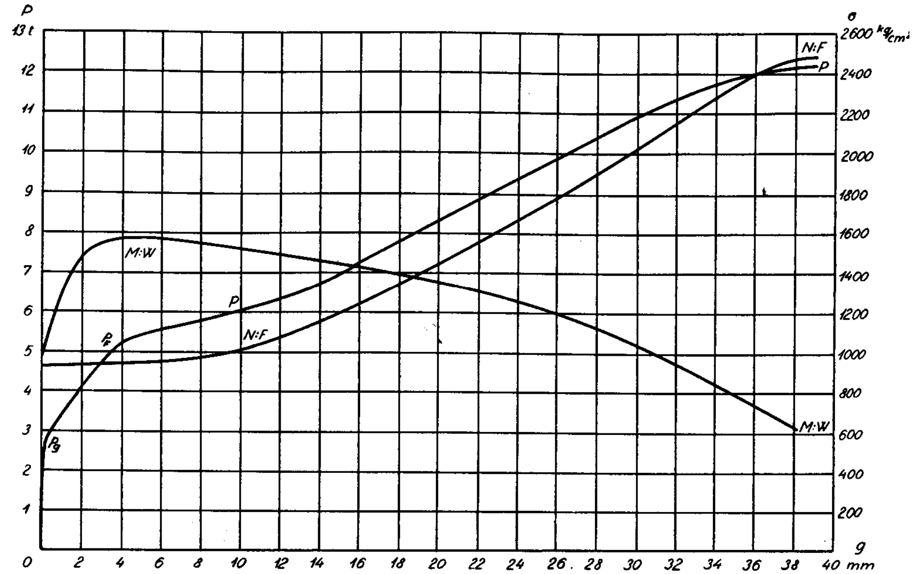
The derivation of the "rope effect" is shown in **Paper 35-7-4**. The rope effect is present in all failure modes.

Near failure the deformations become big resulting in additional tension in the fastener, which becomes inclined so that part of the load may be taken by a direct tension component parallel to the load. Both effects are proportional to the tension force and are taken into account in Eurocode 5 for failure modes 1c, 2 and 3. The effect is demonstrated in figure 5.

Load-deformation curves are shown in Figure 6. For a joint with a dowel (bolt without tension) the slip limit is $P = 0$. Between the slip limit and the yield load P_F the dowel is pressed against the wood and making it



a) Dowel



b) Prestressed bolt

Figure 6. Load-slip curves and bending and axial stresses for a dowel and a prestressed bolt. g is deformation (slip), P is the load, N:F is the axial stress in the dowel and M:W is the bending stress in the dowel. From [Johansen, 1941].

possible to take an axial force N resulting in friction between the dowel and the wood.

For joints with a (prestressed) bolt the slip is 0 until $P = P_F$ where P_F is the friction load corresponding to the prestress and the load-deformation curve is shifted upwards by the friction load, resulting in an increase in load-carrying capacity.

It is relatively simple to derive expressions for the load-carrying capacities for the described failure modes using only the equilibrium conditions. see Annex: Theoretical load-carrying capacity expressions.

Wood-to wood or wood-based panel-to-wood connections

Single shear joints: (2)

$$R_{v,k} = \min \begin{cases} f_{h,k,1} t_1 d \\ f_{h,2,k} t_2 d \\ \dfrac{f_{h,1,k} t_1 d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1+\dfrac{t_2}{t_1}+\left(\dfrac{t_2}{t_1}\right)^2\right]+\beta^3\left(\dfrac{t_2}{t_1}\right)^2} - \beta\left(1+\dfrac{t_2}{t_1}\right)\right]+T \\ 1{,}05\dfrac{f_{h,1,k} t_1 d}{2+\beta}\left[2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{y.k}}{f_{h,1,k} d\, t_1^2}-\beta\right]+T \\ 1{,}05\dfrac{f_{h,1,k} t_2 d}{1+2\beta}\left[2\beta^2(1+\beta)+\dfrac{4\beta(1+2\beta)M_{y.k}}{f_{h,1,k} d\, t_2^2}-\beta\right]+T \\ 1{,}15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,k} f_{h,1,k} d}\ +T \end{cases}$$

Double shear joints: (3)

$$R_{v,k} = \begin{cases} f_{h,k,1} t_1 d \\ 0{,}5 f_{h,2,k} t_2 d \\ 1{,}05\dfrac{f_{h,1,k} t_1 d}{2+\beta}\left[2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{y.k}}{f_{h,1,k} d\, t_1^2}-\beta\right]+T \\ 1{,}15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,k} f_{h,1,k} d}\ +T \end{cases}$$

with

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} \tag{4}$$

$$T = \frac{F_{ax,k}}{4} \tag{5}$$

where

$R_{v,k}$ the characteristic load-carrying capacity per shear plane per fastener
t_i the timber or wood-based panel thickness or penetration depth, with i either 1 or 2
$f_{h,k,i}$ the characteristic embedding strength in timber member i
d the fastener diameter
$M_{y,k}$ the characteristic fastener yield moment
β the ratio between the embedding strength of the members
T the contribution from the rope effect
$R_{ax,k}$ the characteristic axial withdrawal capacity of the fastener.

Correction factors

In the expressions (J.2.2) and (J.2.3), the first terms on the right hand side without the factors 1,05 and 1,15 are the load-carrying capacities according to the Johansen yield theory, whilst the second term $T = F_{ax} / 4$ is the contribution from the rope effect.

The factors are correction factors to compensate for the very simple way the design values are derived from the characteristic value, viz.

$$R_{y,d} = k_{\text{mod}} R_{y,k} / \gamma_M \tag{6}$$

whether the load-carrying capacity depends solely on the wood properties or partly also on the steel properties. In the latter case it would be more correct to introduce k_{mod} and the partial safety factors directly on the material parameters;

$$R_y \approx \sqrt{M_y f_y} \sim \sqrt{\frac{M_k}{\gamma_{M,steel}}\frac{k_{\text{mod},wood} f_{h,k}}{\gamma_{M,wood}}} = \frac{R_k}{\sqrt{\gamma_{M,steel}\gamma_{M,wood} / k_{\text{mod},wood}}} \tag{7}$$

Typically $\dfrac{\gamma_{M,wood}}{\gamma_{M,steel}}$ is about 1,2 and k_{mod} about 0,9 i.e.

$$R_{y,d} \approx 1{,}15\frac{k_{\mathrm{mod},wood}R_k}{\gamma_{M,wood}}$$

For the other failure modes with bending in the dowel, the effect is smaller and a factor of 1,05 has been chosen.

In **Paper 27-7-2** the corrections are (wrongly) explained as special system factors.

Limitation of rope effect

The contribution to the load-carrying capacity due to the rope effect should be limited to the following percentages of the Johansen part of the load carrying capacity:

Round smooth nails	15 %
Square smooth nails	25 %
Other (threaded) nails	50 %
Screws	100%
Bolts	25 %
Dowels	0 %

If $R_{ax,k}$ is not known, then the contribution from the rope effect should be taken as zero.

For single shear fasteners the characteristic withdrawal capacity $R_{ax,k}$ is taken as the lower of the capacities in the two members.

For the withdrawal capacity $R_{ax,k}$ of bolts the resistance provided by the washers may be taken into account

Steel-to-wood connections

The characteristic load-carrying capacity of a steel-to-timber connection depends on the thickness of the steel plates. Steel plates of thickness less than or equal to $0,5d$ are classified as thin plates and steel plates of thickness greater than or equal to d with the tolerance on hole diameters being less than $0{,}1d$ are classified as thick plates.

The difference between thick and thin plates is that for thick plates as outer members, it is assumed that the dowel can be restrained at the surface by a moment $M = M_y$. For thin plates $M = 0$.

In **Paper 28-7-3** it is shown that for many connector nails it may be assume that they act as fully restrained in steel plates down to a thickness of $0,5d$.

Thin steel plate in single shear:

$$R_{y,k} = \min\begin{cases} 0.4 f_{h,k} t_1 d \\ 1.15\sqrt{2M_{y,k} f_{h,k} d} + T \end{cases} \quad (8)$$

Thick steel plate in single shear:

$$R_{v,k} = \min\begin{cases} f_{h,k} t_1 d \\ f_{h,k} t_1 d\left[\sqrt{2+\dfrac{4M_{y,k}}{f_{h,k} d\, t_1^2}}-1\right] + T \\ 2.3\left[\sqrt{M_{y,k} f_{h,k} d} + 1\right] + T \end{cases} \quad (9)$$

Steel plate of any thickness as middle member in a double shear connection:

$$R_{v,k} = \min\begin{cases} f_{h,1,k} t_1 d \\ f_{h,1,k} t_1 d\left[\sqrt{2+\dfrac{4M_{y,k}}{f_{h,1,k} t_1^2 d}}-1\right] + T \\ 2.3\sqrt{M_{y,k} f_{h.1.k} d} + T \end{cases} \quad (10)$$

Thin steel plates as the outer members in double shear connection:

$$R_{v,k} = \min\begin{cases} 0,5 f_{h,2,k} t_2 d \\ 1,15\sqrt{2M_{y,k} f_{h,2,k} d} + T \end{cases} \quad (11)$$

Thick steel plates as outer members of a double shear connection:

$$R_{v,k} = \min\begin{cases} 0,5 f_{h,2,k} t_2 d \\ 2,3\sqrt{M_{y,k} f_{h,2,k} d} + T \end{cases} \quad (12)$$

$$T = \frac{F_{ax,k}}{4} \qquad (13)$$

where

$R_{y,k}$ characteristic load-carrying capacity per shear plane per fastener
$f_{h,k}$ characteristic embedding strength in the timber member
t_1 the smaller of the thickness of the timber side member or the penetration depth
t_2 thickness of the timber middle member
d fastener diameter
$M_{y,k}$ characteristic fastener yield moment
T rope effect contribution
$R_{ax,k}$ characteristic withdrawal capacity of the fastener.

Failure modes

The failure modes for steel to timber joints are illustrated in Figure 7.

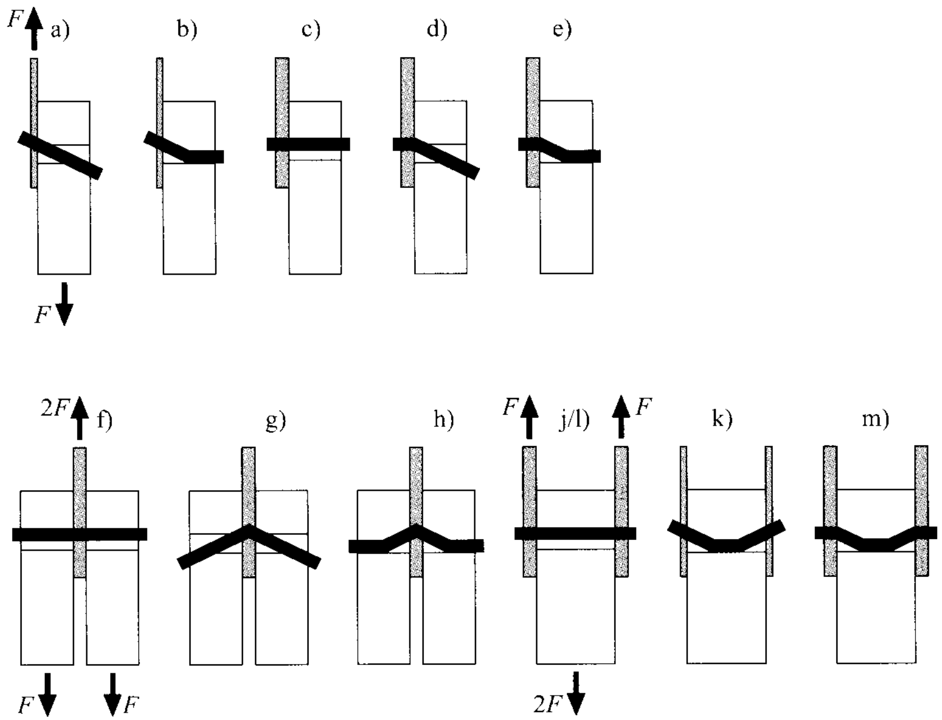


Figure 7. Failure modes for steel-to-timber joints.

Embedding strength

The determination of the embedding strength values in Eurocode 5 for softwoods and panel materials is described in **Paper 20-7-1.**

Other papers on embeddings strength are **Paper 25-7-2** and **Paper 41-7-5**

The embedding strength of fasteners in solid wood panels is treated in **Paper 39-7-5**

Yield moment

One of the important parameters in the expressions for the load-carrying capacity is the yield moment. For nails a standardised test method is described in EN 409[3].

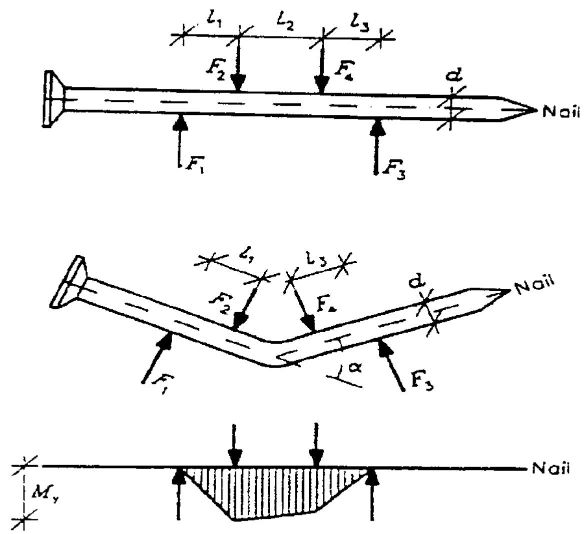


Figure 8. Nail loading, nail deformation and bending moments according to tests described in EN 409. Since F2 and F4 may differ the midsection may also be subjected to shear. There are no tensile force because the loads acts perpendicular to the fastener.

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3. EN 409 Timber structures – Test methods – Determination of the yield moment of dowel type fasteners

The principle is set out in Figure 8. The test methods aim at reducing the influence of shear, and the effect of the support forces becoming inclined. The yield moment is taken as the maximum moment for a rotation α less than 45º. For thin dowels the method works well, the moment becomes almost constant for increasing values of α **.**

For bolts K. W. Johansen used the elastic moment

$$M_y = \frac{\pi}{32} d^3 f_y \tag{14}$$

where

M_y yield moment [Nmm]
d diameter [mm]
f_y yield strength in tension [Nmm^2]

Also the moment corresponding to full plastic behaviour

$$M_y = \frac{d^3}{6} f_y \tag{15}$$

has been used.

However it was pointed out in **Paper 31-7-6** and **Paper 31-7-7** that neither of these moments or a moment found by EN 409 are relevant for large diameters: They disregard the influence of strain hardening and in practice a bending angle of 10-20º is more appropriate than 45º. Based on tests a moment of

$$M_{y,k} = 0,3 f_{u,k} d^{2,6} \tag{16}$$

was introduced in Eurocode 5.

Later equation (16) has also entered Eurocode 5 for the yield capacity of dowels with thin diameters like nails and staples. Whether this is correct is discussed **Paper 38-7-5**. The conclusion is that it is on the safe side to use this equation in general for all diameters. But for small values of d (like stables) the bending capacity according to becomes larger than the fully plastic value.

Load distribution

The load carrying capacity R_n of a connection with n fasteners in line in the load direction, does generally not equal the load carrying capacity of a single fastener multiplied by n . Therefore, an effective number of fasteners $n_{ef} < n$ has been introduced. R_n is calculated as:

$$R_n = n_{ef} R_{single} \tag{17}$$

Even assuming ideal conditions - identical load-slip curves of single fasteners - the distribution of the load in multiple-fastener joints is non-uniform when the fasteners are aligned parallel to the direction of loading, because of the different elongations of the connected members. For example, consider Figure 9: Between the first and second nail member 1 is loaded by force F minus fastener load 1 while member 2 resists only fastener load 1. Assuming the same extensional stiffness for both members, the elongation of member 1 between the first and second nail will be greater than the corresponding elongation of member 2. These different elongations must be compensated for by different displacements of the first and second nail. Different displacements mean - at least as long as the yield load is not yet reached – different fastener loads. This effect is also found in e.g. riveted steel structures where it is called the Volkersen effect.

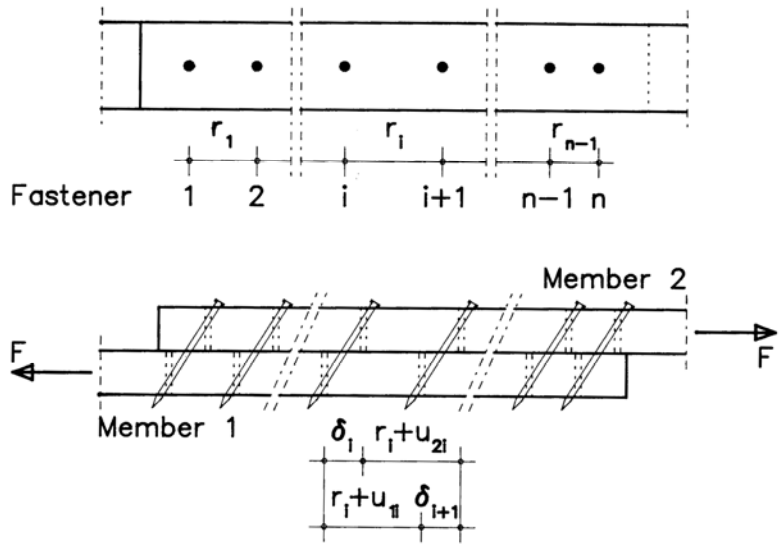


Figure 9. The influence of member elongations. From ***Paper 23-7-2****.*

If the load is increased over a proportional limit, the most highly stressed fasteners at the ends of the joint begin to deform plastically. Moreover, the embedment strength in the contact areas between these connectors and the wood is reached, and redistribution of load from the fasteners at the ends to those in the centre of the joint will result. After each fastener has reached its yield load, the differences in fastener loads become minimal and the joint reaches its yield load.

Since nailed joints are very ductile, load distribution in nailed joints should not affect the load-carrying capacity and this is confirmed by the tests described in **Paper 23-7-2** that concludes: The maximum load of a multiple-nailed joint can be estimated as the sum of those for individual nails, provided joint failure is by nail yielding. Irrespective of this conclusion an effective number $n_{ef} < n$ has without any argument been introduced in Eurocode 5 for nails.

For other fasteners, test results of several researchers indicate, that the ultimate load per fastener decreases, sometimes considerably with increasing number of fasteners arranged parallel to load. This suggests that the failure mode in many connections may not be attaining the joint's yield load. Instead, joint load capacity may be constrained by preliminary wood splitting. Consequently, the potential load capacity of the connection is not realised because load-slip curves of single fasteners break off and ideal redistribution of load is prevented. Oversized and misaligned bolt holes or split ring grooves tend to make the situation even worse: by causing differences in initial slip of single fasteners which makes the load distribution very uneven. This may lead to some single fasteners reaching their maximum load while other fasteners just begin to carry load because of their greater initial slip. In case of long-term or repeated loading, creep deformations and residual plastic deformations after previous higher loading also affect load distribution.

Simplified expressions

The complete set of equations may look a little complicated and many proposals for simplification by omitting some equations or combining two or more. Examples are given in e.g. **7-100-1**, **31-7-7, 31-7-8, 37-7-3** and **40-7-4.** CIB-W18 has, however maintained that with the spread of computers there is no need for such simplified methods. In reality it is easier and safer to use the complete set of equations. An argument is also that none of the simplified methods can cope with the rope effect.

Annex: Theoretical load-carrying capacity expressions

In the following expressions are derived for single-shear and symmetrical double-shear joints. The possible failure modes are shown in Figure A.1.

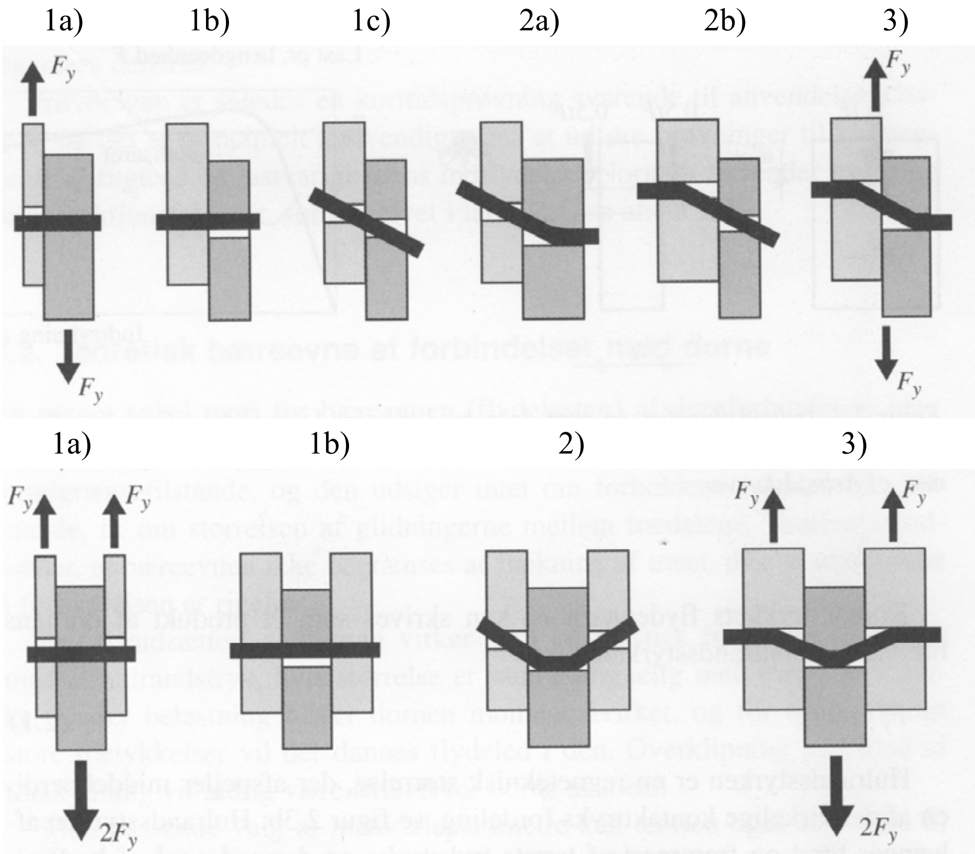


Figure A.1. Possible failure(yield) modes for single-shear and symmetrical double-shear joints.

In the single shear joint, the dowel will either remain straight (failure modes 1) or bend in one or two yield hinges (failure modes 2 or 3 respectively). For the double shear joint failure modes 1 correspond to a movement of the dowel either in the side members or the middle member. In failure mode 2 two yield hinges occur in the middle member while the dowel remains straight in the side members. In failure mode 3 four yield hinges are formed within the dowel, two in the middle member and one in each outer part.

In each member there are the following possibilities:

- the dowel makes a translation without rotation
- the dowel remains straight but rotates
- the dowel bends at a yield hinge.

It is found convenient initially to analyse the situation in one member and then combine the results. The first case – the translation – is so simple that no further mentioning is needed. The other two cases, where a dowel is loaded with a force F_y, resulting in "yielding" in the wood and maybe also in the dowel are shown in Figure A.2.

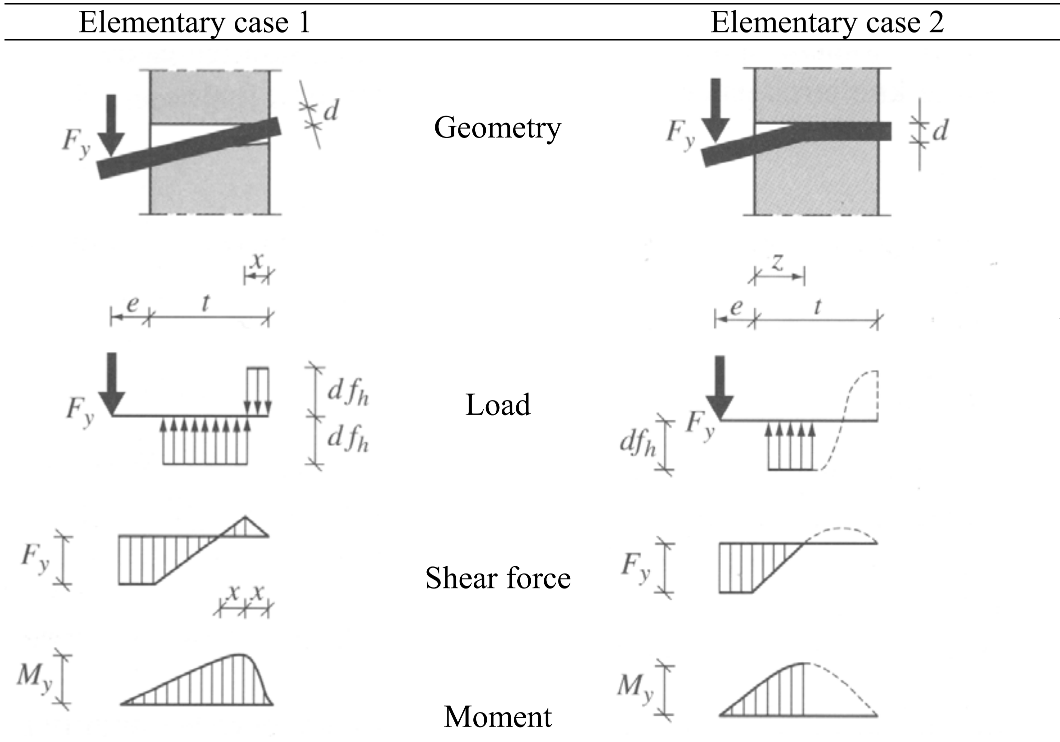


Figure A.2. Dowel loaded by the force F_y in the distance e from the surface. To the left Elementary case 1 (without yield hinge), To the right Elementary case 2 (with one yield hinge).

Elementary case 1

The dowel yield moment is greater than the bending moment. The dowel remains straight and rotates around a point with the distance x from the right surface of the member, see Figure A.2left. By projection and moment around the force

$$F_y = (t-2x)df_h \tag{A.1}$$

$$df_h\left[x\left(t-\frac{x}{2}+e\right)-(t-x)\left(\frac{t-x}{2}+e\right)\right]=0 \tag{A.2}$$

from which

$$x = t+e-\frac{1}{2}\sqrt{(t+2e)^2+t^2} \tag{A.3}$$

$$F_y = \left(\sqrt{(t+2e)^2+t^2}-(t+2e)\right)df_h \tag{A.4}$$

The maximum moments is found where the shear force is zero. Moment equilibrium about the forces on one side of this point renders

$$M_{\max} = x^2 df_h \tag{A.5}$$

Elementary case 2

With the ideal assumptions the dowel will remain straight until a yield hinge is formed at a distance z from the surface, see Figure A.2right. Since the moment M_y is a maximum moment, the shear force is equal to 0 in this point. Vertical equilibrium and moment equilibrium about the yield hinge give

$$F_y - zdf_h = 0 \tag{A.6}$$

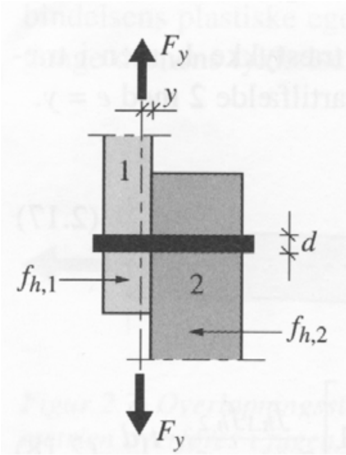
$$F_y(e+z) - zdf_h\frac{z}{2} - M_y = 0 \tag{A.7}$$

From these equations

$$z = \sqrt{e^2+\frac{2M_y}{df_h}}-e \tag{A.8}$$

$$F_y = \left(\sqrt{e^2+\frac{2M_y}{df_h}}-e\right)df_h \tag{A.9}$$

Single shear joint
For the single-shear joint shown in Figure A.3 the load-carrying capacity can now be determined for the possible failure modes shown in Figure A.1



Figur A.3. Single shear joint. It is assumed that the yield hinge i placed in member 1 at a distance y from the joint (positive as shown). Since the shear force is 0 the load is acting in this line.

The member thicknesses are t_1 and t_2 and the embedding strengths $f_{h,1}$ and $f_{h,2}$ with

$$\beta = \frac{f_{h,2}}{f_{h,1}} \tag{A.10}$$

For failure mode 1c the load-carrying capacity yielding takes place simultaneously in the two members , where the situation corresponds to Elementary case 1, i.e. equation (A.4) apply. For member 1 $e = -y$.

$$F_y = \left[\sqrt{(t_1 - 2y) + t_1^2} - (t_1 - 2y)\right] d f_{h,1} \tag{A.11}$$

For member 2 $e = y$.

$$F_y = \left[\sqrt{(t_2 + 2y) + t_2^2} - (t_2 + 2y)\right] d f_{h,2} \tag{A.12}$$

By elimination of y:

$$F_y = \frac{f_{h,1} t_1 d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right] + \beta^3\left(\frac{t_2}{t_1}\right)^2} - \beta\left(1 + \frac{t_2}{t_1}\right)\right] \tag{A.13}$$

For failure mode 2a (A.11) is still valid for member 1; but in member 2 (A.9) apply with $e = y$:

$$F_y = \left(\sqrt{y^2 + \frac{2M_y}{d f_{h,2}}} - y\right) d f_{h,2} \tag{A.14}$$

By elimination of y:

$$F_y = \frac{f_{h,1} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta) M_y}{f_{h,1} d t_1^2}} - \beta\right] \tag{A.15}$$

For failure mode 2b the yield hinge is placed in member 2 and correspondingly:

$$F_y = \frac{f_{h,1} t_2 d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta) + \frac{4\beta(1+2\beta) M_y}{f_{h,1} d t_2^2}} - \beta\right] \tag{A.16}$$

For failure mode 3 there is a yield hinge in both members. For member 2, (A.14) still apply, while (A.9) apply for member 1 with $e = -y$.

$$F_y = \left(\sqrt{y^2 + \frac{2M_y}{d f_{h,1}}} + y\right) d f_{h,1} \tag{A.17}$$

By elimination of y:

$$F_y = 2\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2 M_y f_{h,1} d} \tag{A.18}$$

The load-carrying capacity is found as the minimum value found by these expressions.

Symmetrical double shear joints

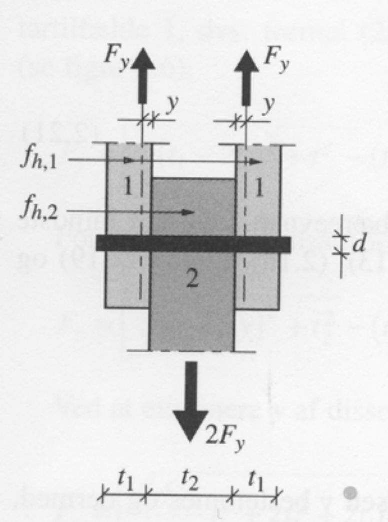


Figure A.4. Symmetrical double shear joint.

The thickness of the outer members is t_1 and of the middle member t_2. The embedding strengths are $f_{h,1}$ and $f_{h,2}$ with $\beta = f_{h,2}/f_{h,1}$.

As shown in Figure A.1 there are 4 failure modes. For failure mode1a:

$$F_y = t_1 d f_{h,1} \tag{A.19}$$

and for failure mode 1b:

$$F_y = 0,5 t_2 d f_{h,2} \tag{A.20}$$

For the failure modes 2 and 3 F_y is the same as for the single-shear joints since F_y is per shear.

12-7-2 H J Larsen
Design of bolted joints

The paper describes for the first time in CIB/W18 the Johansen theory that forms the basis for the present design rules for laterally loaded bolted joints. The results are compared with the design rules in Germany. The paper also contains approximation reducing the calculations considerably. With the spread of computers the opinion of CIB/W18 is that there is no need for such methods.

19-7-6 L R J Whale, I Smith
The derivation of design clauses for nailed and bolted joints in Eurocode 5

Based on contracts from both the Commission of the European Communities and the UK Department of the Environment, TRADA has study the properties of mechanical timber joints under short-term lateral loading. Emphasis was placed upon deriving reliable embedment characteristics for solid timbers and sheet materials, and a broad range of timber species, and wood-based sheet materials were included in the test programme.

Seven timbers were selected for embedment testing; UK grown Sitka spruce and Scots pine, European redwood and whitewood, spruce-pine-fir, keruing and greenheart. These were loaded parallel or perpendicular to the grain in either a compressive or a tensile mode, with nails ranging from 2.65 to 6 mm in diameter and bolts from 8 mm to 20 mm in diameter.

Sheet material specimens of Finnish birch plywood, Finnish spruce plywood, French pine plywood, Canadian Douglas fir plywood, 4.8 mm and 8 mm tempered hardboard were also tested, with nails ranging from 2.65 mm to 4 mm in diameter. The thickness of solid wood specimens was limited to twice the connector diameter, whilst actual board thicknesses were used in the case of the sheet material specimens. Toleranced holes, approximately 1.5 mm oversize, were provided in all bolted specimens. Nailed hardwood specimens were prebored to 80 % of the nail diameter.

20-7-1 L R J Whale, I Smith, H J Larsen
Design of nailed and bolted joints. Proposals for the revision of existing formulae in draft Eurocode 5 and the CIB Code

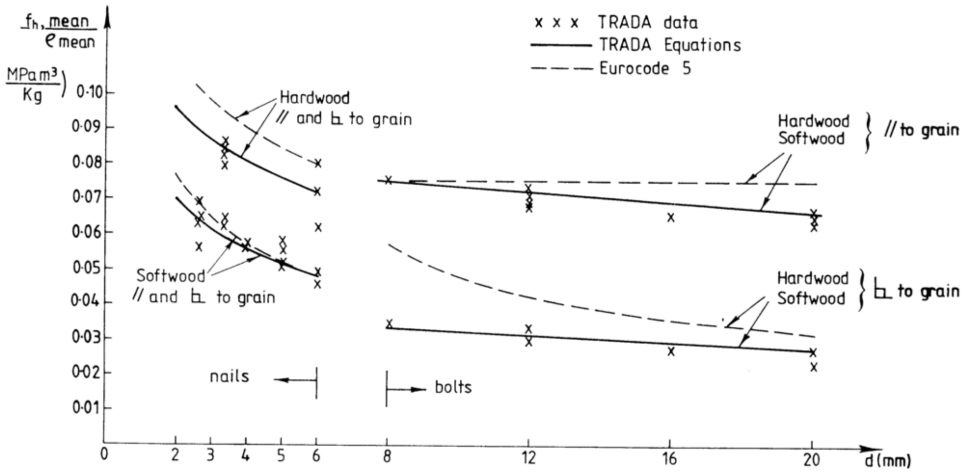
Based on the results of a comprehensive investigation by TRADA on short-term properties of nailed and bolted joints (**Paper 19-7-1**) and the application of that data in deriving design equations for Eurocode 5 (**Paper 17-7-6**) and in accordance with discussions on these papers meeting, slight modifications are made to the design approaches which had originally been put forward. Results from over 400 joint tests, carried out as part of the TRADA investigation, are also presented as a testament to the accuracy of the design equations which are proposed.

Furthermore, the results of tests on nailed plywood and tempered hardboard specimens, which through lack of time had been precluded from the previous analysis, are presented. These are used in an assessment of the design approaches to board material-to-timber joints which currently exist in the (CIB code, 1983) and the draft (Eurocode 5, 1986). As a result, modifications are proposed which extend the range of design possibilities for these types of joint.

The basic wood strength parameter determining the lateral load carrying capacity of joints with nails, bolts, screws and other dowel-type fasteners is the embedding strength f_h.

The results of TRADA's recent research on these types of joint, in (Smith, 1983) and (Whale and Smith, **Paper 19-7-6**) are summarised in Figure 1, where also the values assumed in the draft (Eurocode 5, 1986) are shown.

Figure 1 f_h mean/ρ mean as a function of diameter d for nails or bolts in solid timber



In the TRADA work, embedding strengths were found for plain round nails with a diameter, *d*, between 2.65 and 6.0 mm loaded parallel and perpendicular to the grain in softwoods (mean density, R an about 400-500 kg and in dense hardwoods (~mean about 700-1000 kg/m^3). In softwoods no preboring was used. In hardwoods holes were prebored to 80 % of the nail diameter.

The embedding strength was also found for bolts with a diameter between 8 and 20 mm loaded parallel and perpendicular to the grain. The holes were approximately 1.5 mm oversized. This followed earlier work by (Smith, 1983) in which embedding strengths were determined in European whitewood for plain steel dowels in close-fitting holes.

For bolts, no difference was found between the softwood and dense hardwood specimens in respect of their embedding strength density trends.

It is conceivable therefore, that the difference between nails in softwood and hardwoods is attributable to the effect of preboring, rather than to inherent differences in the behaviour of the wood species as had previously been assumed in **Paper 19-7**-1. This approach is certainly more appealing from the codification viewpoint, and is supported by the fact that the differences found in Figure 1 compare well with those reported by (Wilkinson, 1973) for the effects of lead holes in softwoods.

The curve in Figure 1 for nails in softwood corresponds to (see **Paper 19-7-6**):

$$f_{h,mean} = 0,09 \rho_{mean} d^{-0,36}$$

where ρ_{mean} is defined by volume and mass at 20 °C/65 % RH.

In the absence of contradictory data, this relationship will be assumed for all nails without preboring irrespective of timber species.

In Figure 1 the ratio for a given nail diameter between f_h for hardwood and for softwood varies between 1.40 for d = 2,65 mm and 1.48 for d = 6.0 mm. Hereinafter a constant value of 1.44 will be assumed.

The curve in Figure 1 for bolts loaded parallel to the grain corresponds to (see **Paper 19-7-6**):

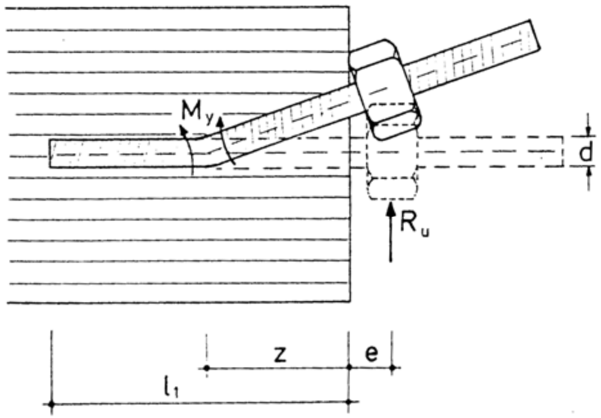
$$f_{h,mean} = 0,082(1-0,01d)\rho_{mean}$$

For the load acting at an angle α to the grain the following will be assumed:

$$f_{h,mean,\alpha} = \frac{f_{m,mean,0}}{2.3\sin^2\alpha + \cos^2\alpha}$$

22-7-1 J Ehlbeck, M Gerold
End grain connections with laterally loaded steel bolts. A proposal for design rules in the CIB code

Introduction
This paper presents a calculation model for steel bolts driven in end-grain of glulam or solid timber under lateral loading.



Test results and conclusions
Tests from Riberholt and Möhler/Hemmer with steel bolt diameters of 16 mm yielded mean ultimate load-carrying capacities of 9.8 kN and 10.3 kN, respectively. The wood density was in both cases approximately 450 kg/m^3. The yield moments of the steel bolts were specified with 20 kNcm and 21.8 kNcm, respectively. The eccentricities were 10 mm.

Using the proposed equation with $f_{h,90/0}$ = 9.5 N/mm^2, the ultimate load-carrying capacities amount to 9.8 kN for Riberholt's tests and 10.4 kN for Möhler/Henimer's test, which is in line with the test results.

These investigations indicate, that a determination of the ultimate load-carrying capacity of steel bolts in end-grain under lateral loading can sufficiently accurate be calculated by using the formula given in the paper.

25-7-2 J Ehlbeck, H Werner
Softwood and hardwood embedding strength for dowel-type fasteners, Background of the formulae in Eurocode 5

Introduction
The embedding strength of timber and wood-based materials as well as the yield moment of the fasteners are (besides the joint's geometry) governing properties for determining the load-carrying capacities of joints with dowel-type fasteners. The embedding strength depends on the type of fastener, the joint configuration (such as member thicknesses, end and edge distances as well as spacing of the fasteners), the manufacturing of the joint (e.g. predrilled holes), and the wood species or the quality of the wood-based materials. Thus, the embedding strength is not a special material property, but a system property.

Many research works have been done to describe this property and to collect sufficient test data. The test methods used differed, however, in many cases significantly so that the results can not be compared without any reservation. It was therefore an important task of the responsible European standardization committee to produce a test standard EN 383 "Timber Structures - Test Methods - Determination of Embedding Strength and Foundation Values for Dowel-type Fasteners".

The embedding strength of some hardwoods was tested according to this European, harmonized test method. The tests were made under different load-grain angles using bolts and dowels. Additional tests were performed with European whitewood (picea abies) under 90° load-grain angle. When evaluating all test data, the results published by [Whale and Smith, 1986 a and b] were also used because these tests under loading parallel to grain were in line with the main principles of EN 383.

Conclusions
The variance of the embedding strength of hardwoods (coefficient of variation 15%) is larger than the variance of the hardwood densities (coefficient of variation 8%). This taking into account, the characteristic embedding strength, $f_{h,k}$, can be derived assuming a Gaussian distribution:

Hardwood, parallel to grain

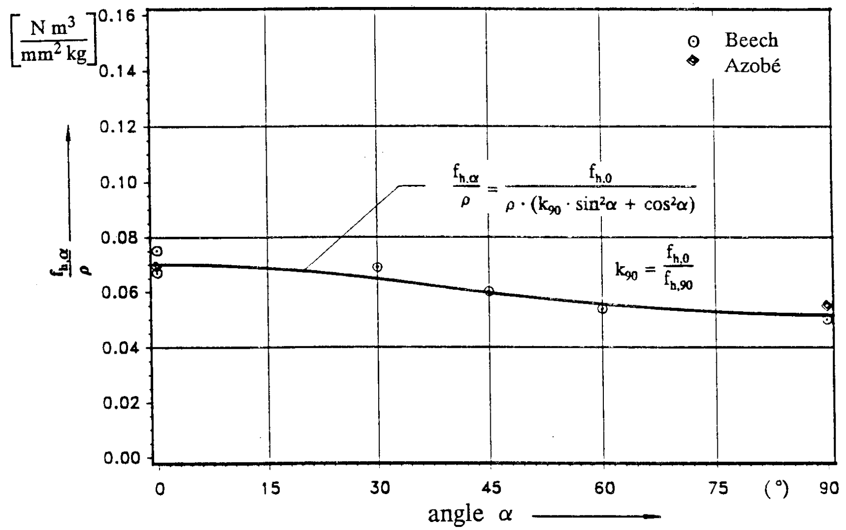
$$f_{h,0,k} = 0,09 \cdot \left(1 - 0,01d\right)\rho_k$$

Hardwood, perpendicular to grain

$$f_{h,0,k} = 0,09 \cdot \left(1 - 0,016d\right)\rho_k$$

$f_{h,0}$ embedding strength parallel to grain, N/mm^2
$f_{h,90}$ embedding strength perpendicular to grain, N/mm^2
ρ_k characteristic density, kg/m^3
d fastener diameter, mm

Similar to the behaviour of wood under compressive stresses the embedding strength decreases with increasing load-grain angle, α . This effect is also depending on the fastener diameter, i.e. the decrease of embedding strength is more distinct with large than with small diameters. As a good fitting approximation can be used the Hankinson formula (see the figure):



Ratio $f_{h,\alpha}$ / ρ_k over load-grain angle α. Hardwood, predrilled holes, smooth round fasteners, diameter d = 30 mm.

For nailed joints without predrilled holes the embedding strengths are approximately independent of the load-grain angle because incipient cracks due to splitting tendency are reducing the embedding strength parallel to grain. This is generally assumed in all design codes, where load-carrying capacities or allowable loads for nailed joints are independent of the load-grain angle. In this case, $k_{90} = 1$.

Proposals

Based on these investigations the following proposals are given to be used in the Eurocode 5 for characteristic embedding strengths for

bolted and dowelled joints:

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90}\sin^2\alpha + \cos^2\alpha}$$

$$f_{h,0,k} = 0,082 \cdot (1 - 0,01d)\,\rho_k$$

$$k_{90} = 1,35 + 0,015\,d \quad \text{(softwood)}$$

$$k_{90} = 0,90 + 0,015\,d \quad \text{(hardwood)}$$

nailed joints (d < 8 mm)

$$f_{h,k} = 0,082\,d^{-0,3}\rho_k \qquad \text{(not predrilled holes)}$$

$$f_{h,k} = 0,082 \cdot (1 - 0,01d)\,\rho_k \qquad \text{(predrilled holes, as for bolts)}$$

It can be seen from the tests that the embedding strengths of hardwoods are approximately 10% higher than those of softwoods with same density. This could be made allowance for in design codes if further investigations verify this observation.

29-7-8 L Davenne, L Daudeville, M Yasumura
Failure of bolted joints loaded parallel to the grain. Experiment and simulation

Abstract

This paper deals with the analysis of failure of joints with dowel-type fasteners in glued laminated timber under static loading. The single bolt loads the wood parallel to the grain. The joint failure is due to a cracking along the grain direction.

An experimental program was carried out on joints for different structural parameters and bolt diameters.

Fracture is analysed by use of linear elastic fracture mechanics concepts. Possible pure or mixed modes of fracture are investigated. The crack propagation condition is assumed to be based on a comparison of the

energy release rate with a critical value. An analysis of elastic stresses is carried out for the prediction of the onset of cracking.

The critical energy release rate value was obtained from experimental results of fracture tests under three point bending. The comparison between experimental and numerical results for the simulation of fracture in joints shows that the linear elastic fracture mechanics provides a good approximation of load-bearing capacity of bolted joints and may help improve design codes.

Conclusion

LEFM is a simplified approach consisting in the comparison of the energy release rate with a critical value. This method has been applied for the determination of the load carrying capacity of a mechanical joint with a single bolt. The loading was parallel to the grain. The fracture mode that leads to failure is the mode I of opening. The critical energy release rate was chosen equal to the fracture energy value issued from bending tests. Comparisons of the load carrying capacity obtained experimentally and numerically with the Finite Element method is encouraging by considering the coarse assumptions of the simplified approach.

Adjust this simplified approach for calculation rules might provide an improvement for the determination of the ultimate load in the design of bolted or nailed joints.

30-7-6 A Mischler
Influence of ductility on the load-carrying capacity of joints with dowel-type fasteners

The load-carrying capacity of joints with laterally loaded dowel-type fasteners is, in Eurocode 5, based on a plastic design method, which was developed by Johansen (1949). In this model, it is assumed that both the dowel and the timber behave like rigid-plastic materials. Brittle failure modes such as splitting or plug shear are not taken into account by this design method. Plastic analysis methods are only valid to describe the load-carrying capacity on condition that no premature brittle failure will occur.

The deformations in the joint before failure have to be large enough, so that the dowel can reach the yield moment. In all other cases the failure-load will be less than the calculated load-carrying capacity. In Eurocode 5 no indication is given about ductility conditions. It is assumed that minimum requirements for spacing, end- and edge distances prevent brittle failure modes. However, tests on multiple fastener joints show sometimes brittle failure in timber splitting even if minimum requirements are satisfied.

In this paper, the importance of ductile failure is shown. It is also discussed how the joint has to be designed to achieve these ductile failure modes.

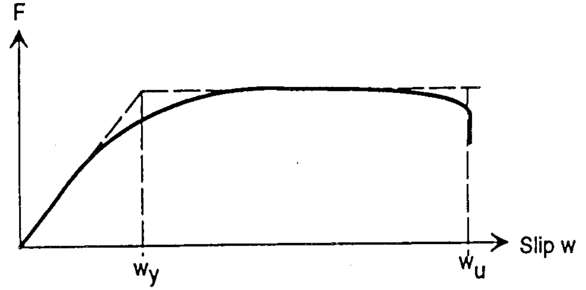
In timber construction, joints constitute the weak points of the construction. Therefore, joints should always be designed of such a kind that their failure occurs only after great deformations (i.e. ductile).

The ductility of a joint exercises a decisive influence

– *on the general behaviour of the structure*
  Especially when loaded in tension, wood behaves extremely brittle. In the joint range, the cross-sectional area is reduced by holes for the dowels and by slits for steel plates, according to the respective design. These notches create stress concentrations which cannot be reduced by plastic deformations because of the brittle failure of wood in tension. Therefore, ductile behaviour of a timber structure is only obtainable if the joints develop large plastic deformations before failure. Even if the load-carrying capacity of a single joint is reduced by such a ductile design, the collapse load of the whole structure increases, because a plastic redistribution of the internal forces becomes possible.
– *on the bearing behaviour of the joint*
  When the dowels are stressed up to their plastic moment, the maximum load-carrying capacity of the dowel joint is reached. For this, large elastic and plastic bending deformations of the dowels are necessary. Thus, the whole joint has to allow enough big deformations before failure. If the joint fails before achieving these deformations, the plastic load-carrying capacity cannot be reached. As an example, this would be the case when splitting failure in the wood occurs.
– *on the load-carrying capacity of a group of fasteners*
  A mechanical joint in timber construction normally consists of more than one fastener. The load-carrying capacity of the hole joint depends on how far the sum of the individual fasteners capacities have to be reduced. There is an unequal distribution of load among the fasteners if the fasteners are placed in a line parallel to the direction of loading. A certain balancing of these unequal forces is possible in case of large

plastic deformations before failure of the joint. As for wood, the failure in embedment is not ductile enough to allow the necessary deformations, the balancing of the forces has therefore to occur by plastic deformations of the dowels. Even if the design of the joint is ductile (slender dowels) the load-carrying capacity of the joint is smaller than the sum of the individual fastener capacities.

The demand for a high ductility of the joint is not contradictory to the demand for high stiffness: The slip modulus for the serviceability limit state is determined between 10 and 40 % of the load-carrying capacity, i.e., in the linear range of deformation. Given a certain ultimate strength deformation w_u the ductility of the joint becomes bigger with smaller elastic deformations w_y (see the figure).



measure of ductility = $\frac{w_u}{w_y}$

Conclusions

The Johansen-theory was developed and validated for single fastener connections. The equations give in Eurocode 5are based on this theory but they are used without any reduction for multiple-fastener joints up to 6 dowels in line.

It is shown that the model of Johansen is only valid for ductile failure modes. The investigations made by Jorissen [1996] prove that a multiple fastener joint shows a ductile failure only under condition that mode III according to Johansen is reached. Therefore doweled joints have to be designed in such a way that they develop large plastic deformations before failure. This is only possible if the dowel slenderness *l/d* is higher than 4.

Even if the multiple fastener joint fails in a ductile way, the load-carrying capacity is smaller than the sum of the individual fastener capacities.

31-7-2 A Mischler
Design of joints with laterally loaded dowels

Introduction

The load-carrying capacity of joints with dowel type fasteners is in most codes based on the load-carrying capacity of the single fastener. Therefore it has been the aim of many researchers to determine the characteristic load-carrying capacities of single fasteners.
The following comparison between a multiple fastener connection and a single fastener may point out, that many factors which can strongly affect the load-carrying behavior of a multiple fastener joint are neglected in tests on single fasteners.

Principles for the design of connections

Timber connections have to be designed according to the following three main requirements
– ultimate resistance
– stiffness
– ductile failure.

In timber construction joints constitute the weakest points of the structure. Therefore joints should be designed of such a kind, that they develop plastic deformations before failure (i.e. a ductile failure mode).

The ductility of the connection has an important influence
– *on the load-carrying behaviour of the whole structure:*
  Even if the load-carrying capacity of the single joint is reduced by such a ductile design, the ultimate strength of the whole structure may increase, because a plastic redistribution of the internal forces becomes possible.
– *on the load-carrying capacity of the joint:*
  The load-carrying capacity according to the European Yield Model can only be reached when no premature brittle failure in the timber occurs. In multiple fastener connections even small fabrication tolerances lead to an uneven load distribution among the fasteners. A certain balancing of these unequal forces is possible by plastic deformations in the joint.

For joints with multiple dowel type fasteners, ductile behaviour is typically only possible if the failure occurs after significant plastic deformation of the steel dowels. This failure mode (Type III, according to the European Yield Model) can be reached when fasteners of effective slenderness ratio bigger than the limit slenderness ratio are used.

To reduce the risk of timber splitting the dowel strength and the spacing among the fasteners has to be adapted to the timber properties. The use of high strength dowels creates higher stresses in the timber. Therefore the spacing among the dowels has to be adapted also to the strength of the dowels. High strength dowels should generally not be used in timber of low tensile strength perpendicular to grain.

If the resistance of the fasteners is higher than the resistance of the timber in the net section, a brittle tensile failure occurs in the timber. Therefore an optimal adjustment of the number of fasteners on the tensile resistance of the timber in the net section is also necessary to avoid brittle failure.

To prevent a tensile failure the 5th percentile of timber strength should be bigger than the 95th percentile of the strength of the connection.

Verifications in joint design

The failure of a timber joint is often caused by the failure of the timber parts. Therefore two separate verifications have to be done in the design of timber joints:

– Resistance of the timber parts in the connection
– Resistance of the fasteners.

The verification of the timber should consider:

– the general stress distribution
– the local stress-peaks due to the load application by the fasteners
– the reduced cross-sectional timber area in the joint.

The load application by the fasteners creates additional shear and tension perpendicular to grain stresses in the joint. Two failures in plug shear are possible:

– shear failure of one row of dowels
– shear failure of a group of rows

To avoid the shear failure in one row the dowel strength and the spacing of the dowels has to be adapted to the shear strength of the timber.

A failure of a group of rows is possible by inadequate placing of the dowels. By applying a concentrated force parallel to grain only little distribution perpendicular to the grain direction is possible, due to the non-isotropic properties of timber. Therefore a reduced effective width has to be considered if the fasteners are not well distributed over the whole width of the timber.

31-7-6 A Jorissen, H J Blass
The fastener yield strength in bending

Introduction

The strength of single fastener connections with dowel type fasteners is, in Europe, determined according to the Yield Model, first presented by K.W. Johansen (1949). The fastener's yield capacity and the embedment strength are both governing material properties.

The fastener's yield capacity in bending is often determined according to prEN 409 (1994). Although the method, described in this standard, is meant for nails with a diameter < 8 mm, it is also used for other dowel type fasteners, e.g. bolts. It is discussed whether using this method for diameters > 8 mm are recommendable or not.

Summary and conclusion

In this paper the determination of the fasteners yield strength in bending for large diameter dowel type fasteners is discussed. ENV 1995-1-1 requires a tension test. A bending test for small diameters, (nails) is described in 409, which is also frequently used for large diameters.

Since dowel type fasteners are loaded in bending, a bending test prevails for large diameter dowels also. However, since the bending angle required in EN 409 (= 45º) is never reached for large diameters, this required angle should be reduced (e.g. 10°).

The factor 0.8 in the equations given in Eurocode 5 seems to reflect bending angles reached at failure load reasonable well.

32-7-1 M Mohammad, J H P Quenneville
Behaviour of wood-steel-wood bolted glulam connections

Abstract
This paper details verifications tests carried out at the Royal Military College of Canada (RMC) on wood-steel-wood bolted glulam connections. Twelve groups of specimens were tested. Specimens configurations were selected in such a way to include fundamental cases. Comparisons between experimental results and predictions from proposed equations for wood-steel-wood and wood-steel bolted connections are given. Proposed design equations were found to provide better predictions of the ultimate loads than current design procedure especially for bearing. However, row shear-out predictions seem to over-estimate the shear strength. Adjustment may need to be made to the proposed equations for row shear-out by introducing the effective thickness concept instead of using the full member thickness. Better predictions for row shear-out were achieved using the effective thickness instead of the full thickness. The research program is described in this paper along with results and proposed design equations for wood-steel-wood bolted connections.

Conclusions
Based on the validation tests of the proposed design equations for wood-steel-wood bolted connections, it can be concluded that:
5. Current Canadian design code (086.1-94) leads to over-designed wood-steel-wood bolted glulam connections, especially with multiple bolts, where it under-estimates the failure loads.
6. Proposed design equations for wood-steel-wood bolted connections provide better predictions of the ultimate loads than current design procedure, especially for bearing.
7. Better predictions for row shear-out can be achieved if the effective thickness principle is used.

32-7-2 M Ballerini
A new set of experimental tests on beams loaded perpendicular-to-grain by dowel-type joints

Abstract
The results of an experimental programme on the splitting strength of beams loaded perpendicular-to-the-grain by dowel-type joints are reported. The ultimate loads of beams failed by splitting are analysed and the types of failure are highlighted. Finally, the effectiveness of available prediction formulas from literature are compared and discussed with regard to this set of experimental data.

Introduction
Timber elements loaded over their depth by perpendicular-to-grain connections should be avoided in timber engineering. This way of loading indeed, is responsible for local perpendicular-to-grain tensile stresses, that may lead the beams to failure for splitting at load levels considerably lower than the bearing capacity of connections or of the bending strength of timber elements. Nevertheless, since in some designs this kind of loading may be fixed, the problem of the evaluation of the splitting strength of beams is still actual.

The Eurocode 5 takes this problem into consideration by means of a very simple formula which doesn't take into account the effect of the joint's geometry and assumes a linear relationship between the splitting strength and the distance of the furthest row of fasteners from the loaded edge of the beam.

Some researcher think this formula unsatisfactory.

Ehlbeck, Görlacher and Werner indeed, on the basis of the results of previous experimental researches, have developed a different formula. The formula is based on both theoretical considerations (with the support of linear elastic numerical analyses) and on the best fitting of the experimental results. Moreover, TACM van der Put, on the basis of an energetic approach in the framework of the Linear Elastic Fracture Mechanics, has developed a third prediction formula.

If the above mentioned experimental results are compared with available prediction formulas, it is possible to notice that the best fit is supplied by the formula of Ehlbeck, Görlacher and Werner, but also the one of TACM van der Put (which doesn't take into account the joint's geometry) gives reasonable results.
However, if the attention is focused on connections with few fasteners it is possible to detect how the first one is characterised by quite large scattering while the second one overestimates the splitting strengths.
Since the joint configuration affects considerably the strength of the beams, a new experimental programme on beams loaded by connections made with only one or two dowels (not much investigated in previous re-

searches) has been carried out at the University of Trento. The main intent of the programme is to provide a set of tests able to exploit the strength of beams loaded by a single connector and to evaluate, if possible, the influence on this strength of a second connector. Another characteristic of this set of tests is the slenderness of the beams which is higher than those of previous researches.

In the following the experimental programme and the test set-up will be described in detail. The experimental results will be presented, the types of failure highlighted and the influence on the strength of the main parameter exploited. Finally the effectiveness of available prediction formulas will be compared and discussed with regard to this set of experimental data.

Conclusions

A new set of experimental results on the splitting strength of beams loaded perpendicular to the grain by dowel type connections have been illustrated and the failure modes have been reported in detail. The main features of this experimental research is the wide range of the not-dimensional parameter a taken into consideration (from 0.1 to 0.7) and the slenderness ratios of the beams, well above those of previous experimental programmes.

The analysis of the results has allowed exploiting the effect on the splitting strength of the distance of the furthest fastener from the loaded edge of the beam. Moreover, the comparison with the failure loads of the beams with joints made by two dowels has allow the derivation of a possible way to take into account the effect of more rows of fasteners.

Finally, the comparison with the available prediction formulas and with the design rule of EC5 has allowed deriving the following:

– the formula of Ehlbeck, Görlacher and Werner is the one that gives the better appraisal of the splitting strength of the beams of this experimental research;
– the one of TACM van der Put is globally good but overestimates the strength of the beams of about 50 %,
– the design rule of EC5 is the one that shows the lower agreement with the test data

32-7-4 I Smith, P Quennevile
Predicting capacities of joints with laterally loaded nails

Abstract

Capacities of joints with laterally loaded nails may be predicted using "European yield" type models (EYM's) with various levels of complexity. EYM's presume that a nail and the wood on which it bears exhibit a rigid-plastic stress-strain response. Consideration is given in this paper to the 'original' model published by K.W. Johansen in 1949, an empirical approximation proposed by L.R.J. Whale and coworkers in 1987, and a curtailed and 'simplified' model proposed by H.J. Blass and coworkers in 1999. Predictions from the various EYM's are compared with experimentally determined ultimate capacities of single shear joints. Experiments covered a fairly broad range of combinations of 'head-side' and 'point-side' member penetrations. The impact of modeling assumptions is illustrated in the context of the Canadian timber design code. Suggestions are made regarding the necessity level of complexity for nailed joint models used in design.

Conclusions

European Yield Model type calculations give accurate predictions of the maximum load capacities of nailed single shear timber-to-timber joints. A 'simplified' model based on a proposal by Blass et al is the most appropriate choice. Two model equations should b used in conjunction with a restriction on the minimum head-side member thickness. Calculations are valid across the range of head-side and point-side nail penetrations that are of interest to designers. The reference capacities for laterally loaded nailed joints tha tare specified in the Canadian timber design code should be revised.

33-7-5 H J Blass, A Bienhaus, V Krämer
Effective bending capacity of dowel-type fasteners

Introduction

The load-carrying capacity of connections with dowel-type fasteners like bolts, dowels or nails may be determined on the basis of Johansen. According to Johansen the load-carrying capacity depends on the geometry of the connection, the bending resistance of the dowel and the embedding strength of the timber or wood-based material. For the bending resistance of the dowel, Johansen assumed the elastic moment capacity of the dowel's cross-section, the possible increase due to plastic deformations was disregarded. The design equations in Eurocode 5, which are derived from Johansen's work, are based on a rigid-plastic behaviour of both, the dowel under bending moments and the wood under embedding stresses and take into account the plastic moment capacity of the dowel. According to Johansen, three different failure modes are possible for timber-timber -connections in double shear. Failure mode 1 corresponds to the embedding failure of the middle or side member, respectively, where the embedding strength according to EN 383 is defined as "an average compressive stress at maximum load in a specimen of timber or wood-based sheet product under the action of a stiff linear fastener". In failure modes 2 and 3, apart from the embedding strength of the wood the bending capacity of the fastener is reached. Failure modes 2 and 3 of dowels loaded in double shear correspond to identical failure modes of dowels loaded in single shear.

According to EN 409 the yield moment of a nail is determined at a bending angle of 45°. For such a large bending angle, the whole cross-section of the fastener is assumed to be under plastic strain.

For bending angles below 45° only the outer areas of the cross-section of a fastener are deformed plastically. In this case, the reserve capacity can only be partially used and the fastener bending moment lies between the elastic and plastic bending capacity of the often circular dowel cross-section.

Effective bending capacity of dowel-type fasteners

For different types of fastener the bending angle $\alpha(d)$ at ultimate load was determined, resulting in minimum values of the bending angle α. For this purpose, the governing parameters were conservatively chosen, resulting in maximum values for the steel tensile strength and minimum values for the characteristic density. For connections with bolts or dowels, the tensile strength was chosen as 1000 N/mm^2 and the characteristic density as 350 kg/m^3.

A limiting deformation of 15 mm was assumed resulting in the following approximate expression for the effective bending capacity of bolts and dowels:

$$M_{y,k} = 0,24 f_{u,k} d^{2,7} \text{ Nmm}$$

where

$f_{u,k}$ fastener tensile strength in N/mm^2
d fastener diameter in mm

Using this equation for the bending capacity of bolts or dowels, the decreasing bending angle with increasing fastener diameter is implicitly taken into account.

33-7-6 H J Blass, B Laskewitz
Load-carrying capacity of joints with dowel-type fasteners and interlayers

Introduction

The design rules in Eurocode 5 for joints with dowel-type fasteners loaded perpendicular to their axis do not take into account an interlayer or a distance between the members to be connected although this may significantly influence the load-carrying capacity of the joint.

One example of a connection with an interlayer is a joist hanger attached at a shear wall. In this case, the load is transferred from the steel plate through the wood-based panel into the studs. In these cases not considered by the design rules the structural engineers and the building authorities are uncertain about the joint's load-carrying capacity.

The load-carrying capacity of timber-to-timber or steel-to-timber-joints with an interlayer may be derived according to the theory of Johansen which forms the basis of the design rules for dowel-type fasteners in Eurocode 5. A condition for this is the knowledge of the embedding strength of the different materials and the moment capacity of the dowel-type fasteners.

Conclusions

Because of the lack of knowledge about the load-carrying capacity of connections with dowel-type fasteners, where a wood-based panel is put between the members to be connected, the theoretical values of the load-carrying capacity based on the Johansen theory were derived and verified by a small number of tests. Two cases were considered in the theoretical models: one with a rigid connection between interlayer and one timber member and another case without a connection. For the case with connection, tests were performed with rigid and semi-rigid connections, respectively.

The slip between the timber member and the attached interlayer did not influence the load-carrying capacity of the connection. This statement is true, if the connection between interlayer and timber member is designed to carry the load introduced into the wood-based panel. In this case, the theoretical model based on a rigid connection may be used to calculate the load-carrying capacity of the connection. In all other cases, the conservative model disregarding a load transfer between wood-based panel and timber member should be used.

36-7-11 A Ranta-Maunus, A Kevarinmäki
Reliability of timber structures, theory and dowel-type connection failures

See 2.9 Strength grading

38-7-5 A Jorissen, A Leijten
The yield capacity of dowel-type fasteners

Abstract
Since the load carrying capacity of connections with dowel type fasteners is determined using the so-called Johansen equations, the yield capacity of the dowel type fasteners is mostly of importance. Some years ago there was a discussion within CIB-W 18 about the yield capacity of dowel type fasteners with diameters larger than 12 mm. It was found, that the yield capacity of those fasteners cannot be determined according to EN 409, where a bending test up to a bending angle of 45 degrees is prescribed; an angle of 45 degrees cannot be reached for "large" diameters.

Based on this discussion, equation (1) was introduced in Eurocode 5 for bolts and dowels.

$$M_{y,k} = 0,3 f_{u,k} d^{2,6} \tag{1}$$

where

$M_{y,k}$ characteristic value for the yield capacity in Nmm
$f_{u,k}$ characteristic tensile strength in N/mm^2
d bolt diameter in mm

More recently, equation (1) has also entered Eurocode 5 for the determination of the yield capacity of dowel type fasteners with thin diameters like nails and staples. Whether this is correct is discussed in this paper.

Conclusion and suggestion for code modification
Since the values for the yield capacity obtained directly from tests are higher than the values obtained whichever equation mentioned in this paper, it is not on the unsafe side to use Eq. (1) for small diameter dowel type fasteners as well.

Driven by the idea to keep it as simple as possible, we like to suggest to keep Eq. (1) into Eurocode 5 for all diameters.

41-7-1 M Snow, I Smith, A Asiz, M Ballerini
Applicability of existing design approaches to mechanical joints in structural composite lumber

Introduction
Despite recent increase of Structural Composite Lumber (SCL) consumption in construction, there is little knowledge about how to make efficient connections using such material. From a design perspective, it is essential that failure characteristics in SCL connections be recognized and understood. To date a very conservative approach based on an assumption of equivalency between performances of SCL and solid wood lumber connections has been adopted in day-to-day design practice in North America. The purpose of this paper is to investigate applicability of existing design approaches to mechanical joints in SCL based on comparison with test data collected at the University of New Brunswick .

Experimental work investigated failure characteristics of mechanical joints constructed using the types of SCL known as Laminated Veneer Lumber (LVL), Parallel Strand Lumber (PSL) and Laminated Strand Lumber (LSL). These materials are manufactured as proprietary products in nominal sizes similar to those for dimensional lumber, and their mechanical responses are highly dependent on manufacturing process variables. LVL, PSL and LSL corresponded to the most diverse alternative commercially available SCL products at the time of the investigation. Thus tested joints were surrogates for establishing how mechanical responses of all mechanical joints in SCL might differ from mechanical responses of similar joints in sawn lumber. Joints were constructed using commonly available dowel-type metal fasteners like bolts, nails and screws. Centre members were either SCL or sawn lumber, with the lumber alternative being the benchmark situation. Side members were made from sawn softwood lumber, SCL, steel plates or a high strength transparent plastic. Single or multiple fasteners in single or double shear arrangements were loaded in joint configurations.

General discussion

Mechanical joints, especially those made using multiple dowel-type fasteners, are complex systems exhibiting a range of complex failure mechanisms. Therefore it is extremely difficult, and arguably impossible, to achieve robustly accurate predictions of which failure mechanisms will govern for particular joint designs. Similarly it is very difficult to predict strengths of joints accurately. Further, if it is desired to predict design capacities of joints using simple models, whether they are explicitly empirical or semi-analytical, it should not be surprising if they fail to make robust predictions of governing mechanisms or of failure loads. Introducing SCL as alternative types of structural members in lieu of products like sawn lumber will not in general alleviate the conundrum in general, but could in specific cases. For example, as discussed in detail by Snow, using a material like LSL instead of easily splitting wood products largely eliminates the need to consider brittle mechanisms and simple EYM type design models yield good results. More generally the message to be gained from this paper is that particular design models will only yield good predictions under well defined circumstances (i.e. for those of which they are intended and calibrated.

Conclusion

The existing Canadian and Eurocode 5 design methods for joints in wood products made using dowel-type fasteners did not perform consistently well for joints with SCL members. The same is true for proposed alternative models that more explicitly address the possibility of brittle failure mechanisms governing the design of joints. It is imperative therefore that existing models and code rule be used with caution especially in an environment where the available range of of structural wood products is evolving rapidly.

41-7-2 P Quenneville, J Jensen
Validation of the Canadian bolted connection design proposal

Introduction

The Canadian wood design standard, 086 has recently adopted a design approach for the design of bolted connection based on the calculation of the resistances of the ductile and brittle failure modes. The engineering community realised that the approach to modify the European Yield Model (EYM) using in-row modification factors (or "row" or "end distance" factors) is not sufficient to account for all brittle failures. For instance, there are situations where the row spacing is not large enough to prevent group tear-out. Various sets of design equations have been proposed recently to account for the ductile and brittle failure modes for various types of bolted and dowelled connections. From work done during the last decade, a set of design equations is proposed to take into account the potential bolted connection failure modes, which are: yielding, row shear, group tear out (known as block shear in Europe), net tension and splitting perpendicular-to-grain. In this paper, the equations and their resistance equations are presented in their development and final design format and compared to past and recent experimental results. It is shown that these equations predict conservatively 99% of the specified resistance of individual available test data.

Conclusions

A set of design equations to determine the resistance of bolted connections failing in a ductile or brittle manner (bearing, row-shear, group tear-out, and net tension) for parallel-to-grain loading is proposed. These equations are based on a numerical model by Bickerdike (2006) and calibrated using experimental data from Masse et al., Mohammad et al., Jorissen, Quenneville and Mohammad, Mohammad and Quenneville, Shin, Reid and Bickerdike.

These equations were used to predict the resistance of dowelled connections tested by Hanhijärvi and Kevarinmäki. The predictions of the averages prooved to be adequate for this set of experimental data as well.

The set of equations were also used to predict specified resistances (parallel-to-grain) for the various connections configurations tested. These values were compared to individual test data of each configuration tested. The equations provide safe predictions in 99 % of cases.

41-7-3 A Polastri, R Tomasi, M Piazza, I Smith
Ductility of moment resisting dowelled connections in heavy timber structures

Introduction

Despite the relative brittleness of large timber members loaded in bending, shear or tension structures built from such material are widely considered to perform well during seismic events. Good seismic behaviour is often attributed to the high ratio between strength and mass that timber components posses and the ability of completed structural systems to dampen and attenuate motions resulting from ground shaking. However, this does not mean that timber structures per se are inherently resistant to seismic actions, which would for example be just as irrational as claiming them to be an inherent fire risk. Like those made primarily from other materials, particular timber construction systems owe any ability to resist damaging effects of external mechanical actions to many factors. Firstly it is important that constructed systems employ inherently stable geometric forms at systemic and substructure levels, and have ability to develop alternative load paths following one or more localised failure events. Secondly, and especially under seismic actions, it is important that systems and substructures be able to accommodate local deformations in components without failure of any embedded connections. That in turn depends on the geometry of components (i.e. individual members and connections) and how appropriately they combine materials. These requirements reflect that in many timber structures ability to absorb kinetic energy and attenuate effects of large amplitude ground motions is strongly dependent on energy dissipation (damping) associated with plastic deformation of metal parts in mechanical connections. Apparent ductility / damping in connection responses results from actual ductility in metal parts, as does the apparent damping they impart to completed systems. This is why only structures that permit their connections to deform substantially prior to occurrence of systemic damage can be expected to perform well if overloaded during seismic events.

The classical structural form adopted in Italy for industrial buildings made of timber employs parallel arranged two-hinge or three-hinge portal-frames having moment resisting

General discussion

Moment carrying connections of types that are common in timber portal-frames in Italy meet the requirements of EC8. This statement is made primarily based on experimental proof that such connections have ductile responses and because there is evidence that individual dowels in such connections responded in a ductile manner for levels of deformation likely to occur within completed building systems. A second point of conformance with requirements of EC8 is that the degradation to be expected under repetitive deformations is not excessive.

The above said, it is important to emphasise that dowels employed in tested connections were relatively slender and therefore developed plastic deformation in bending. The implication is that assuming compliance with requirements of EC8 is contingent on connections having slender dowels. In practice that equates to requiring that connections fail by simultaneous plastic deformation of dowels and crushing of timber beneath them. Site quality control must ensure that the correct grade of steel is used because employing the wrong grade can change the governing failure mechanism to be brittle (i.e. using higher grade steel could prevent formation of necessary plastic hinges).

The formulae of EC5 for predicting capacities and stiffness of simple dowel joints forms a reliable basis from which to predict the behaviour of tested moment connections and it is therefore presumed those same provisions are reliable for design.

Conclusions

Experiments have verified that it is possible to design satisfactory moment connections for two-hinge or three-hinge portal frame structures when effects of seismic actions must be resisted. In practical terms this means that the requirement of Eurocode 8 can be complied with. With only slight modification the connection design provisions of Eurocode 5 can be used to design such moment connections.

41-7-5 U Hübner, T Bogensperger, G Schickhofer
Embedding strength of European hardwoods

Introduction

The embedding strength tests presented in this paper were realised according to EN 383:2007 and with the species beech (Fagus sylvatica L.), ash (Fraxinus excelsior L.) and black locust (Robinia pseudoacacia L.). The test results will be used to dilate the existing base for the standardisation.

Furthermore, the test results were analysed together with those of other authors.

42-7-4 S Franke, P Quenneville
Embedding strength of New Zealand timber and recommendation for the NZ standard

Introduction

For all connections it is important to predict the failure strength as accurately as possible. This includes both the ductile and in some cases especially in timber construction, the brittle failure as well. For the calculation of the ductile failure strength, the European Yield Model (EYM) is used in many standards and accepted as a very accurate model. It forms the basis of the European timber standard Eurocode 5, EN 1995-1-1:2004.

The development of this is based on a multitude of embedding and joint tests with different European and North American wood species by many researchers. Furthermore a continuous adaptation and improvement is reported overseas such as in Hübner et al. 2008. The most important parameters for the EYM are the fastener yield moment and the timber embedding strength, which are known for most of the softwoods and tropical hardwoods.

In the current NZ timber standard NZS 3606:1993, the design concept for bolted connections is not based on the EYM, but depends only on the diameter and the timber thickness. It doesn't predict the different types of failure and overestimate the joint strength partially. There are no embedding strength values, which can be used for the Johansen's yield theory to estimate the yield strength of joints. Furthermore, no formulas are available for the design of joints with the engineered wood product Laminated Veneer Lumber (LVL), which uses becomes more important in structural members. To implement the EYM design concept in the current New Zealand design standard for mechanical connections, it is thus necessary to investigate the material behaviour and to determine the embedment values for Radiata Pine timber and also for Radiata Pine LVL, the two main products used in New Zealand constructions.

Embedding tests parallel, perpendicular and under various load-to-grain angles with different dowel diameters with LVL were conducted and compiled together with results of NZ Radiata Pine to build a database of embedding strength values to implement the European Yield Model into the NZ standard. The embedding strength was evaluated using the 5 %-offset method, the extended proportional limit load according to DIN 52192 and the maximum load, which is either the ultimate load or the load up to 5 mm displacement, according to EN 383:1993 and ISO 10984 respectively. For the embedding strength from Radiata Pine lumber, results from other researchers were used. There is also a comparison with the predicted embedding failure from Eurocode 5. Moreover, the paper presents a comparison of the different available test standards for determining the dowel embedding strength.

Conclusions

Based on the current results from Radiata Pine LVL, the formulas of EC 5 for estimating the embedding strength $f_{h,0}$ for LVL can be used, but it needs further tests to check the sensitive strength behaviour of dowels with a diameter of 8 mm or smaller and to ensure these results. We propose to use a constant factor of 1.39 for the reduction factor k_{90}, which is the mean value of the three lower values of the 8 mm, 16 mm and 20 mm dowels, cp. Figure 12. A summary is given with the equations (7), (8) and (9).

Based on the results for Radiata Pine, lumber we propose to estimate the embedding strength $f_{h,0,k}$ and the reduction factor k_{90} regardless to the dowel diameter as a constant value as follow:

4.4 DOWEL-TYPE FASTENERS LOADED PERPENDICULAR TO GRAIN

22-7-2 J Ehlbeck, R Görlacher, H Werner
Determination of perpendicular-to-grain tensile stresses in joints with dowel-type fasteners – A draft proposal for design rules

Introduction
Joints with dowel-type fasteners loaded at an angle to the grain direction of the wood cause in addition to the embedding stresses considerable local perpendicular-to-grain stresses next to the fasteners. These stresses may under certain conditions lead to failure at a load level lower than the load-carrying capacities of the fasteners themselves.

A simplified method to take into account these stresses is given in the CIB-Code and was also provisionally accepted in the draft Eurocode 5. This design method is, however, unsatisfactory and can lead to uncertainties, e.g. for single loads applied to glulam beams and acting perpendicular to the grain direction. This simplified method does not sufficiently take into account:

– the joint's geometry;
– the number of fasteners in the joint;
– the distribution of the fasteners over the beam depth;
– the perpendicular-to-grain tensile strength of the wood in relation to the actually stressed volume;
– the ratio between the distance a_r of the furthest row of fasteners from the loaded beam edge and the beam depth *h;*
– the range of the area stressed by the perpendicular-to-grain acting load.

Based on tests and theoretical reflections performed at the University of Karlsruhe during the last decade a design procedure for joints with dowel-type fasteners is evaluated and presented for discussion.

32-7-3 M Yasumura, L Daudeville
Design and analysis of bolted timber joints under lateral force perpendicular to grain

Introduction
The fracture of wood is one of the major causes of brittle failure in timber structures. It brings serious damages of timber structures under seismic action by reducing the energy dissipation. Fracture of wood occurs frequently at the joints subjected to the force perpendicular to the wood grain. This failure is not always predictable because the design of timber joints is generally based on the yield theory that does not include the failure of joints due to the fracture of wood. The fracture mechanics is one of the most effective methods to analyze the fracture of timber structures. It is particularly useful for estimating the maximum strength of the joints when the crack propagation is stable. In **Paper 29-7-8**, the fracture of a single bolted joint subjected to a lateral force perpendicular to the grain was analyzed by means of the ASM (Average Stress Method) and the LEFM (Linear Elastic Fracture Mechanics). It showed that the ASM and the LEFM were appropriate tools to predict the crack initiation and propagation of the bolted timber joints subjected to lateral force perpendicular to the grain, respectively. In this study, the load carrying capacities of a single and multiple bolted joints obtained from the lateral loading tests were compared with the results of simulation by means of the LEFM and yield strength.

Conclusions
It was found that the load carrying capacity obtained from the simulation by means of the LEFM agreed very well with the experimental results, and the LEFM was the appropriate tool to estimate the failure of bolted timber joints. It was also found that the load carrying capacity of the joints subjected to lateral force perpendicular to the grain was mostly smaller than the yield strength, and they failed due to the fracture of the wood before the yielding of bolts. Special consideration should be done on design of joints subjected to the lateral force perpendicular to the grain.

34-7-2 M Ballerini, R Bezzi
Numerical LEFM analyses for the evaluation of failure loads of beams loaded perpendicular-to-grain by single-dowel connections

Abstract
The paper presents the results of a numerical study performed to evaluate the splitting strength of beams loaded perpendicular-to-grain by single-dowel connections. The investigation, carried out in the framework of LEFM theory, has allowed the derivation of the stress intensity factors (SIF) of beams loaded at mid-span, for different crack lengths and for different distances a_{ef} of the dowel from the loaded edge of beams.

The stress intensity factors are computed by means of the virtual crack closure integral method and by means of the displacement extrapolation technique.

On the basis of the computed SIFs the splitting failure loads of the beams are derived through the classical Wu's fracture criterion.

Finally, comparisons between the results of the two SIF computational approaches and between the numerical failure loads and the experimental ones are reported and discussed.

Introduction
The design of dowel-type connections that transfer perpendicular-to-grain forces to timber elements is actually performed with respect mainly to the strength of the connections and with little emphasis to the splitting resistance of the beams. In spite of this, it is well known that frequently it is the formation and the propagation of a crack along the grain of the timber elements that limits the resistance of the whole system.

The prediction of the splitting strength of beams loaded by dowel-type connections is a difficult task since it is affected by the influence of many parameters. The main ones are the distance of the furthest row of fasteners from the loaded edge of the timber elements and the connection geometry, summarized in number, size and spacing (horizontal and vertical) of connectors. Although it is well recognised the fundamental contribution of a_r, its role is not completely understood due to above-mentioned not negligible influence of the connection geometry, which masks and makes difficult the analysis of the experimental data.

On the other side, the investigation of the a_{ef} role by means of single-dowel connections is not easy to perform due to the limited embedding strength of timber perpendicular-to-grain. This can be noticed mainly for high values of a_{ef}, and consequently higher splitting strengths of the beams.

For what concerns the theoretical prediction of the splitting strength of the beams, two different approaches are available. The first one, due to Ehlbeck, Görlacher and Werner, is based on the best fitting of the experimental results and on theoretical considerations related to the Weibull theory and to the distributions of the tensile stresses perpendicular-to-grain. The latter one, due to TACM van der Put, is derived on the basis of an energetic approach in the framework of the Linear Elastic Fracture Mechanics.

Although both approaches give reasonable predictions with respect to the experimental data, the structures of prediction formulae are completely different highlighting distinct evaluations of the contributions of the various parameters.

To overcome the above-mentioned limits of experimental investigations, numerical FE analyses can be profitably used to obtain the right contribution of the various parameters affecting the splitting strength of beams loaded by perpendicular-to-grain joints. Particularly, for the nature of the problem considered, numerical analyses in the framework of the Linear Elastic Fracture Mechanics (LEFM) seem the most effective ones. Different researchers have already applied LEFM for the study of the strength of timber elements when crack propagation occurs.

In the following the results of a numerical survey devoted to the evaluation of the influence of a_{ef} on the strength of beams loaded by single-dowel connections are illustrated. Synthetically, the approach consists in the numerical derivation of the stress intensity factors at the crack tip for different crack lengths, different dowel positions and two beam heights (200 and 400 mm); in an evaluation of the failure loads of the timber beams by means of the application of the Wu's fracture criterion with appropriate values of the fracture toughness.

Conclusions
The presented parametric numerical work is characterized by a considerable amount of analyses performed to catch the influence of the fundamental parameters a_r and α on the strength of beams with single-dowel connections. Moreover, two different SIF computational approaches have been used for comparison and validation purposes.

The results of the research allow deriving the following outcomes:

- the results given by the two SIF computational approaches are in good accord, however the ones of the crack closure integral method are more reliable;
- the analysis of the course of computed SIFs has allowed to highlight the influence of parameters a_r;
- the numerical failure loads derived on the basis of Wu fracture criterion are in a good agreement with the results of the experimental data;
- the course of these numerical failure loads has allow to highlight the influence of main parameters (a_r, and α) on the splitting strength of beams also for those cases where testing is problematic because of the high embedding stresses;
- the fittings made on the course of the numerical failure loads show how the structure of the prediction formula of van der Put is in a quite good agreement, the parameters of the power fitting however are different for the two beam sizes;
- lastly, a relationship between the numerical critical crack lengths and α, has been derived.

34-7-3 H J Larsen, P J Gustafsson
Dowel joints loaded perpendicular to grain

Abstract
The results of tests with doweled joints in LVL loaded perpendicular to grain are reported. Four joint configurations were tested: 1, 2 and 3 dowels in line and 2 dowels side by side. Further tension and splitting properties perpendicular to grain were determined. Some of the specimens were stored and tested in standard climate (23 °C/65 % RH) but most were stored in an open barn in Southern Sweden. The specimens were long-term loaded in the barn with target load levels relative to the short-term strength of 65 % and 80 %. Load was applied winter, spring, summer, autumn 2000 and winter 2001 to determine the influence of different climatic conditions. The main results are: The short-term strength is not influenced by the seasons. The short-term strength of the joints can be predicted by a simple equation based on linear fracture mechanics. The load-carrying capacity of two dowels side by side and a single dowel are identical. The effect of loading time for joints loaded perpendicular to grain is much more severe than for timber: the time to failure under a load level of 60 per cent is only 180 days. A safe load level is only about 30 percent of the short-term strength. The drying distortions of the specimens used to determine the tensile strength (40x70x280 m) have a great influence on the results, the mean value is only 0.67 MPa, the minimum value only 0.25 MPa.

35-7-7 A J M Leijten
Splitting strength of beams loaded by connections, model comparison

Introduction
Initially the problem of perpendicular to grain splitting of beams by connections was tried solved by empirical models. Lately models based on fracture mechanics were developed. Empirical models are only valid within the range of parameters tested therefore there is more credit in pursuing general physical oriented models. Empirical models for the problem concerned tend to have many parameters to take all possible influences and effects into account as is shown by the German model evaluated below. The last years two models are published both based on fracture mechanical principles. The first is by Van der Put & Leijten (**Papers 33-7-7** and **34-7-1**) the other by Larsen & Gustafsson (**Paper 34-7-3**).

Model (1) by v. d. Put/Leijten (Eurocode5)

$$F_u = 2,58\mu b\sqrt{GG_c \frac{h}{1-h_e/h}}$$

Model (2) by Larsen /Gustafsson

$$F_u = 2,82\eta b\sqrt{GG_c h_e}$$

Model (3) is a simplified Model (1), see **Paper 33-7-7**

$$5,16\mu b\sqrt{GG_c}\, h_e / h$$

where:

F_u splitting shear strength of the beam loaded by a connection at mid span in [N]. For connections at the end of a cantilever the splitting strength is $F_u/2$.
b width in [mm]
μ behaviour factor
G shear modulus [N/mm^2]
G_c fracture energy [N/mm^{1,5}]
h beam depth in [mm]

German model

$$F_{90,k} = k_s k_r \left(6.5 + \frac{18a^2}{h^2}\right) \cdot (t_{ef} h)^{0.8} \cdot f_{t,90,k} \tag{4}$$

where:

$$k_s = \max\begin{cases} 1 \\ 0.7 + \dfrac{1.4 a_r}{h} \end{cases} \quad and \quad k_r = \frac{n}{\sum_{i=1}^{n} \left(\dfrac{h_1}{h_i}\right)^2}$$

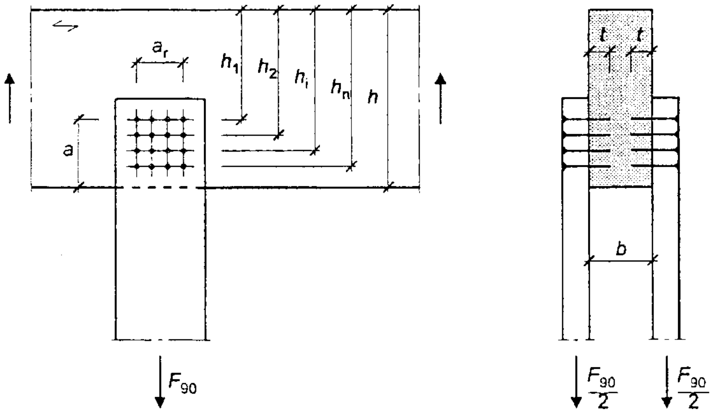


Figure 1: Example of connection with nails.

Conclusions

It can be concluded:

- The efficiency parameter η of Model (2) is in many cases outside it validity range ($0 < \eta < 1.0$)
- Model (2) is able to cope with different type of connections and in many cases underestimates high strength results and overestimates low strength results
- Differences between Model (1) and (3) are always small
- Model (4) is apparently, unable to cope with the Canadian data sets regarding the mean of the test values and grossly overestimate the capacity. On the lower 5 % Model (4) is conservative.
- Model (1) transformed to the Eurocode 5 design equation fits the lower 5% rather well.

Summary

Evaluation of the models shows that Model (1) and (3) are well able to represent the mean and 5 % test values of all data series. The empirical Model (4) fit to the lower 5 % test data is conservative while for the mean it grossly overestimates the strength capacity in a number of data sets.

35-7-8 O Borth, K-U Schober, K Rautenstrauch
Load-carrying capacity of perpendicular to the grain loaded timber joints with multiple fasteners

Abstract

Based on the simple numerical model described in Proceedings of the International RILEM Symposium „Joints in Timber Structures", Stuttgart, 2001, ultimate loads of practice-related joints with multiple fasteners can be estimated easily in the framework of the Linear-Elastic Fracture Mechanic (LEFM). As failure criteria exclusively those of the Linear-Elastic Fracture Mechanics are accepted.

First the numerical model will be checked for a sufficient consideration of properties and parameters of the physical model. For further calculations suitable assumptions and simplifications are made. The assumption of the critical crack length is the main focus of these investigations. They will be verified by acknowledged theoretical approaches and comparisons with experimental results from other scientists.

As a result, ultimate loads of practice-related joints with multiple fasteners can be specified. Using the numerical model described above, the ultimate load of these joints basically depended on the configuration of the fasteners. For beams with a smaller girder depth fracture mechanic concepts can possibly lead to an overestimation of ultimate loads.

Summary

The examinations have shown the possibility to determine the ultimate fracture loads for practice-related joints with mechanical fasteners within the framework of the Linear-Elastic Fracture Mechanics.

An increasing distance between fasteners and loaded edge was represented in the structural model as an increase of the load-carrying capacity. As the most important parameter the arrangement and number of the fasteners were determined. Other parameters are circumstantial. By use of fracture-

mechanical failure criteria the distribution of the load application has more positive effects on the load-carrying capacity in grain direction than in force direction perpendicular to the grain. Therefore the presented structural model corresponds with experimental results.

Exceptions are perpendicular to the grain loaded joints on structural members with low girder depth (approx. $H < 30$ cm). Here the low stiffness results in compressing cracks with a lower distance from the loaded edge. The consequence is an overestimation of the load-carrying capacity. The numerical model also supplies an overestimation of the fracture loads of perpendicular to the grain loaded joints arranged above the axis of gravity. For such joint configurations other failure criteria become authoritative.

35-7-9 M Yasumura
Determination of fracture parameter for dowel-type joints loaded perpendicular to wooden grain and its application

Introduction

During the CEB-W 18 Meeting in Delft, 2000, a formula to predict the lateral capacity of dowel-type joints loaded perpendicular to the wooden grain was proposed (**Paper 34-7-9**). This formula includes what we call the "fracture parameters" consisting of G and G_c. Instead of applying individual material properties of G and G_c which are difficult to determine from the material tests, fracture parameters will be obtained by conducting simple tension test of dowel-type joints consisting of a single or multiple dowel fasteners. This study proposes the test method to determine the fracture parameter for dowel-type joints loaded perpendicular to the wooden grain. Tension tests of dowel-type joints were conducted with different edge and end distances, and the suitable configuration of specimen and dowel disposition are proposed with the aid of FE analysis. The obtained fracture parameters were applied to validate the formula for predicting the failure load of dowel-type joints.

36-7-7 M Ballerini, A Giovanella
Beams transversally loaded by dowel-type joints: Influence on splitting strength of beam thickness and dowel size

Abstract

The paper reports the results of a new set of experimental tests on beams loaded perpendicular-to-grain by dowel-type connections. The experimental programme consists of two different test series. The first one, which concerns large specimens with multiple dowel connections, has been performed essentially to investigate the actual influence of the beam thickness on the splitting strength. The second one, which involves smaller specimens with single dowel joints, has been carried out to confirm the soundness of the results of a previous research and to demonstrate the limited effect of the connection strength on the splitting resistance of beams.

The experimental results of both test series are illustrated and discussed with reference also to the results of previous research. Finally, the failure loads are compared with the bearing capacity predicted by the final drafts of the new European and German design codes for timber structures.

Conclusions

The results of a new experimental programme on the splitting strength of beams loaded perpendicular to the grain by dowel type connections, performed to investigate the actual influence of the thickness of the beams and of the fasteners size, have been illustrated and discussed in detail.

The main features of this experimental research are the dimensions of the beams of the first series (800 mm high beams with a width ranging form 80 to 200 mm), and the size of fasteners' diameter.

The analysis of the results allows the following statements:

– the tests data show that beam thickness affects linearly the splitting strength;
– the tested joint configurations show to have no, or a very limited influence on the strength;
– the results on beams with single dowel connections show that the dowel diameter (and consequently the embedding strength of the joint) doesn't affect the splitting strength of beams;
– consequently, the tests results of the present research confirm the soundness of the results reported in **Paper 32-7-2.**

Finally, the comparison of tests results with the design formulae included in the latest drafts of the new Eurocode 5 and E DIN 1052 shows the following:

– the design rule proposed by Eurocode 5 describes quite well the strength of specimens with single dowel connections but it appears not able to estimate the strength of specimens with a more complex joint configuration;
– the design rule suggested by new DIN 1052 is able to predict quite well the strength of specimens of both test series in spite of the fact that it estimates a strength greater than zero when the effective depth tends to zero.

36-7-8 J L Jensen
Splitting strength of beams loaded by connections

Abstract
A fracture mechanics model for calculation of the splitting strength of dowel-type fastener joints loaded perpendicular to grain has previously been presented by Van der Put & Leijten, Leijten & Jorissen, and Leijten, and now forms the basis for design in Eurocode 5. The Van der Put/Leijten model is based on an assumed distribution of sectional forces in the cracked part of the beam. Especially the disregard of normal forces has been subject to some discussions at previous CIB-meetings. In the present paper, a model is presented using a distribution of sectional forces, which includes normal forces and satisfies the static equilibrium equations. The solution obtained is essentially the same as the Van der Put/Leijten solution, but is derived without any simplifying assumptions. The Van der Put/Leijten expression appears as a special case of the present model, namely by assuming zero crack length or by only including the contributions from shear deformations.

The abovementioned models consider a single joint loaded perpendicular to grain. However, tests have indicated that the failure load of two single joints spaced along the grain is less than twice the failure load of a single joint. An attempt was made to use models based on the same approach as applied to the single joints to analyze beams with two joints. However, the two-joint models based on the present approach seem not to successfully explain the interaction phenomenon observed in tests.

Conclusions
A linear elastic fracture mechanics model for calculation of the splitting failure load for dowel-type fastener joints loaded perpendicular to grain was presented. The model is based on an approach similar to a previous model presented by Van der Put & Leijten, but includes normal forces in the cracked parts of the beam. The Van der Put/Leijten solution appears as a special case, namely for zero crack length or by only considering contributions from shear deformations.

Models for two joints spaced along the grain were developed based on the same approach as used in the abovementioned model. Those models predict either the same failure load as the single joint model (cracks merged between the joints) or twice the failure load of the single-joint model (cracks not yet merged between the joints). This, however, seems not valuable for explaining existing test data.

36-7-9 J L Jensen, P J Gustafsson, H J Larsen
A tensile fracture model for joints with rods or dowels loaded perpendicular-to-grain

Abstract
Tensile splitting failure from glued-in rods or dowels loaded perpendicular to grain is studied. A new quasi-nonlinear fracture mechanics model based on beam-on-elastic-foundation (BEF) theory is presented. The model is applied to plates with edge dowels, to beams with dowels, and to beam splice joints made up of rods glued in along grain. The BEF theory is verified by FEM calculations and test results for the same three kinds of joints. The test results for dowel joints in plates agree very well with the theory. The same applies for dowel joints in beams with small edge distances. For larger edge distances both the BEF and the FEM calculations overestimate the load-carrying capacity. In this case it may partly be explained by a change of failure mode from pure splitting to a combination of compression failure perpendicular to grain and splitting. Also the test results for beam splice joints with glued-in rods agree well with the theory.

37-7-5 M Ballerini
A new prediction formula for the splitting strength of beams loaded by dowel type connections

Abstract
The paper presents a new semi-empirical prediction formula for the splitting strength of beams loaded perpendicular-to-grain by dowel-type connections.

The formula is derived on the basis of the results of the main experimental research carried out by different authors by means of the analysis of the influence of different parameters. It takes into account the influence of both the beams' size and the connections' geometry. The formula is initially derived for beams loaded by single-dowel connections and then corrected to cover the case of beams loaded by multiple-dowel connections.

The prediction ability of the formula is illustrated, discussed and compared with the prediction formulas available from literature which have been adopted in the new European and German design codes for timber structures. Finally, a design proposal is derived and compared with experimental results.

Introduction
Beams loaded on their depth by dowel-type connections can fail by splitting at load levels which may be considerable lower than the ones of the connections or of the beams. This is particularly true when the distance from the loaded edge of the furthest row of fasteners (h_e) is small compared to the beam height (h).

Due to this reason such engineering solution for the transmission of forces should be avoided or adequately designed by means of properly reinforcements.

Nevertheless by the presence of possible reinforcements, the need of a valid prediction formula for the splitting strength of beams loaded by dowel-type connections is widely recognised. Unfortunately, the evaluation of the splitting strength is a difficult task due to the large number of involved parameters.

The most recent drafts for the new European (EC5) and German (DIN 1052) design codes for timber structures suggest two different design formulae which take into account the several parameters in a very different way.

The design formula embodied in the draft of the new EC5 is based on the work originally developed by Van der Put, on the basis of an energetic approach in the framework of the Linear Elastic Fracture Mechanics, and recently put forward again in **Paper 35-7-7.** Essentially, this formula assumes a linear relationship with the beam thickness and an influence of the square root of the distance of the furthest row of fasteners from the loaded edge of the beam (h_e). On the contrary, it neglects any influence of the connection geometry.

The design formula embodied in the draft of new DIN 1052 is an evolution of the prediction formula derived by Ehlbeck, Görlacher & Werner (**Paper 22-7-2**), which was based on both empirical and theoretical considerations. With respect to the Van der Put formula, it assumes a different influence of the loaded edge distance (by means of the non-dimensional parameter $\alpha = h_e/h$) and also a different non-linear influence of both the beam thickness (b) and the beam height (h) as a result of the assumed Weibull failure theory. Moreover, it explicitly considers the influence of the joint configuration on the splitting strength.

Both prediction formulae seem to have some correct and some wrong assumptions.

The formula of Van der Put seems to be able to predict more correctly the influence of the beam height (and also of the beam thickness), as reported in the recent experimental and numerical researches of Ballerini and Yasumura performed on beams loaded by single-dowel connections. Nevertheless, it doesn't takes into consideration the effect of the joint geometry which is instead clearly detectable in all the experimental researches: from the oldest (Möhler & Lautenschläger) to the more recent ones.

On the other side, the formula embodied in the recent draft of the new German design code for timber structures correctly takes into account the effect of the joint geometry, however it seems less reliable from the viewpoint of the influence of beam height and thickness.

In the present paper a new semi-empirical prediction formula is derived on the basis of theoretical considerations and on the main outcomes of the different experimental researches. Initially, the formula is derived for the case of beams loaded by single-dowel connections. Afterwards, the formula is corrected to cover the case of beams loaded by multiple-dowel connections. This second step is developed considering initially the effect of the width of the connection and then the effect of the connection height.

Conclusions

The derivation of a new semi-empirical prediction formula for the splitting strength of beams loaded by dowel-type connections has been illustrated in detail. The formula is based on the results of some theoretical and numerical works of different authors and on the data of a large number of experimental researches.

The prediction capability of the new formula has been compared with the one of two different design formulae embodied in the new Eurocode 5 and in the new E DIN 1052.

As a result of the comparison the following drawings can be derived:

- the prediction formula of Van der Put is able to predict quite well the strengths of beams loaded by single-dowel connections but it is prevented to predict the ones of beams loaded by multiple-dowel connections since it doesn't take into account the effect of the joint geometry;
- the prediction formula embodied in the new DIN 1052 is not very reliable in the prediction of the strength of beams with different heights, however it is able to predict well the strength of a large set of data and globally of the whole set of experimental data;
- the new prediction formula has the same prediction ability of the DIN formula, it predicts better the strength of beams with single-dowel connections but it is less effective with respect to some experimental data set.

Finally, since the new formula is very easy and it has a good level of prediction ability, a simple design proposal has been derived and compared with experimental data.

38-7-1 M Ballerini, M Rizzi
A numerical investigation on the splitting strength of beams loaded perpendicular-to-grain by multiple dowel-type connections

Abstract

The paper presents a numerical study on the splitting strength of timber beams loaded perpendicular-to-grain by dowel-type connections. The aim of the parametric numerical investigation is to find out the influence of main connections parameters on beams splitting strength. The analyses are carried out in the framework of Linear Elastic Fracture Mechanics (LEFM). They are performed on beams loaded at mid-span by both single and multiple dowel connections. The strength of beams is derived by means of stress intensity factors (SIFs) at the crack tips in mode I and II through the classical Wu's fracture criterion.

The main investigated parameters are the connection width (k_r), the connection depth (h_m), and the number of rows of fasteners (n); they are analysed for different beam heights (h) and for different distances of the furthest row of fasteners from loaded edge (h_e).

The numerical results are compared with relationships embodied in a new semi-empirical prediction formula– based on a statistical survey on experimental data – recently developed by the author. The first part of the paper presents shortly the new semi-empirical prediction formula and its prediction ability. The second part of the paper reports the main results of the parametric numerical analyses and their comparison with the effect assumed by the semi-empirical prediction formula. Finally, the strength of a beam with a complex connection is numerically investigated and the result compared with the experimental data and the predicted value.

Conclusions

A parametric numerical investigation or splitting strength of beams loaded by stiff multiple-dowel connections has been presented in its main aspects. The aim of the numerical parametric study is the investigation of the effect of connections geometrical parameters on the overall splitting strength of beams. For comparison purpose, a recently derived semi-empirical formula based on both numerical and experimental results has been shortly illustrated.

43-7-6 B Franke, P Quenneville
Failure Behaviour and Resistance of Dowel-Type Connections Loaded Perpendicular to Grain

Introduction

The load capacity of mechanical connections loaded perpendicular to grain is limited by either a brittle failure, such as splitting of the wood, or ductile failure behaviour, such as the bending of the fasteners and/or the embedding or bearing failure of the wood under the fastener. Most international design standards predict the ductile failure using the accepted European Yield Model (EYM) with good accuracy. However, different design equations are used for the prediction of the splitting failure of the wood. These

are, on the one hand the design equations in the EN 1995-1-1:2004 based on the fracture mechanics and on the other hand, the German design equations in the DIN 1052:2008 based on a strength criterion.
Experimental test series are traditionally used to investigate the load capacities and to develop design concepts. A further option is to use numerical simulations of the failure behaviour of connections. A numerical model developed was used to investigate the nonlinear behaviour including the ductile and brittle failure of dowelled connections loaded perpendicular-to-grain with different geometry layouts. The numerical results reached show that important parameters change the load capacities, the stress-strain distribution and the failure behaviour of the connection. The comparison of the numerical load capacities with the EN 1995-1-1:2004 shows that connection parameters such as number of rows and columns or the connection width are not considered. The investigation of these different parameters presented in this paper shows at this stage potential to improve the design methods.

Conclusion

The results show that the German design standard DIN 1052:2008 respect the most important parameters and shows a good agreement with the load capacities reached in the numerical test series and experimental test results. The use of the European design standard EN 1995-1-1:2004 for the prediction of the load capacity of dowel type connections loaded perpendicular to grain should be used very carefully because the effect of important parameters are not taken into account and in some case the load capacities were over predicted.
The German design standard can be improved through calibration of the factors k_s, k_g and k_r which respect the connection width or the number of rows or by introducing a facto r taking into account the number of columns. In further research, an extended numerical parameter study with additional geometry parameters as well as experimental test series can complete the results obtained. Further studies are being undertaken to develop analytical solutions considering these parameters.

4.5

GLUED JOINTS

20-18-2 P J Gustafsson
Analysis of generalized Volkersen-joints in terms of linear fracture mechanics

Abstract

The strength of lap joints in pure shear is studied theoretically by a non-linear fracture mechanics approach, taking into account the complete τ-δ curve of the bond line, including its softening branch. Bending effects and peel stresses are not considered.

The purpose is to present a unified method of analysis which incorporates more conventional methods as special cases and which makes it possible to study the significance of various parameters. A joint-characteristic brittleness ratio is introduced. Fracture energy, G_f, defined as the area under the **τ-δ** curve, is found to be an important parameter. The significance of the shape of the τ-δ curve is studied and the optimal shape is deduced.

The range of application for more conventional methods of analysis is indicated and limiting conditions for application of 1D analysis of overlap joints are proposed. A linear elastic fracture mechanics strength expression is developed and a method for simple calculation of the self-similar stress distribution is proposed. For cases where adherend fracture may occur, it is found that a small increase in adhesive strength may produce a drastic decrease in joint strength.

Conclusions

From the above study of 1D joints in pure shear, the following conclusions may be drawn.

– By means of a non-linear fracture mechanics approach, taking into account the complete τ-δ curve of the bond line, it is possible to analyse the strength of joints with different characteristics in a unified manner. The joint strength may be described as a function of a brittleness ratio which also characterizes the joint and its fracture performance.
– Fracture energy, G_f, is an important parameter which depends on both the ascending and descending branch of the τ-δ curve.
– For ductile joints the strength, τ_f, of the bond line is the governing strength parameter while for brittle joints the strength is determined by G_f. In the intermediate range both τ_f, G_f and the shape of the τ-δ curve have influence.

– The unified theory makes it possible to estimate when more conventional methods of strength analysis are applicable. It is also estimated when the 1 D analysis is applicable.
– A linear elastic fracture mechanics expression for the strength of 1 D joints can be obtained. The self-similar stress distribution in the fracture process region can be calculated for an arbitrary non-linear τ-δ curve in a simple manner.
– If adherend fracture may occur, a small increase in adhesive strength may produce a drastic decrease in joint strength.

20-18-3 H Wernersson, P J Gustafsson
The complete stress-slip curve of wood-adhesives in pure shear

Abstract
A test method and test results regarding the complete stress-slip curve of adhesive bond lines in pure shear are presented. The test program concerns the bond between wooden adherends (Pinus Silvestris) and comprises the adhesives PVAc, polyurethane and resorcinol/phenol, tested after different curing time, 4 and 16 days, and at various rates of deformation, 1 mm/min - .0625 mm/min. The test set-up might be described as a further development of the thick adherend test and is designed in order to enable a stable test performance also during strain softening and fracture. The test set-up is also designed at the purpose of achieving pure shear, i.e. zero peel stress, uniform stress distribution and avoidance of adherend fracture.

It is found possible to obtain a complete τ-δ curve of a bond line. The deformation capacity of a bond line during plastic hardening and softening is found to be very large as compared with the first linear elastic deformation. From the test results, τ-δ curve, peak shear stress, τ_f, fracture energy, G_f, and a bond line characteristic ratio, τ_f^2 / G_f is evaluated. Characteristic differences between the performances of the different adhesives are found. In some cases, curing time and rate of deformation are found to have a significant influence on the performance of the bond line.

34-18-1 R J Bainbridge, C J Mettem, J G Broughton, A R Hutchinson
Performance Based Classification of Adhesives for Structural Timber Applications

Abstract
Many of the recently developed and emerging technologies aimed at enhancing the opportunities for structural timber require the use of adhesives. Examples of innovative developments include timber composite materials, structural connections, localised reinforcement and improved repair and renovation work. These offer substantial benefit, but are reliant upon use of non-traditional timber adhesives, often with thick bond line or 'gap-filling' capabilities.

It is vital that users can specify an adhesive with the correct properties for the intended application but currently no performance classification system is available. Whilst, under laboratory conditions, innovations employing adhesive bonding methods and materials can achieve connections demonstrated to be significantly more efficient than those used at present, these are specific to controlled conditions and a single adhesive brand formulation. However, in industry there remain perceptual barriers coupled with a lack of practical information that has prevented the technology from realising its considerable potential.

A provisional basis for the formulation of a performance-based classification of structural timber adhesives is proposed in this paper, drafted in the context of the Eurocode 5 structural design approach. This includes review of CEN mandates on structural adhesives, polymeric sealant performance classification standards and existing structural timber adhesive guidance. Recommendations are also made concerning the development of the proposed classification basis.

Conclusions
Conclusions drawn from the review work and other research to date:
– CEN mandates M127 and M128 provide a useful starting point in combination with EN 301 and prEN 1504-4 for development of a performance classification methodology for adhesive systems for use in timber structures.
– The performance classification of polymeric sealant materials for construction purposes presented in ISO 11 600 demonstrates a model for development of a broadly similar performance classification methodology for adhesive systems for use in timber structures

– Through identification of applications, it is apparent that there are three types of adhesive applications, (thin bond, gap filling and adhesive based mortars/grouts). These can be linked to actual applications, by way of consideration of the substrate materials. The choice of adhesive is a function of the substrates and the process by which the adhesive is incorporated in the final product, component or assembly.
– The key factors to be considered in order to establish a link between adhesive performance and timber design codes are duration of load effects and service class effects, which act in combination to define the design behaviour model for both serviceability and ultimate limit states.
– A limited range of experimental evidence for this is already available, with certain confirmatory of practical importance, and with certain adhesive classes.

35-18-1 C Bengtsson, B Källander
Creep Testing Wood Adhesives for Structural Use

Introduction and background
Lack of approval procedures for adhesives is hampering, the development of wood products. There is an urgent need for fast and reliable approval procedures for new wood adhesive types, new gluing processes and new glued wood based products. SP is conducting research on test procedures to determine creep properties of wood adhesives for structural purposes. The research has been aimed at developing fast and reliable methods for approval of structural adhesives. The work is carried out within the framework of CEN / TC193) / SCI / WG4 with financial support from the Swedish Wood Association.

Existing test methods for adhesives for structural use have been developed for aminoplastic and phenolic adhesives such as Phenol Resorcinol Formaldehyde (PRF) and Urea Formaldehyde (UF). These adhesives show very little or no creep and hence no test procedures for creep deformation or creep rupture testing of structural wood adhesives have been established. New adhesives such as PolyUrethane (PU) and Emulsified Polymer Isocyanates (EPI) show certain creep tendencies. In order to approve such adhesives, the amount of creep must be determined and related to demands of the finished products.

As for all accelerated test methods, it is crucial that the developed test methods reflect the expected failure modes in the climates that the glued structure will meet in practice. An important aspect regarding creep is the glass transition temperature of the adhesive. An accelerated test at high temperature and moisture could result in failures that never would occur at lower temperatures. There is also the question of whether a test for wood, adhesive or glulines is developed. The combined effects of temperature and moisture on the adhesive as well as the wood properties will limit the possibilities to accelerate creep tests if failure modes are to maintain unaltered.

A serious difficulty regarding accelerated creep test methods for adhesives is the question on how to set the requirements for the tests. Without long term experience of the actual adhesive types in real practice, we need to establish initial requirements based on theoretical assumptions. SP suggests that such initial requirements should be based on the load levels, climates and time spans set in Eurocode 5.

Although the developed test methods should be as fast and cheap as possible in order to simplify the introduction of new products, it is crucial that the developed test methods produce safe and reliable results. One important aspect is then that the limited experience of the test methods and the lack of established requirements make it important that the test methods produce results that can be re-evaluated when new experience is gathered.

Conclusions and recommendations
The results have shown that both methods European 3535 and European 4680 can differentiate between adhesives with different creep properties. Possibly both methods can in the future be used in approval of PU adhesives for structural purposes.

The tests with European EN4680 show large variations in results of the same 1 component PU
adhesive type, both between individual test samples as well as between different batches. Similar variations are likely to influence also results of the European EN 3535 test method. The effects of such variations on the testing and approval procedure need to be evaluated.

Conclusions regarding the European EN 3535 method
The European RN 3535 method gives a fail / pass test result within an eight weeks period after production of test samples. The test result depends on time to failure for the weakest of 72 glulines surfaces. The complicated sample design and non centred load application makes the method

sensitive to variations in sample production and orientation of the coil springs.

Since climatic load, mechanical load and test requirements are closely linked in the test procedure, it is difficult to re-evaluate earlier tests results as new knowledge is made available.

As of today no tests have been made to determine proper requirement levels for the European EN 3535 method. Neither has it been possible to calculate suitable requirement levels based on Eurocode 5 due to the complexity of the test sample, the not centred load application and the mixed climates in the test. This has had the effect that the suggested requirements for European EN 3535 lack both experimental and theoretical foundation and would, if implemented, lead to the disqualification of already established adhesive systems.

Additional comparative tests between the European EN 3535 method and alternative long-term duration of load tests are needed in order to establish proper requirements.

Conclusions regarding the European 4680 method

The European 4680 method follows established principles for determination of duration of load and creep factors. The European 4680 method predicts the shear stress level corresponding to 10000 h time to failure for samples tested at constant climate 50 °C / 75 % RH. In addition to this, the method determines that the samples show acceptable time to failure at 80 °C and 3 N/mm^2 shear stress level.

The method separates the effects of climate, shear stress level and time. The European 4680 method produces numerical data that can be used to pass or fail an adhesive as well as determine and compare properties of different adhesives. The data provided by the European 4680 method can be re-evaluated when new knowledge is made available. The suggested test procedure and the requirements for the European 4680 method are supported by Eurocode 5.

The test results indicate that the European 4680 method can measure creep properties of adhesives. The method has shown a large variation in times to failure between samples tested at the same shear stress level. This can be influenced by variations in gluline properties, but can also be influenced by the sensitivity of the method to irregularities in specimen geometry and application of the load from the coil spring. The test procedure could be improved if the load would be introduced to the centre of the glu-line.

The method can as it is designed today be implemented as a European test method. Further tests incorporating comparative tests with the European 4680 method and long term duration of load tests of full size structural members could later be used to adjust requirements levels.

38-18-1 B Yeh, B Herzog, T G Williamson
Adhesive performance at elevated temperatures for engineered wood products

Abstract

Engineered wood products, such as I-joists and laminated veneer lumber (LVL), have traditionally been manufactured with phenolic-based adhesives in North America. In the last few years, however, an unprecedented number of non-phenolic-based adhesive systems, such as polyurethane-based and isocyanate-based adhesives, have been introduced to the engineered wood products industry. While these relatively new adhesives have demonstrated their compliance with most international adhesive standards, concerns have been raised by the fire services and others on the performance of these adhesives at an elevated temperature, such as just below the ignition temperature of wood.

It is recognized that full-scale fire assembly tests, such as ASTM E119, CANIULC 5101, or ISO 834, address the fire performance of wood assemblies. However, it is arguable that the adhesive, as a critical component of glued engineered wood products, needs to be evaluated to ensure that the adhesive will not degrade below the wood substrates being adhered when exposed to extreme heat prior to wood ignition. Unfortunately, no international adhesive standards exist in testing the adhesive to such an elevated temperature.

A task committee, as chaired by the first author, was formed in June 2004 by the engineered products industry in North America to address this issue with input from key adhesive suppliers to the industry. Through coordinated efforts, an industry standard was developed and adopted by the engineered wood products industry in March 2005. Test data suggests that this standard can be used to screen out those adhesives that significantly lose adhesive bond strength at the temperature near the wood ignition temperature. This standard has been submitted to the International Code Council (ICC), the U.S. code evaluation agency, and the ASTM Commit-

tee on Adhesives (D14.30) for adoption. This paper presents the background infomiation and test data used to develop the industry standard.

Conclusions

An industry standard developed by the engineered wood products industry in North America provides an evaluation method on adhesive bond performance at an elevated temperature near wood ignition. The test method appears to be capable of segregating adhesive performance at an elevated temperature if the adhesive bond is exposed to a temperature above 204 ºC (400 ºF) for an extended period of time. However, while this standard is being adopted by a variety of product and test standards, concerns on the severity of the specified temperature and exposure duration remain to be addressed.

39-18-1 V Rajcic, A Bjelanovic, M Rak
Comparison of the pull–out strength of steel bars glued in glulam elements obtained experimentally and numerically

Introduction

The paper presents the comparison of results obtained experimentally and numerically as well as the efficiency of trained Neural Network in prediction of the strength capacity of the bolted joints made with bars of cold-drawn smooth steel and threaded steel bars glued-in glulam elements. Joint specimens were undergone to destructive one-side pull-out test. The experiment was the starting point of the whole process with purpose of determining the tensile strength of joint. The whole research work and comparison of the experimentally obtained results have been made of several different levels and goals. The first one focused on fracture behaviour of joints made of cold-drawn smooth and threaded bars using the same adhesive layer, EPOCON 88 KGK. The second comparison level was focused on fracture behaviour of joints made of threaded bars while different species of bond layer was used SIKADUR 31 RAPID and EPOCON 88. The third level of research based on comparison of experimental and numerical results for joints bolted with threaded bars only where the results of 3D parametrically prepared models undergone to linear YE analysis in COSMOS/M program. Satisfactory correlation between experimentally obtained results and those that numerically produced was an ultimate issue that been intended to achieve.

Results of FE linear analysis obtained on 3D numerical models used for generating of NN database. The decisive thesis we try to verify was: if we could find the validity of relationship between results obtained experimentally and numerically, then we can use only numerical model to continue researching and make it extensively. We can use then only numerical FE model to explore an influence on joint strength capacity caused by variation of diameters, load angle and anchorage depths of threaded bars, as well.

We also try to make a move over and use results of the FE analysis to generate Neural Network databases. Why Neural Network? The most important fact is that inputs and outputs of FE analysis were recognisable for generated Neural Networks that we trained on them. The whole set of data inputs and outputs have been selected to represent a brief geometrical description of the joint as well as its state of load bearing capacity. The successful completion of an attempt have been achieved when trained NN models produced results of their own, based on, until then, unseen inputs. The interesting thing is correlation of the axial strength obtained by experiments, Neural network prediction strength and design rules suggested for characteristic axial capacity in tension given by Eurocode 5-Part 2-Appendix A.

Conclusion remark

8. Although cold-drawn smooth bars are also declared for possible use in glued-in bolt connections, this paper with its results emphasize that we must be very careful with Adhesion between smooth steel bars and glue is questionable. There is also possibility loosing strength due to corrosion of the steel bar. As it could be seen from the experimen smooth steel bars just slips trough glue. Ultimate axial force (see Fig. 10) is significa smaller than characteristic axial capacity in tension suggested by Riberholt (1988):
   $R_{ax,k} = f_{w,s}\rho_k d\sqrt{l_g}$
   where $f_{w,s}$ is strength parameter. For brittle glues such as epoxy the value is 0.52 and about the same or smaller then characteristic axial capacity in tension suggested by Eurocode 5:
   $R_{ax,k} = \pi d_{equ} l_a f_{v,k}$
   It is of course logical to use rebars or threaded bars but somebody less informed must be warned that smooth steel bars are not allowed to be used in glued-in bolts joints. Or their axial tension strength capacity should be reduced.

9. From the diagrams it could be seen that the axial strength is somewhat higher for bolts glued-in perpendicular to grain direction.
10. The glued-in bolts joints should be used with lots of extra care, because during the experiments some of the results were significantly failed regarding axial strength capacity without mistakes in production.
11. Neural network could be very good tool in prediction strength capacity of the glued-in bolts because it can contains much more data which are valuable such as temperature in glue or in wood, moisture in both materials, production method, properties of various kind of glue.

39-18-2 M Frese, H J Blass
The influence of the grading method on the finger joint bending strength of beech

Abstract
During the last meeting in Karlsruhe the authors presented a paper providing background information about the determination of the characteristic bending strength of beech glulam. A design proposal was derived to calculate the characteristic bending strength depending on the characteristic tensile strength of the lamellae and the characteristic finger joint bending strength. It was experimentally and numerically proved that visual strength grading of beech provide for strength class GL36 and mechanical grading for GL48.

The current paper gives now more detailed information about the influence of the strength grading method on the characteristic finger joint bending strength with regard to beech glulam requirements. Therefore 108 bending tests on finger joints manufactured from visually graded beech boards were performed. A further 319 tests on finger joints manufactured from mechanically graded beech boards were carried out. All the bending tests were conducted flatways according to EN 408 with a span of 15 times the height. The test results confirm a characteristic finger joint bending strength of 56 N/mm^2 in case of visual and 70 N/mm^2 in case of mechanical grading.

Conclusions
On the basis of Fig. 1 and the experimental data the following conclusions can be drawn:

- In DIN 1052 visual strength grading of beech in class LS 10 and LS13 corresponds to the strength classes D35 and D40, respectively. The characteristic finger joint bending strength of such visually graded boards amounts to nearly 56 N/mm^2. Hence it is possible to establish GL28 and GL32 with standard visual strength grading methods.
- Assuming that visual strength grading of beech boards being free from knots corresponds to D50 and that finger joint manufacture provides a characteristic bending strength of 58 N/mm^2 it is even possible to produce GL36.
- Mechanical strength grading of beech boards having a dynamic MOE determined from longitudinal vibrations of at least 15000 N/mm^2 and additional demands on knots are precondition for strength classes equal to or greater than D60. For those boards a characteristic finger joint bending strength amounts to nearly 70 N/mm^2. Under optimised production conditions in terms of finger joint manufacture higher values are even possible. Hence strength classes up to GL52 are imaginable.

43-7-1 T Tannert, T Vallée, F Lam
Probabilistic Capacity Prediction of Timber Joints under Brittle Failure Modes

Abstract
To predict the capacity of timber joints is difficult due to the anisotropic and brittle nature of the material, the complex stress distribution as well as the uncertainties regarding the associated material resistance. This paper describes a probabilistic method for the capacity prediction of timber joints under brittle failure modes. The method considers the statistical variation and the size effect in the strength of timber using a Weibull statistical function and presents an explanation for the increased resistance of local zones subjected to stress peaks. The method was applied to three different types of joints: (i) adhesively bonded double lap joints; (ii) CNC fabricated rounded dovetail wood-to-wood joints; and (iii) linear friction welded joints. Experimental and numerical investigations were carried out to to determine the failure modes and capacities of these joints. The statistical distributions of the material strengths were obtained with small scale specimen tests: the problem of using different test configurations is discussed. Finite element analyses were applied to determine the stress distribution and provide input data for the capacity prediction. Experimental

and numerical results were found to be in good agreement for all three joint types. This paper furthermore shows how the probabilistic method has to be formulated in the framework of current standards to predict characteristic values of joint capacities. The proposed method has immediate application for the design improvement of the investigated joints and can be extended to other joints; e.g. dowel type connections.

Discussion and Conclusions

The capacity prediction of timber joints is difficult due to the anisotropic and brittle nature of the material, the complex stress distribution as well as the uncertainties regarding the associated material resistance. This paper describes a probabilistic method to predict the capacity of timber joints under brittle failure modes. The method considers the statistical variation and the size effect in the strength of timber using a Weibull statistical function. The design method presents an explanation for the increased resistance of local zones subjected to high stress peaks as it takes into account not only the magnitude of the stress distributions, but also the volume over which they act [26]. The method, besides yielding accurate predictions, has the additional benefit of relying solely on objective geometrical and mechanical parameters, excluding any empirical input.

The method was applied to three different types of joints: (i) adhesively bonded double lap joints; (ii) CNC fabricated rounded dovetail wood-to-wood joints; and (iii) linear friction welded joints. The paper reports on experimental and numerical investigations to determine the failure modes and capacities of these joints.

The statistical distributions of the material strengths were obtained with small scale specimens. The mechanical parameters can be determined using standardized test specimens exhibiting different shapes and material volumes, and subsequently have to be brought into a coherent mechanical form, i.e. volume. An increase of accuracy can be expected by using a less disparate set of tests; using samples that are more comparable in their geometry and volumes, for example a set of off-axis tension tests that allow to formulate the failure criterion more straightforwardly.

The experiments for the adhesively bonded and the welded joints were carries out on high quality almost defect free timber. The authors are aware that this leads to consider an idealized situation, since in practical applications such a selection is unlikely to occur. However, using less strictly selected timber will in first instance only increase the scattering of material strength, without altering the principles behind the dimensioning method subsequently developed.

Finite element analyses were applied to determine the stress distribution and provide input data for the capacity prediction. The subsequent application of the probabilistic strength prediction method proved to be sufficiently accurate, as it has been demonstrated for all investigated joints. The capacity determination of systems, including joints, exhibiting brittle failure has, for a long time, been considered difficult, and often solved using empirical methods. This paper furthermore shows how the probabilistic method has to be formulated in the framework of current standards to predict characteristic values of joint capacity. Since brittle failure tends to be well described by extreme value probability density functions distributions, e.g. Weibull, such statistics also lead to good agreement between experimentally and numerically determined characteristic values of capacities.

This paper offers a new approach by implementing probabilistic concepts in an engineering context. Firstly the method overcomes the difficulties raised by the timber’s inherent brittleness and strength variability; secondly, it offers an alternative to much more complex fracture energy based methods, for which the correct input data is complicated to generate. Its implementation proved to be straightforward, and sufficiently accurate to predict joints capacities of the investigated joints under brittle failure over a large set of parameters. The proposed method has immediate application for the design improvement of the investigated joints and can be extended to other types of timber joints.

43-12-1 S Aicher, G Stapf
Fatigue behaviour of finger jointed lumber

Introduction

Solid lumber is increasingly replaced in engineered timber structures by finger jointed lumber used as studs and beams. There are several reasons for this, being i) the significantly increased yield of higher quality/strength lumber by cutting out the knots and other strength reducing timber defects, ii) a highly increased production flexibility (drying, storage, sizes, delivery) and iii) a considerably higher dimensional stability of sticks of longer length, being a prerequisite of today's wood machining technology. At present, finger jointed lumber is almost exclusively used in constructions designed for static loads. However, there is a demand for finger jointed lumber to be used as well in dynamically loaded constructions, e.g. bridges, wind mills, roller coasters, etc. The limited existing research data on tension fatigue strength of finger joints in small clear wood and in structural sized lumber indicate a fatigue limit reduction factor in a very wide range of about 0,30 to 0,55 vs. static strength. The rather limited knowledge on fatigue loaded finger-jointed lumber contributed to the fact that no difference is made today in Eurocode 5, Part 2, Design of Timber Structures – Bridges, with regard to fatigue of solid lumber without finger joints and with finger joints. As there are no explicit hints on the eventual differences in the fatigue behaviour between both timber materials a design engineer might use them in an alternating manner such as in design of exclusively statically loaded constructions.

In order to get more insight into the fatigue behaviour of finger jointed solid lumber a research project is presently being conducted at the MPA University Stuttgart on fatigue of finger jointed structural sized pine lumber. Hereby, the effects of moisture and preservation treatment are also considered. The paper presents some results of the research project and gives a comparison with the fatigue reduction values k_{fat} of Eurocode 5, Part Further effects of preservation treatment are discussed.

Conclusions

Fatigue bending tests with finger jointed pine lumber were performed with a stress ratio of 0,5. The nominal cross-section was 100 mm x 200 mm. The tests were conducted up to a maximum (limit) stress cycle number of $2 \cdot 10^6$, beyond which unbroken specimens were considered as survivors and tested in ramp load for determination of residual strength.

The fatigue results were approximated for derivation of a median S-N curve by a linear regression including the ramp load results. The obtained linear median Wöhler curve has a slightly less steep slope as the equations of Eurocode 5 – Part 2 for bending and shear loading. Whether this holds true for all and especially negative R-ratios is not evident. It has also to be checked whether the more unfavourable finger joint configuration with “lying fingers” elicits similar fatigue results. According to the all of the test results (ramp load, fatigue, residual strength data) the median fatigue bending strength limit (for $R \approx 0,5$) seems to be in the range of 35 % to 45 % of the ramp load strength.

The limited number of results in each of the test series B1 and B2 do not enable a reliable answer as to whether the specific preservative treatment has an impact on the fatigue behaviour or not. On the other hand, the data do not give any indication of a significant difference in the fatigue of treated and untreated finger jointed pine lumber. This statement however, is necessarily related to the very dry moisture state of about 10 % of both test series.

43-18-1 B Walford
Comparison of API, RF and MUF adhesives using a draft Australian/New Zealand Standard

Abstract

Tests were done using MGP15 slash pine (i.e. non-standard) with an aqueous polymeric isocyanate (API), a resorcinol (RF) and a melamine-urea (MUF) adhesive to AS/NZS4364:2007 (Int), for hydrolytical stability, shear block strength, delamination and creep. The API was found to have a durability intermediate between that of the RF and the MUF adhesives. The draft standard needs to be improved by deletion of the hydrolytic test that involves water bath and oven treatment of specimens. The delamination test could also be deleted as it gives information that is essentially the same as the boil/freeze/dry shear block test. The creep test needs modification to prevent specimens buckling.

Conclusions

The hydrolytical test involving ovens and water baths should not be an option in the draft standard because it is open to different interpretations, is very time-consuming, and gives unrealistic answers.

- The shear block and delamination tests are quick and simple to do and give answers that are essentially equivalent which suggests that one of the tests could be deleted.
- The creep test needs modification to prevent specimens buckling.
- The API adhesive showed durability intermediate between that of RF and MUF.
- The durability of both the API and the MUF adhesives were significantly reduced in the presence of liquid water.
- The API and MUF may have performed better at high temperatures if they were subjected to water vapour rather than liquid water. This is a matter for further research

4.6 GLUED-IN RODS

ESSAY 4.3 H J Larsen
Glued-in rods

The sad story of glued-in bolts in Eurocode 5
Right from the beginning there was no doubt that glued-in bolt should be included in the Eurocode 5, and clauses were included from 1986 in its predecessor the CIB/W18 Timber Design Code based on **Paper 19-7-2**.

There were however many conflicting views on the draft, and when the time for publication drew near first for ENV 1995-1-1, then for EN 1995-1-1, it became obvious that it would not be possible to come to an agreement. There were many bits of research and all researchers found that their bit overruled all the other bits.

It was agreed to postpone the topic to the bridge Eurocode EN 1995-2. To get a basis for the drafting funding was obtained from the European Commission. Information about the project called GIROD is given in **Paper 34-7-8**. The project was split into several packages.

One was drafting an agreed proposal for a chapter in the Eurocode 5. Unfortunately the partner responsible and paid for this package never delivered the proposal. There was an amateurish draft, but it was never published or discussed in CIB/W18 or in the responsible drafting group. Just before the last meeting in CEN TC 250/ SC5 responsible for the bridge Eurocode some of the GIROD partners come up with a draft that most felt was acceptable, but again some felt that they themselves could have done it better and that some of their pet ideas were not included. And on the spur of the moment it was decided to give up and leave glued-in rods out.

And there it stands until a new generation takes over.

In the following the rejected proposal is shown and compared to two other proposals: The old one set out in **Paper 19-7-2** and for a time included in the drafts for ENV 1995, and a German one given in the National Application Document (DIN V EN 1995-1-1/NA 1:2004-12) to Eurocode 5.

Proposal in prEN 1995-2 discussed for formal vote

Annex C (informative)
Bonded-in steel rods

C.1 General

(1) The use of bonded-in rods should be limited to structural parts assigned to service classes 1 and 2.

(2) It should be verified that the properties of the adhesive and its bond to steel and wood are reliable during the lifetime of the structure within the temperature and moisture ranges envisaged.

(3) Rods should be threaded or deformed bars.

(4) The shear strength of the adhesive and its bond to steel and timber should be verified by tests.

(5) For service class 2, the values of k_{mod} according to EN 1995-1-1 clause 3.1.3 should be reduced by 20 %.

C.2 Axially loaded rods

C.2.1 General
(1) The load-carrying capacity of connections made with bonded-in axially loaded rods should be verified for the following failure modes:
– failure of the steel rod;
– failure of the adhesive and its bond to steel and timber;
– failure of the timber adjacent to the glue-line;
– failure of the timber member (e.g. pull-out failure of a whole timber block with several bonded-in rods).

(2) The design load-carrying capacity should generally be limited by the strength of the rod.

(3) The expressions given are based either on the outer diameter d of the rod; or when strength of the adhesive is not critical, on an equivalent diameter d_{equ} equal to the smaller of the hole diameter, d_h, and 1,15d.

Note: For threaded rods, the outer diameter is equal to the nominal diameter; for most deformed reinforcing bars used as rods, the outer diameter is about 10 % greater than the nominal diameter.

(4) Minimum spacings and edge and end distances should be taken according to figure C.1.

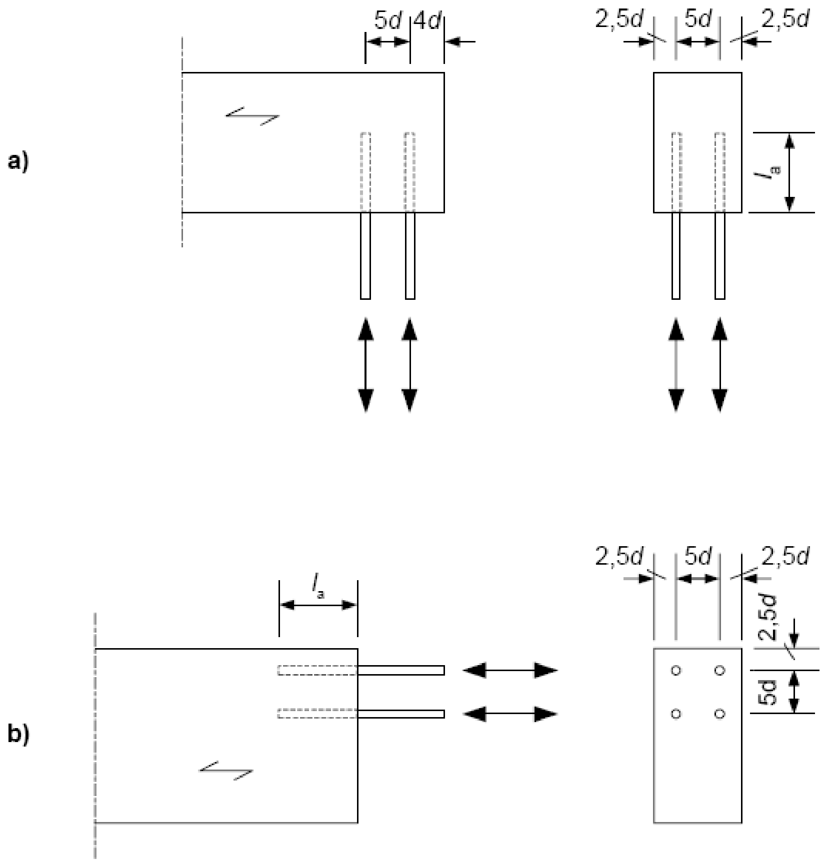


Figure C.1 – Minimum spacings and distances for axially loaded rods loaded
a) perpendicular to the grain
b) parallel to the grain

(5) The minimum anchorage length $l_{a,\min}$ should be taken as:

$$l_{a,\min} = \max\begin{cases} 0,5d^2 \\ 10d \end{cases} \tag{C.1}$$

where:
$l_{a,min}$ is the minimum anchorage length in mm, see figure C.1;
d is the outer diameter of the rod in mm.

C.2.2 Ultimate limit state

C.2.2.1 Failure of individual rod
(1) The characteristic axial load-bearing capacity in tension of the steel rod, $R_{ax,k}$ in N, should be taken as:

$$R_{ax,k} = \min\begin{cases} f_{y,k} A_{ef} & \text{(a)} \\ \pi d_{equ} l_a f_{ax,k} \dfrac{\tanh \omega}{\omega} & \text{(b)} \end{cases} \tag{C.2}$$

where:

$f_{y,k}$ is the characteristic yield strength in N/mm^2;
A_{ef} is the effective cross-sectional area of the rod in mm^2;
d_{equ} is the equivalent rod diameter in mm, see C.2.1(3);
l_a is the anchorage length in mm;
$f_{ax,k} = 5,5$ N/mm^2;

$$\omega = \frac{0,016\, l_a}{\sqrt{d_{equ}}}; \tag{C.3}$$

For rods in compression, the possibility of buckling should be taken into account for design compression stresses greater than 300 N/mm^2.

C.2.2.2 Failure in the timber member
(1) The effective timber failure area, A_{ef}, of a rod inserted in direction parallel to the grain, see figure C.2, should be taken as the smaller of
- an effective width, b_{ef}, of $3d$ on each side of the centre of the rod;
- the area derived from the actual geometry where the distance is smaller than $6d$ or the edge
  distance is smaller than $3d$.

(2) In a group of rods inserted in direction parallel to the grain, the characteristic resistance parallel to the grain of one rod, $R_{ax,k}$, should be taken as:

$$R_{ax,k} = f_{t,0,k} A_{ef} \tag{C.4}$$

where:

$R_{ax,k}$ is the characteristic load-carrying capacity of one rod;
$f_{t,0,k}$ is the characteristic tensile strength of the wood;
A_{ef} is the effective timber failure area.

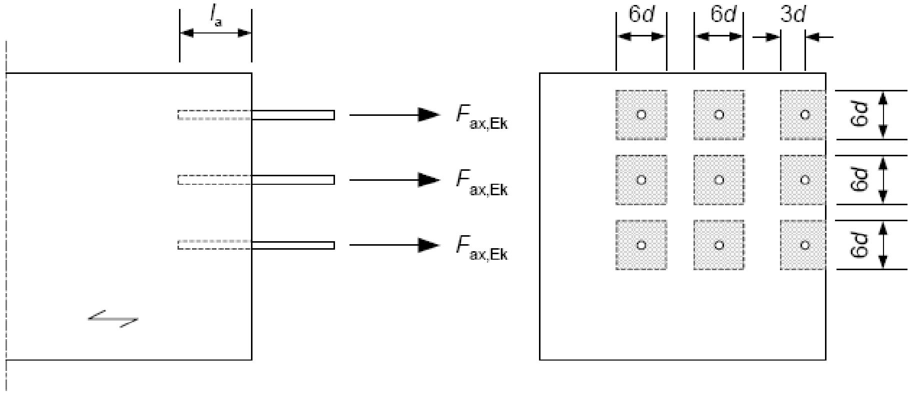


Figure C.2: Effective areas for anchorage forces parallel to the grain with b_{ef} = 6*d*

(3) For rods inserted at an angle to the grain, EN 1995-1-1 clause 8.1.4 applies where h_e is the loaded edge distance to the end of the rod and b is replaced by b_e.

C.2.3 Serviceability limit states

(1) The instantaneous slip modulus, K_{ser}, in N/mm per rod should be taken as

$$K_{ser} = 0,004 d^{1,8} \rho_{mean}^{1,5} \tag{C.3}$$

where:

d is the diameter of the rod, in mm;
ρ_{mean} is the mean density of the wood in kg/m^3.

C.3 Laterally loaded rods

C.3.1 Ultimate limit state

(1) The provisions of EN 1995-1-1 section 8 for laterally loaded dowels apply.

(2) For laterally loaded bonded-in rods inserted parallel to the grain, the embedding strength should be taken as 10 % of the embedding strength perpendicular to the grain.

(3) For bonded-in rods inserted at an angle α to the grain, linear interpolation should be applied.

C.3.2 Serviceability limit states

(1) For rods inserted perpendicular to the grain, the slip modulus K_{ser} in N/mm per rod should be taken as

$$K_{ser} = 0,04 d \rho_{mean}^{1,5} \tag{C.4}$$

where:

d is the effective rod diameter, in mm;
ρ_{mean} is the mean density of the wood in kg/m^3.

Note: For threaded rods the effective diameter of the rod corresponds to about 90 % of the outer diameter; for deformed reinforcing bars to the nominal diameter.

(2) For rods inserted parallel to the grain, K_{ser} should correspondingly be taken as

$$K_{ser} = 0,08 d \rho_{mean}^{1,5} \tag{C.5}$$

(3) For bonded-in rods inserted at an angle α to the grain, linear interpolation should be applied.

C.4 Combined laterally and axially loaded rods

(1) For combined laterally and axially loaded bonded-in rods, the following condition should be satisfied:

$$\left(\frac{F_{ax,d}}{R_{ax,d}}\right)^2 + \left(\frac{F_{la,d}}{R_{la,d}}\right)^2 \leq 1 \qquad \text{(C.6)}$$

where:
$F_{ax,d}$ is the axial design load;
$F_{la,d}$ is the lateral design load;
$R_{ax,d}$ is the axial design load-carrying capacity;
$R_{la,d}$ is the lateral design load-carrying capacity.

C.5 Execution

(1) The surfaces of the holes should be clean cut.

(2) With several rods in a group to be tightened, the tightening should be uniform.

(3) It should be insured that the hole is completely filled with adhesive.

(4) At the time of gluing the rods, the moisture content of the timber should not be more than 15 %.

Proposal given in Paper 19-7-2.

The characteristic load-carrying should be taken as

$$F_{ax,k} = \min \begin{cases} k_{thread} f_y A_{ef} \\ f_{ax,k} d_{equ} \sqrt{l_a} \end{cases}$$

where
$f_{ax,k}$ = 167 N/mm^{1,5} for polyurethane adhesives and $f_{ax,k}$ = 133 N/mm^{1,5} for epoxy and resorcinol adhesives.

Proposal given in The German National Application Document to EN 1995-1-1

The load-carrying capacity should be taken as

$$F_{ax,k} = \min \begin{cases} f_{y,k} A_{ef} \\ \pi d l_a f_{a,k} \end{cases}$$

where

$$f_{ax,k} = \begin{cases} 4 & & l_a \leq 250 \text{ mm} \\ 5,25 - 0,005 l_a & \text{for} & 250 \text{ mm} < l_a \leq 500 \text{ mm} \\ 3,5 - 0,0015 l_a & & 500 \text{ mm} < l_a \leq 1000 \text{ mm} \end{cases}$$

Example

As an example the load-carrying capacity is calculated for a threaded rod M 20 with $f_{y,k}$ = 240 N/mm^2. A_{ef} = 245 mm^2.

Proposal in EN 1995-2 (bridges)

With a hole diameter of 20 mm, the equivalent diameter is

$$d_{equ} = \min \begin{Bmatrix} 22 \\ 1,15d = 23 \end{Bmatrix} = 22 \text{ mm}$$

The minimum anchorage length is

$$l_{a,\min} = \max \begin{Bmatrix} 0,5d^2 = 200 \\ 10d = 200 \end{Bmatrix} = 200 \text{ mm}$$

With

$$\omega = \frac{0,016 l_a}{\sqrt{22}} = 3,41 \cdot l_a \cdot 10^{-3}$$

the characteristic axial load-carrying capacity is

$$R_{ax,k} = \min \begin{cases} 240 \cdot 245 \cdot 10^{-6} = 58,8 \\ \pi \cdot 22 \cdot l_a \cdot 5,5 \cdot \dfrac{\tanh \omega}{\omega} = 380 \dfrac{\tanh \omega}{\omega} \end{cases}$$

The axial load-carrying capacity depending on $l_a \geq 200\,\text{mm}$ is shown in Table 1 and Figure 1.

Proposal in Paper 19-7-2

For epoxy adhesives

$$F_{ax,adh,k} = f_{ax,k} d_{equ} \sqrt{l_a} = 133 d_{equ} \sqrt{l_a}$$

The load-carrying capacity is shown in Table 1 and Figure 1.

German Proposal

$$F_{ax,shear,k} = \pi d l_a f_{a,k}$$

where

$$f_{ax,k} = \begin{cases} 4 & & l_a \leq 250\text{ mm} \\ 5{,}25 - 0{,}005 l_a & \text{for} & 250\text{ mm} < l_a \leq 500\text{ mm} \\ 3{,}5 - 0{,}0015 l_a & & 500\text{ mm} < l_a \leq 1000\text{ mm} \end{cases}$$

The load-carrying capacity for is also shown in Table 1 and Figure 1.

Table 1. Characteristic load-carrying capacity for epoxy adhesives

	Eurocode 5-2 (bridges)				Paper **19-7-2**	German NAD
l_a mm	$F_{ax,steel.k}$ kN	ω	$F_{ax,shear.k}$ kN	$F_{ax.k}$ kN	$F_{ax.k}$ kN	$F_{ax.k}$ kN
160	52,92	0,55			37,1	40,2
180	52,92	0,61			39,3	45,2
200	52,92	0,68	66,1	52,9	41,5	50,3
220	52,92	0,75	70,8	52,9	43,5	53,3
240	52,92	0,82	75,1	52,9	45,4	58,8
260	52,92	0,89	79,1	52,9	47,3	58,8
280	52,92	0,96	82,7	52,9	49,1	58,8
300	52,92	1,02	85,9	52,9	50,8	58,8
320	52,92	1,09	88,9	52,9	52,5	58,8
340	52,92	1,16	91,5	52,9	52,9	58,8
360	52,92	1,23	93,8	52,9	52,9	58,8
380	52,92	1,30	95,9	52,9	52,9	58,8
400	52,92	1,36	97,8	52,9	52,9	58,8

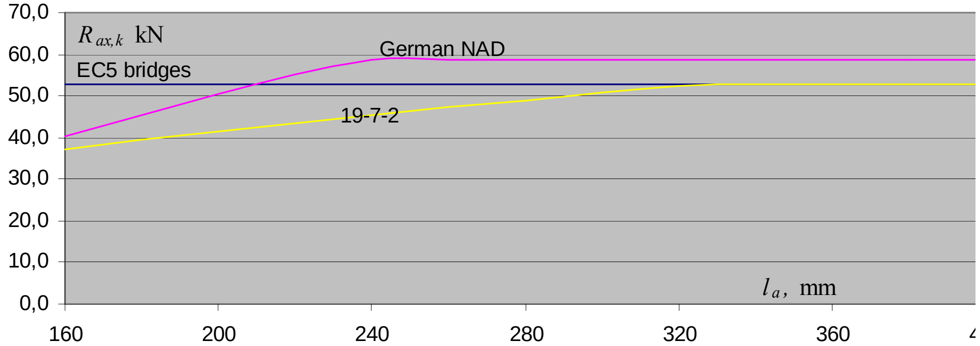


Figure 1. Load-carrying capacities.

If $k_{thread} = 0{,}9$ is used also in the German proposal it gives almost the same result as Eurocode 5 Bridges for $l_a > 200$ mm.

The German proposal differs from the other two by not taking into account the strength reduction given in Eurocode 3 for threaded rods.

The proposal in EN 1995-2 differs from the other two by giving a minimum anchorage length in addition to the general requirement that failure should be due to steel yielding and not adhesion failure.

19-7-2 H Riberholt
Glued bolts in glulam

Summary and conclusions
An economic and convenient production method for glued-in bolt connections has been developed. The bolts were placed in oversized holes and glue was then injected. For practical reasons only threaded rods were used. The axial capacity of these is not very sensitive to minor defects in the bonding to the steel.

The tests show that the bolt connection can be used even for large changes in the moisture content of the wood. The steel surface is to a certain degree protected against corrosion by the employed polyurethane and epoxy glues. But under severe corrosive conditions zinc coatings or similar should be used.

The axial and lateral load-carrying capacity of a bolt connection has been found experimentally. The capacities were similar to those reported in the literature, but here essentially smaller distances to the wood edges and in between the bolts were used. Further, the axial capacity was found to be higher for the more flexible polyurethane glue.

For axial loaded bolt connections long-term tests were carried out in which the moisture content of the wood was changed. It appears that for out door service the-long term capacity is approximately half the short-term capacity of dry specimens.

Further for some selected types of connections with glued-in bolts, tests were carried out in order to verify the proposed calculation methods and theories of the static behaviour. The following were investigated:

- Moment stiff column-foundation joint
- Trimmer joint
- Moment stiff frame corner
- Splitting of laterally loaded glulam beams

In general it can be, stated that glued-in bolt connections can be designed with a capacity comparable to that of a glulam element. As an example the moment capacity of the tested moment stiff column-to-foundation joint was found to be approximately 75 % of that of the glulam column. And the connection could easily be designed stronger.

The tests with a frame corner revealed that, if several glued bolts are intended to carry the load together, then it is essential to design the connection so that the forces can be distributed evenly. Otherwise, the glued length employed has to be of a size that the bolt yields before being pulled out. Because in this way an initial uneven force distribution can be equalized before failure occurs due to ductility of the bolt connection.

Object
The object of the project was:

A. Development of a reliable production method.
B. Development of corrosion resistant glued bolt connections for outdoor service.
C. Determination of the axial strength and the lateral load-carrying capacity for glued bolt connections. Further determination of minimum distances for obtaining the load-carrying capacities.
D. Accomplishment of some full scale tests to elucidate suitable applications and as background for dimensioning methods.

As starting basis the project had the report: “H Riberholt, Glued bolts in glulam, Department of Structural Engineering, Technical University of Denmark, 1977” which deals with threaded rod connections glued with resorcinol glue.

26-7-4 J Kangas
Design of joints based on V-shape glued-in rods

Abstract
In this study a method is presented how to design the capacity of moment resisting, joints of glued laminated timber structures based on the properties of in V- shape glued-in rods. Design rules for anchorage capacity of the rods and capacity of timber in the joint area are given. Design rules are based on an experimental research, which is briefly introduced.

The, technology has originally been developed in TSNIISK in Moscow. It was introduced in CIB-W18 in **Paper 22-7-11**. Research work began already in 1975. It has been almost unknown for the rest of the world mainly because the publications were in Russian.

The method is based on ribbed steel rods, which have been glued at skew angles into the glulam. The rods take effectively the forces in their direction up to the tensile capacity of the steel. When the rods have been welded on to steel plates, the forces can be carried forward in the same manner as known in steel structures.

This article presents some results of the experimental research carried out in Technical Research Centre of Finland (VTT) during the years 1991-3. The goal of this research was to deepen and to enlarge the basis for design rules and instructions for production.

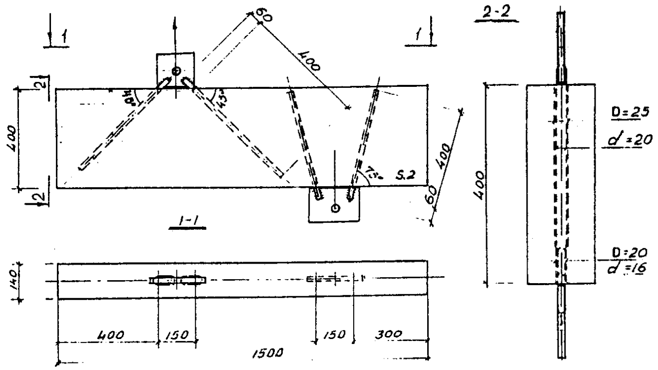


Fig. 3. Test specimen of V-anchor joints loaded in tension.

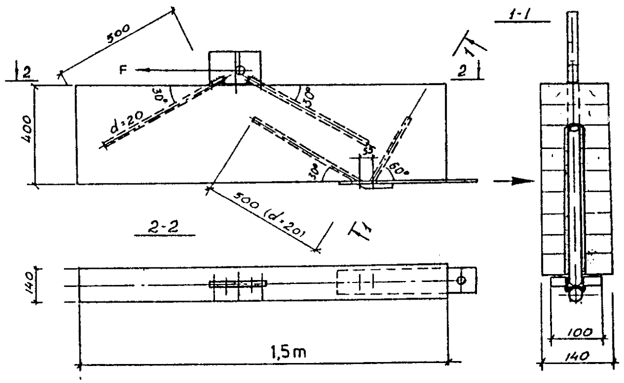


Fig. 4. Test specimen of V-anchor joints loaded in shear.

28-7-9 C J Johansson, E Serano, P-J Gustafsson, B Enquist
Axial strength of glued-in bolts. Calculation model based on non-linear fracture mechanics – a preliminary study

Abstract

Axially loaded bolts have been studied theoretically and experimentally. A theoretical model developed for lap joints in pure shear has been used. The model, which consists of an analytical expression, is based on a combination of Volkersen theory and nonlinear fracture mechanics. Properties of the bond line, such as the fracture energy, the shear strength and the shape of the $\tau - \delta$ curve are included in the model. The moduli of elasticity of the adherents are also considered. The bond line properties have been determined experimentally on very short threaded bolts, with a bond line length of only 5 mm. Both relatively ductile and brittle adhesives have been studied. Bolts have also been analysed by means of a finite element method. A good correspondence with the analytical model was obtained for different values of the fracture energy and the shear strength. An important prerequisite in the model is that the stresses perpendicular to the bond line are negligible. The finite element analysis showed that such stresses only occur locally at the ends of the bond line and that they are relatively small. The model allows simple analysis of how different factors affect the axial strength of glued-in bolts. It can for example be shown, that for short bolts the bond line shear strength is the decisive factor. For long bolts the fracture energy is the most important. Moreover, due to its simplicity and other characteristics, the model may be of great interest for future codes of practice.

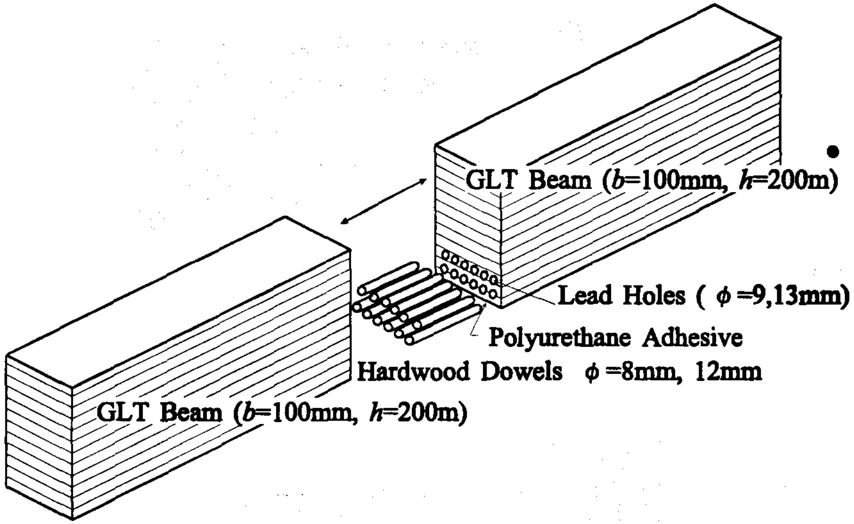
Conclusions

A simple model based on Volkersen theory and non-linear fracture mechanics can be used to predict the pull out strength of bolts glued in parallel to the fibre direction.

The shear strength and the fracture energy required in the model can be obtained by curve fitting to pull out strength data from bolts with different lengths or possibly also by testing small glued-in bolt specimens and recording the complete $\tau - \delta$ curve.

30-7-1 K Komatsu, A Koizumi, T Sasaki, J L Jensen, Y Iijima
Flexural behavior of GLT beams end-jointed by glued-in hardwood dowels

Abstract
Recently in Japan, hardwood dowel has been getting a new reputation as an alternative jointing device for engineered timber joints because it can be harmonized with timber structural members more gently than such jointing system as steel drift-pins or bolts with steel gusset plates. As the first step of utilizing this natural jointing device for engineered timber structures, a series of pull-out shear strength test has been done on single dowel joints glued with polyurethane adhesive.



End-jointed GLT beams using adhesive and hardwood dowels.

This paper follows after the first step mentioned above, and gives preliminary results on the theoretical prediction of flexural behaviour of GLT beams which were end jointed using several hardwood dowels with polyurethane adhesive as shown in the figure.

32-7-12 H J Blass, B Laskewitz
Effect of spacing and edge distance on the axial strength of glued-in rods

Introduction
Glued-in rods have been used for several years in timber structures to transfer high forces from one structural element to another. In addition they are used as a reinforcement perpendicular to the grain in timber members. However, generally accepted design rules for glued-in rods are still missing.

Since 1998, the European Union supports the research project GIROD, in which design rules for glued-in rods will be drafted for inclusion in Eurocode 5. This research project is divided into several working packages carried out by different European partners. The University of Karlsruhe is responsible for working package 3 where the minimum spacing and the minimum end and edge distances of glued-in rods are determined.

This paper presents test results with axially loaded rods glued in parallel to the grain in oversized holes in glued laminated timber members and proposes minimum rod-to-edge distances and rod spacing for timber members.

Conclusion
When using glued-in rods in timber structures it is suggested to use spacings of 5 times and edge distances of 2.5 times the rod diameter. Otherwise, a decrease in load-carrying capacity should be taken into account. It is recommended not to use edge distances below two times the rod diameter because of inevitable inaccuracies when drilling the holes for glued-in rods.

32-7-13 C J Mettem, R J Bainbridge, K Harvey, M P Ansell, J G Broughton, A R Hutchinson
Evaluation of material combinations for bonded-in rods to achieve improved timber connections

Abstract
Well designed and executed adhesive bonded structural connections can be extremely efficient and may possess many desirable attributes in terms of manufacture, performance, aesthetics and cost. The use of bonded-in rods is an important feature of many of the methods for achieving connections using adhesives.

Bonded-in rods are now being used in several European countries, but the performance requirements and the design rules differ, which is a seri-

ous obstacle to trade. A standard on design of timber (Eurocode 5 -Design of timber structures - Part 2: Bridges) has been drafted by CEN TC250/SC5 and contains an informative Annex on the subject. There is also a 3-year European project in progress at the time of writing titled "GIROD - Glued In Rods For Timber Structures", and performed through collaborative activities between partners from UK, Germany and Sweden.

This paper presents findings from studies initiated prior to the GIROD project, which focus specifically upon the influence of geometric and material variations upon strengths and modes of failure. Innovative routes explored include thick glue lines, use of various epoxy and acrylic adhesives, surface prepared ferroreinforcement rods and FRP composite rods. These have been based upon experimentation through physical test and numerical modelling, within the context of developing information of value in aiming to derive and validate limit states design procedures.

Conclusions

FRP materials appear to have a useful role for execution of bonded-in rod type connections. However they require different design considerations and should not necessarily be considered as direct substitutes for steel rods.

Different adhesive types have different behaviours, but there are also noticeable differences between brands within a generic type. Therefore specific data should be employed relevant to a resin system deployed in accordance with manufacturer guidance.

Extremely thick bond lines have been investigated in this research, and pull-out strengths increased linearly with bond line thickness, consistent with the increase in bonded area of timber at large hole diameters. FEA results clearly exhibit a fall in peak shear and peel stress at the adhesive/timber interface with increasing bond line thickness.

These studies have highlighted some of the potential problems with the current design basis centred upon yield capacity of rods when employing a wider range of materials. Employment of high performance adhesives in thick bond lines together with a re-assessment of rod design capacity could provide design benefit utilising the fuller range of available materials.

The geometric configuration will combine with the material properties of rod, adhesive and host timber to define the failure mechanism. The implications for design vary. As far as ULS behaviour is concerned, the interaction of material combinations and implications of selection reflect the chosen basis of design and the way in which the material behaviour to failure is tackled with a view to establishing satisfactory reliability in the end solution.

Lightness and ease of bondability (especially through peel ply techniques) mean that FRP solutions may provide a valuable alternative in new-build and in renovation and repair work. In either case, the combination of bonding technology and appreciation of design concepts in a fundamentally different material may pose challenges to the full implementation across construction.

Process issues are of importance, and design needs to be attuned to specific materials of known quality, reliance upon workmanship and low visual inspection opportunity. Therefore the communication, monitoring, feedback and overall skill levels are paramount in achieving low defect levels and suggest that these systems are better suited to pre-fabrication in preference to site manufacture solutions. There is however clearly scope for partial prefabrication with site processes focussed upon mechanical connection of adhesive bonded rods.

There is a clear need for quality control type test configurations to be developed, especially bearing in mind simple, easily operable procedures suited to SME manufacturers who dominate timber engineering operations throughout Europe.

33-7-10 J Kangas
Capacity, fire resistance and gluing pattern of the rods in V-connections

Summary

A summary is reported of a large research work on joints of glulam structures based on ribbed steel rods, which have been glued-In at skew angles into the glulam. This paper presents the design of these connections and test results of their loading as well as test results of fire resistance tests under loading. The design method is based on large test series. A model for the design of the moment bearing connection has been developed. To the vector sum of the capacities of the rods forming V-shape also the calculated dowel capacity of the rods with steep direction angle can be added.

Fire resistance tests were carried out with beams under constant bending load with V-connection in the middle span. Fire protection and design is simple. Just 20 mm layer of timber or rock wool is needed to cover the steel rods for each 30 min of fire resistance. Rod distances and spacing

have been varied. Minimum values of those in V-connections are proposed.

Concluding Remarks

Design method is proposed for the V-connections, in which the steel rods are glued at skew angles into the glulam. It is based on large test series. The V-connection is not vulnerable to splitting of wood in the anchorage area. Due to limited variation the anchorage strength of the rods can have a high design value in all grain direction bigger than 15°. The calculated dowel capacity of the rods with a steep direction angle also can be added to the vector sum of the capacities of the rods forming V-shape.

V-joint is quite rigid. However, like mechanical joints it has ductile properties due to yielding of the longer rod. The capacities of V-joints can be summed without reduction on the condition that the anchorage length allows yielding of the rods. The connection can carry bending moments up to the capacity of the glulam and is not vulnerable to splitting of the wood in the joint area. The tensile capacity of timber shall be checked for the tensile force in the connection. The tensile force perpendicular to grain shall be designed separately

33-7-11 R J Bainbridge, K Harvey, C J Mettem, M P Ansell
Fatigue performance of bonded-in rods in glulam, using three adhesive types

Abstract

Bonded-in rods are an economical and architecturally and industrially attractive means of forming connections within a timber structure, and of providing local reinforcement to critical zones of timber members. They also provide an important technology for the repair and upgrading of historically important timber structures that exist throughout Europe.

Structures are subject to variable actions throughout their design lives, which normally result in the repeated development of stresses below the ultimate strength of the material. There is however a theoretical potential for failure to be induced even at low stress levels, due to fatigue.

This paper presents the findings of experimental investigations in relation to the use of commercial adhesives to bond threaded steel rods into oversized holes in order to achieve structural timber connections. The work is part of the European Union supported GIROD project, which aims to develop design rules for inclusion in Eurocode 5.

Three types of adhesive are considered – an epoxy, polyurethane and a filled phenol resorcinol formaldehyde, in tests whereby threaded steel rods were axially loaded in a variable tension cycle whilst bonded parallel to the grain of a host glulam member. Fatigue lives are examined for two geometries of test specimen and results from different failure modes are compared to the theoretical performance of the component materials.
Four potential failure modes were identified (timber failure, cohesive failure at timber/adhesive interface, bond line failure and steel rod failure), all of which were observed within a single geometry of specimen. It is demonstrated that the adhesive type has a clear influence upon both the fatigue life and the likely failure mode.

Conclusions

A revised basis for design of bonded-in rods for structural timber connections is under preparation through the GIROD project. This draws heavily upon both existing knowledge and experimental investigation. The fatigue behaviour discussed herein is an area where experimental investigations are adding to the knowledge of these systems and expanding the scope of addressable issues in their design.
The structure of the Eurocodes provides opportunities to link-in test evidence to design methods in both of the instances discussed, primarily due to the explicit nature of the code formulation and the more fundamental link between limit states basis and material properties observed directly from tests.

Observations and projected fatigue lives presented herein must be taken in the context of extrapolations based upon a limited data set, lacking confirmatory data at high numbers of load cycle.

The majority of fatigue failure modes were common to those observed in static test counterparts to the fatigue test specimens. Significant incidents of alternative failure modes were, however, also recorded, especially failures in the steel rods.

It is apparent that different adhesive types behave in fundamentally different ways with respect to the fatigue performance and the eventual mode of failure at the fatigue ultimate limit state.

The geometry of the test specimens is at least as important under the conditions of this test as the adhesive type, but the general order of performance across the adhesive types was found to be consistent between specimen sets.

34-7-4 J Kangas, A Kevarinmäki
Quality control of connections based on in V-shape glued-in steel rods

Introduction

Design method of connections based on glued-in rods forming V-shape has been presented in CIB. It is based on large test series of high capacity connections with numerous rods during the past ten years. In Finland standardised steel elements for these connections and a special apparatus for manufacturing the connections have been developed. They were also used in making the connections for the latest test series. The quality control method is developed for the manufacturer who is specialised in fabricating structures with these apparatuses. The purpose is to prevent or at least to minimise the effect of the possible production errors. Already thirty years glued in screws have been used in Finland in the heel connections of glulam columns. They have their own quality control method.

This paper presents a method for the quality control of manufacturing the V-connections.

34-7-6 A Bernasconi
Behaviour of axially loaded glued-in rods – Requirements and resistance, especially for spruce timber perpendicular to the grain direction

Introduction

Glued-in rods allow the introduction of high forces in timber elements. The bond between rods and timber is given by an adequate bond system with sufficient ultimate strength and rigidity, in order to assure a strong bond between the rod material and the timber. To describe the behaviour of glued-in rods and to give a basis for the design, it is necessary to determine the influence of different parameters on the strength of the bond system.

Under normal timber building conditions the gluing of steel is complex. The profile of the steel rods - in form of threads or ribs - is an interesting possibility to avoid the gluing of steel. The mechanical connection given by the ribs assure the load transfer from the steel to the glue-line. In the case of a sufficient resistance of the glue system a shear failure of the timber adjacent to the hole occurs. That way the best capacity of the bond is reached.

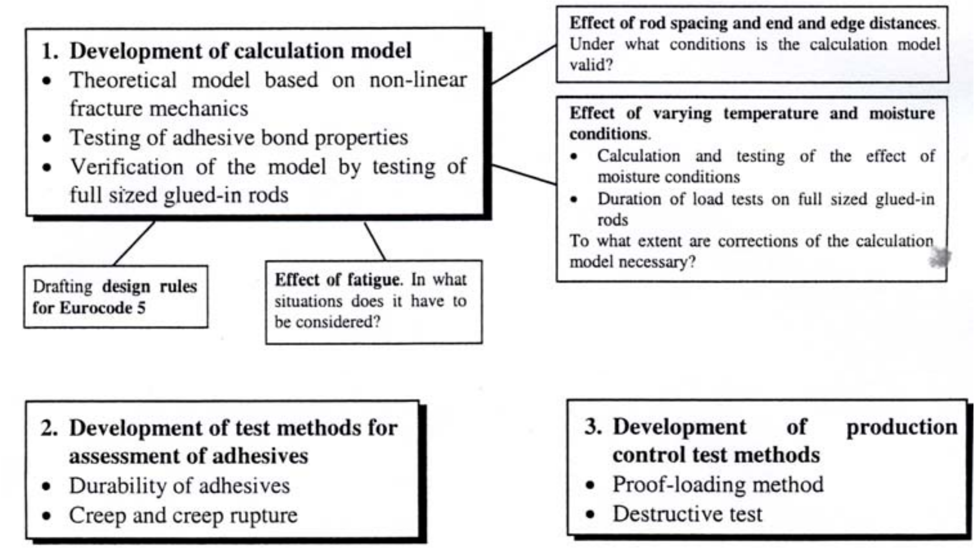
The shear strength of the timber depends on several parameters. The description of the influence of the different influence factors and the description of the stress distribution in the bond line is the starting point for the experimental analysis of the timber strength with use of pull-out tests.

Following the analysis of the results from several experimental pull-out tests on glued-in rods perpendicular to the grain direction with spruce timber, it shows that the strength depends principally on the hole diameter. The other factors of influence have a smaller influence on the strength. These results allow to give a proposal for the design of glued-in rods perpendicular to the grain with spruce timber.

34-7-8 C Bengtson, C-J Johanson
Glued-in rods for timber structures

Summary and introduction

Glued-in rods have been used during a number of years in several of the European countries which are active in timber engineering. They are economically, architecturally and industrially attractive means of transferring forces within a structure and of providing local reinforcement to critical zones of timber members. They also provide an important technology for the repair and upgrading of historically important timber structures which exist throughout Europe. Notwithstanding their importance, internationally accepted design rules for glued-in rods do not exist. This paper presents the main findings within the EU-project "Glued-in rods for timber structures" (GIROD). The project started in February 1998 and ended in March 2001. The paper is based on summaries written by the different partners in the project: Swedish National Testing and Research Institute (SP, coordinator), University of Lund (Sweden), TRADA Technology Ltd (UK), University of Karlsruhe (Germany) and FMPA in Stuttgart (Germany). Additionally, glulam- and adhesive producers from Sweden and Germany participated in the project. The main structure of the project is given below:



Within the GIROD-project the work is focused on rods glued-in with three different adhesive types: two-component epoxy (EP), two-component polyurethane (PUR) and phenol-resorcinol-formaldehyde (PRF). Mainly steel rods were used within the project bi also some fibre reinforced plastic rods (FRP) were tested.
The description of the GIROD-project given here is based on the project structure give above.

Objectives
The objective of the GIROD project is to provide the information required to prepare
standards that will allow an increased, more advanced and more reliable use of glued-in
rods in timber structures. When the project started the working plan was as follows:

12. Perform theoretical and experimental work leading to a calculation model for axially loaded glued-in rods based on the adhesive bond properties as well as the wood and rod material properties. This must take into account the effect of varying climatic and loading conditions as well as fatigue. This step will give information required by CEN TC250/SC5 in the preparation of Eurocode 5 - Design of Timber Structures.
13. Develop test methods for the evaluation of adhesives for glued-in rods with respect to strength, durability, creep and creep rupture behaviour under different climatic conditions. This will support the work of CEN TC193/SC1.
14. Derive test methods for the production control of structural glued-in rod connections. This will support the work of CEN TC124/WG6.
15. Development of a calculation model.

Conclusions
– A calculation model based on a combination of Volkersen theory and fracture mechanics gives good prediction of the pull-out strength for adhesives that bond to the rod such as PUR and EP. The pull-out strength is controlled by two material property parameters that can be easily determined in full-scaled pull-compression tests.
– Fatigue is a significant factor in the performance of glued-in rods and needs to be considered in applications like for instance bridges. Failure can occur in the rod, in the adhesive bond line, in the wood substrate and in the interface between wood and adhesive.
– The effect of rod spacing and edge distances have been clearly demonstrated and proposals to be used in design have been made.
– Storage without mechanical loading in variable outdoor climates had a strength reducing effect mainly on PUR-bonded rods. After storage in 85 % RH the PRFbonded rods were most affected.
– Glued-in rods have a DOL behaviour that can differ quite considerably from that of timber and other timber connections. In 85 % RH the behaviour of EP-bonded rods behaved like the Madison curve while PRF and PUR had much shorter time to failure. At 50 °C the PRF behaved in a better way than PUR and EP.
– It is questionable if the method developed for evaluation of the durability of adhesives for glued-in rods is suitable for the purpose. PRF, which is known to give very durable wood-to-wood bonding, obtains extremely low strength values after testing in wet conditions. It seems that the method punishes adhesives that do not bond to the rod.
– The creep-rupture test method developed for small specimens works well. The creep-rupture behaviour of small specimens compared to this behaviour for full-sized specimens will be further investigated.
– A simple production control test method based on proof-loading has been developed. It is able of detecting a number of serious production errors.

37-7-8 R Steiger, E Gehri, R Widmann
Glued-in steel rods: A design approach for axially loaded single rods set parallel to the grain

Introduction

Steel rods bonded in glulam elements are very efficient to introduce high forces into timber structural members as well as to strengthen timber perpendicular to the grain. Research on bonded-in rods started in the late eighties of the last century and attempts to develop design methods and to optimize the application were intensified within the last 10 years. A good compilation of existing knowledge, including lists of basic literature can be found in the proceedings PRO 22 of the 2001 RILEM Symposium on "Joints in Timber Structures" and in the proceedings of the CIB W18-Meetings 28 and 32 -34.

Several design approaches and code models have been published. By comparing these models and approaches, some discrepancy and partly even contradictions between the models, especially regarding the treatment of isolated parameters, can be found. On this background, a small test program was initiated, to study the influence of a selection of these parameters, known or supposed to be determinant on the pull-out strength of single, axially loaded steel rods with a metric screw-thread, bonded with an epoxy-type adhesive in glulam made of spruce. The tests were focused to determine the influence of timber density ρ, rod-to-grain angle (0° or 90°), length l and diameter d of the rod (or the corresponding drill-hole d_h respectively), represented by the slenderness ratio $\lambda = l/d_h$ on the pull-out strength of glued-in rods.

Important objectives of the test program were that it should be based on practical situations and dimensions and that it should enable a comparison with similar test series. These objectives could only be reached by permitting certain compromises regarding the test layout. Although for example in practice the use of one single rod will not or hardly ever be the normal case, all tests described here were carried out on connections with one single rod, because the examination of such a connection provides a good basis to study the influence of isolated parameters. The tested GSA®-system was "optimized" (type of adhesive and rod, geometry) in a way that failure was forced into the timber. The test results and conclusion are therefore specifically valid for the tested system and loading configuration.

A generalisation of the conclusions will not be possible and a comparison to other test series has to be done with some restraint. Nevertheless it is possible to quantify the influence of the parameters focused by the present study on the pull-out strength of axially loaded rods and to propose an adequate design model.

Conclusions

Based on the test results it can be stated, that for the used GSA®-system:

- the pull-out strength of rods bonded in glulam made of spruce depends on the density of the timber around the anchorage zone. For rods set parallel to the grain, the influence of the density is stronger and can be covered by a power function with an exponent of $c_0 = ½$. Rods set perpendicular to the grain exhibit a smaller dependence on the timber density. There the influence of the density could be taken into account with an exponent $c_{90} = 0.25$ or could be disregarded.
- there is a difference of about 20 to 40 % in pull-out strengths according to the rod-to-grain angle (0° or 90°), which should be considered by design models.
- the influence of the anchorage length is marked more for rods set parallel to the grain, than for those set perpendicular. In the case of rods set parallel to the grain, an adjustment based on $l^{-1/3}$ is a good approach.
- the 0°-series show a dependence on the slenderness ratio $\lambda = l/d$, which can be quantified by $\lambda^{-1/3}$. If a design model is based on λ, the influence of the drill-hole diameter on the pull-out strength, which is about $d_h^{1/3}$, is automatically taken into account.
- with regard to an optimal load transmission capacity steel to timber, rods with smaller diameter should be given preference.

37-7-10 C Faye, Le Magorou. P Morlier, J Surleau
French data concerning glued-in rods

French Professional Guide for Glued-in Rods

French Professional Guide entitled 'Guide professionnel - Assemblages bois: tiges ou goujons colles de grandes dimensions' was published in 1999.

- It contains (in 40 pages) the following topics:
- description of the glued-in rod technology,
- specifications and mechanical requirements for wood, adhesives and rods used for this technology,

- spacing rules between rods and between rods and timber edge for axial and lateral load,
- design rules for glued-in rods parallel or perpendicular to the grain and axially loaded,
- design rules for glued-in rods parallel or perpendicular to the grain and laterally loaded,
- rules for glued-in rod production,
- constructive dispositions.

But, in France, the use of glued-in rods in building constructions is limited because:
- there is no evaluating adhesive method for glued-in rod technology,
- there is no production control test method,
- and there is no knowledge on long term behaviour on this technology.

So, a national research project (2002-2004) is carrying out in collaboration between CTBA, LRBB and the French Glued laminated Timber Syndicate to provide the knowledge required to elaborate these documents.

The main works of the national project are:
- short and long term tests on full size specimen,
- short term and long term tests on small pieces,
- development of finite element model to describe mechanical behaviour including temperature and moisture content evolution into glued-in rods specimen submitted to natural temperature variations.

Two commercial adhesives are tested:
- a two-components Epoxy
- a two-components CR 421 from Prubond.

For all experiments of the national project:
- specimen are tested in axial load with rods glued parallel to the grain,
- GL24 wood quality with 12 mm threaded rod was used (chosen by glued laminated producers),
- spacing rules between rods and between rods and timber edge for axial and lateral load was the one specified in the French Guide,
- specimen manufacturing was made by glued laminated producers according to production rules given by adhesives producers.

37-7-13 A Kevarinmäki
Behaviour of fasteners and glued-in rods produced from stainless steel

Introduction

In addition to better durability the use of stainless steel improves the fire resistance of connections. The stainless steel has good fire resistance properties. Unprotected stainless steels have enough capacity in fire resistance class R30 and certain titan stabilised grades also in class R60 without over design in normal temperature. So the fire protections and/or groovings of the effective cross-sections of timber members for the connections may be avoided with the use of stainless steel. However, also in normal temperature the design rules of timber fasteners are normally given only for the non-alloy steel. The stainless steel grades have a low 0,2 -yield value (normally 220 N/mm^2) but a high tensile strength (even > 800 N/mm^2)) with the ultimate elongation more than 40 %. The paper summarizes the results of the research done for the verification of the design rules for the stainless steel fasteners in normal temperature.

Conclusions

The main result of the research was that the ultimate tensile strength of stainless steel may be fully utilized in the design of the connections of timber structures. The Eurocode equations of yield moments may be safely used for stainless steel fasteners with the ultimate tensile strength of stainless steel. However, for full utilization of the stainless steel fastener capacity, it is advisable to use the tested yield moment values for the actual fastener type and dimension. The stainless steel material may be significantly strengthened in the manufacturing of certain type of stainless fasteners (e.g. threaded nails). Also the strain hardening occurred in the bending of the stainless fasteners may be utilized by using the tested yield moment values. The high ductility and strain hardening of stainless steel fasteners improve the capacity and behaviour of the connections of timber structures.

The withdrawal strength of stainless nails and screws is practically the same as with similar non-alloy fasteners, although the edges of fasteners profiles are generally slightly more round with stainless fasteners. However, the present new version of Eurocode 5 (prEN 1995-1-1:2003) gives about two times too high withdrawal capacity values for the common self-tapping screws loaded perpendicular to the grain. It is also in conflict with

the testing and product standards (EN 1382:1999 and EN 14592:2002). The withdrawal capacity presented for the self-tapping screws by Hansen (2002) is proposed for the correction of Eurocode 5, see equations (3.6) and (3.7).
The anchorage strength of glued-in rods made of stainless steel reinforcing bars is the same as with the non-alloy steel rods and it may be calculated according to the equation presented by Kangas (1994). The full anchorage strength of glued-in ribbed stainless steel rods is reached even with the tension stress value of 800 N/mm^2.

42-7-9 J L Jensen, P Quenneville
Connections with glued-in rods subjected to combined bending and shear actions

Introduction

Glued-in rods have in recent years been widely used in timber structures. Such connections provide an efficient means of transferring moments, and they are architecturally attractive. During the last decade, numerous research papers have been produced on this topic, but nevertheless, design rules for glued-in rods have not been accepted into the major design codes. In the final draft of EC5, a previously included informative annex on glued-in steel rods has been removed.

The vast majority of research papers on glued-in rods have concentrated on withdrawal of single rods, usually steel rods. Relatively little attention has been paid to materials other than steel, and very little attention has been paid to applications of glued-in rods to connections as used in real life. Of the few research papers on moment-resisting connections, virtually all have focused on pure bending. No research (other than reported in the present paper) seems to have attempted quantifying the strength of moment-resisting connections with glued-in rods subjected to combinations of bending and shear, nor did the´now removed annex from EC5 deal with combined bending and shear and possible splitting problems.

Conclusions

Glued-in rods are widely used for moment resisting connections. However, no design rules are given in EC5. Future editions of EC5 should preferably contain guidelines not only for withdrawal of single rods, but also for moment resistance, shear resistance, and resistance to combined actions of moment and shear.

Simple models suitable for practical design have been derived for pure moment and pure shear actions based on fracture mechanics, and combined actions have been considered empirically for connections with rods of hardwood. Test show good agreement between theoretical and experimental failure loads. It is believed that the presented models with minor alterations can be adopted for rods of steel, aluminum and other materials.

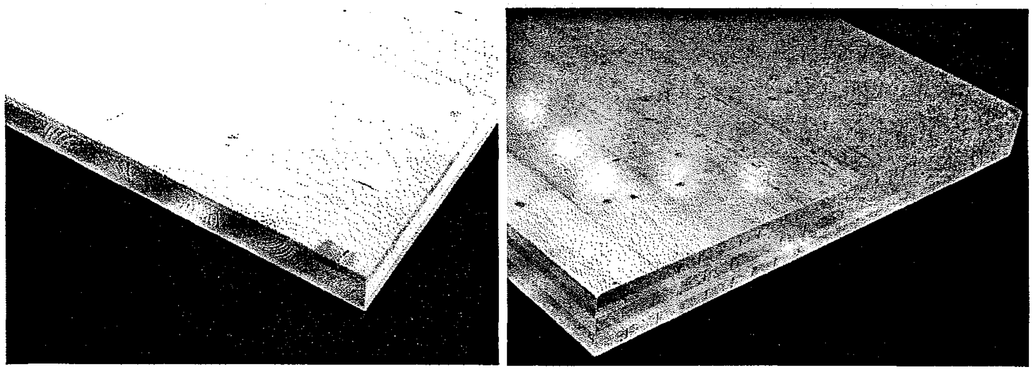
Considerable increase of the shear strength may be obtained in simple ways, e.g. by gluing plywood plates on to the beam ends. Reinforcement by means of screws may prove efficient for this purpose as well.

4.7 JOINTS IN TIMBER PANELS

39-7-5 T Uibel, H J Blass
Load-carrying capacity of joints with dowel type fasteners in solid wood panels

Introduction
Solid wood panels with cross layers have been used more and more frequently in timber engineering in recent years. Their use in constructions requires their connection with each other and with other components of the construction. To that purpose dowel-type fasteners can be used. It is possible to position the fasteners perpendicular to the plane of the panels or in their narrow sides. In this paper only the first possibility will be discussed.

The load carrying capacity for dowel-type fasteners is usually calculated according to Johansen's yield theory. Embedding strength as well as the yield moment of the fasteners are important parameters in this calculation. Withdrawal strength is needed to calculate the load carrying capacity of axially loaded screws or nails. The aim of a current research project at the University of Karlsruhe is to develop a proposal for calculating the load carrying capacity of joints with dowel type fasteners in solid wood panels.



Solid wood panels with cross layers - two examples of products (pictures: Informationsdienst Holz)

40-7-2 T Uibel, H J Blass
Edge joints with dowel type fasteners in cross laminated timber

Introduction
During the 39h meeting of CIB-W18 the authors presented proposals for the calculation of the load carrying capacity of joints with dowel type fasteners positioned perpendicular to the plane of cross laminated timber (CLT). In continuation of the research project the load carrying capacity of edge joints with dowels and screws in CLT was examined. To calculate the load carrying capacity of dowel-type fasteners according to Johansen's yield theory, the yield moment of the fasteners and the embedding strength are needed. The withdrawal strength is necessary to calculate the load carrying capacity of axially loaded screws. In addition, the withdrawal strength is important for estimating the rope effect of laterally loaded connections. In the narrow sides of CLT the fasteners can be positioned parallel to the grain direction. The embedment strength and the withdrawal strength are also influenced by gaps and grooves.

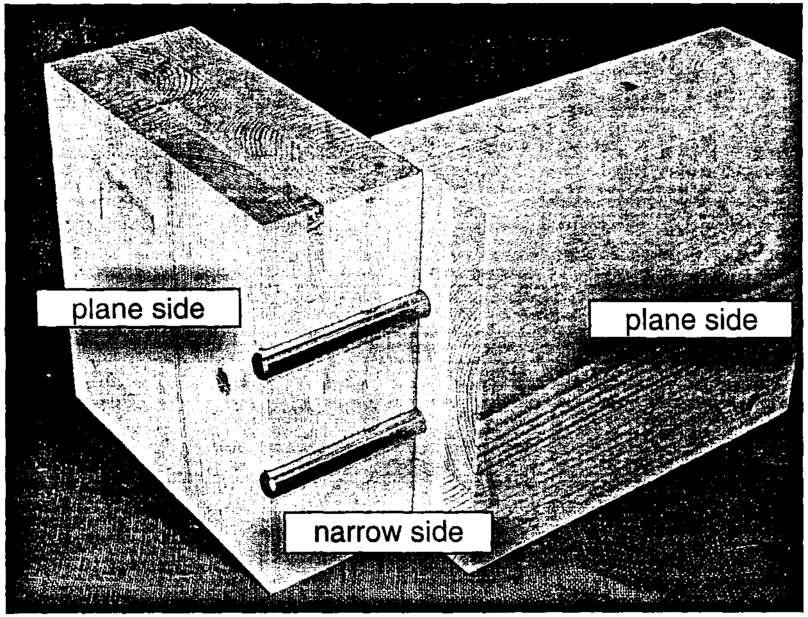


Fig. 1: Opened connection with dowels in cross laminated timber

Conclusions

For calculating the load carrying capacity of edge joints in CLT the parameters embedment strength and withdrawal capacity were examined. On the basis of statistical analysis of a multitude of test results it was possible to develop functions for predicted values of these parameters. Proposals for characteristic values are also given. The validity of the presented equations is limited to CLT with a characteristic density of 400 kg/m3 made of spruce. In tests with connections the required minimum edge and end distances and spacings of fasteners were determined. In addition the tests verify the calculation of the load carrying capacities. To determine the long-term behaviour of edge joints with self-tapping screws in CLT tests are performed.

42-7-10 H J Blass, G Gebhardt
Load Carrying Capacity of Timber-Wood Fiber Insulation Board - Joints with Dowel Type Fasteners

Introduction

So far wood fibre insulation boards (WFIB) are used as thermal and acoustic insulation in timber constructions. As wood-based panels, WFIB are suited for the transfer of loads caused by wind and earthquakes in timber frame constructions. Until present this task has been undertaken by plywood, particle boards and OSB. This new field of application of WFIB has been analysed in a research project at the Universität Karlsruhe.

In this paper a proposal is given to calculate the load-carrying capacity of timber-WFIB-joints with dowel type fasteners. For this purpose the load-carrying capacity of joints in timber and WFIB may be determined according to Johansen's yield theory and an extension of this theory. Tests were carried out to estimate the embedding strength of nails in WFIB and the crown pull-through resistance of staples in WFIB. The test results verified the calculation model and the stiffness properties of timber-WFIB-joints were evaluated.

Conclusions

The embedding strength of nails in WFIB was tested. As result of a multiple regression analysis, the embedding strength may be calculated considering the density of the WFIB and the diameter of the fastener. In further tests, the pull-through resistance of staples in WFIB was examined. The pull-through resistance depends on the thickness and the density of the WFIB. To consider the influence of a counter batten in the failure modes according to Johansen's yield theory, the equations for two failure modes were extended. The results of the previous tests were used to calculate predictive values of the load-carrying capacity of timber-WFIB-joints tested in further tests. The stiffness of timber-WFIB-joints may be calculated depending on the densities of the joint members, the thickness of the WFIB and the diameter of the fastener. With these results the load-carrying and displacement characteristics of timber-WFIB-joints may be evaluated and further be used for the calculation of shear walls with WFIB sheathing.

4.8 LOAD DISTRIBUTION

23-7-2 H J Blass
Load distribution in nailed joints

Abstract
An existing model has been extended to study the influence of plastic deformations on load distribution in multiple-pin timber joints. The model was verified experimentally with tension tests on double-shear specimens made from spruce and connected with nine nails aligned parallel to the direction of loading. The test results agree well with the model predictions. There is no significant influence of number of nails on ultimate load.

Introduction
During the last few years, the strength of mechanical timber connections and its different influencing parameters like embedment strength of wood or moment capacity of nails have been investigated extensively. These works did not consider load distribution in mechanical timber joints, though Steck showed great differences between the modification factors for number of fasteners in several international and national timber design standards and emphasized the need for further research in this area.

Even assuming ideal conditions – identical load-slip curves of single fasteners – the distribution of the load in multiple-fastener joints is non-uniform when the fasteners are aligned parallel to the direction of loading. Therefore, in design standards, the load capacity per fastener decreases with increasing number of fasteners parallel to load.

One reason for the non uniform load distribution is the different elongation of connected members. For example, consider Fig. 1: Between the first and second nail. Member 1 is loaded by force F minus fastener load 1 while member 2 resists only fastener load 1. Assuming the same extensional stiffness for both members, the elongation of member 1 between the first and second nail will be greater than the corresponding elongation of member 2. These different elongations must be compensated for by different displacements of the first and second nail. Different displacements mean - at least as long as the yield load is not yet reached – different fastener loads.

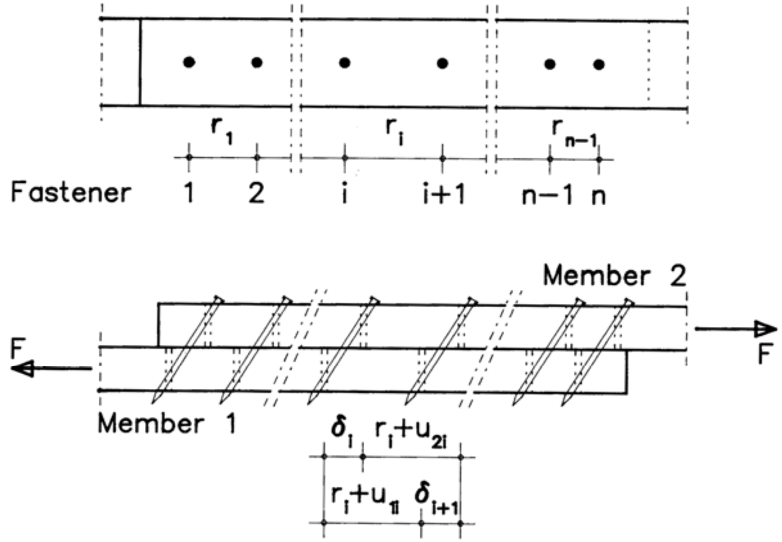


Figure 1.

Assuming linear-elastic load-slip curves, theoretical solutions of this problem were presented by Lantos (1969) and Cramer (1968). The assumption of a linear-elastic behaviour of load-slip curves may be approximately valid in the proportional range, but its extension to ultimate loads is not realistic. If the load is increased over a proportional limit, the most highly stressed fasteners at the ends of the joint begin to deform plastically. Moreover, the embedment strength in the contact areas between these connectors and the wood is reached, and redistribution of load from the fasteners at the ends to those in the centre of the joint will result. After each fastener has reached its yield load, the differences in fastener loads become minimal and the joint reaches its yield load.

It follows that ideally, every fastener might reach its yield load at joint failure. Therefore load distribution in joints should not affect load capacity. Test results of several researchers indicate, however, that the ultimate load per fastener decreases sometimes considerably with increasing number of fasteners arranged parallel to load. This suggests that the failure mode in many connections may not be attaining the joint's yield load. Instead, joint load capacity may be constrained by preliminary wood splitting. Consequently, the potential load capacity of the connection is not realized because load-slip curves of single fasteners break off and ideal redistribution of load is prevented. Oversized and misaligned bolt holes or split ring grooves tend to make the situation even worse: by causing differences in initial slip of single fasteners which makes the load distribution very uneven. This may lead to some single fasteners reaching their maximum load while other fasteners just begin to carry load because of their greater initial slip. In case of long-term or repeated loading, creep deformations and residual plastic deformations after previous higher loading also affect load distribution.

A realistic model to describe load distribution in multiple fastener joints must therefore - apart from different elongations of the joint members - take into account influences from fabrication tolerances and variable load-slip curves within the joint. Wilkinson (1986) presented a model to calculate load distribution in bolted joints. Knowing the different shape of load-slip curves of different fasteners in the connection allows the calculation of load distribution up to ultimate load. Using Wilkinson´s model, the following steps are proposed in developing modification factors for multiple-fastener joints:

- Determine variation of load-slip behaviour in connections.
- Determine variation of initial slip due to fabrication tolerances within joints.
- Extend Wilkinson's model to calculate load distribution taking into account all important parameters.
- Simulate joints with different number of fasteners taking into account correlation of load-slip behaviour within joints.
- Calculate maximum loads of simulated connections and determine characteristic values of ultimate load for different number of fasteners.
- Derive modification factors by comparing characteristic values depending on number of fasteners.

The objective of the present investigation was to develop a model taking into account all important parameters influencing short term load distribution in mechanical timber joints. To verify the model, tests were carried out with specimens, which had been used in previous tests to determine variation in load-slip behaviour within joints. Therefore, single-nail load-slip curves of these specimens were already known.

Summary
An existing model to determine load distribution in mechanical timber joints was extended to take into account the influence of plastic displacements after previous higher load levels. To verify the model, tension tests were carried out with nailed double-shear specimens made from spruce and nine nails arranged parallel to load. The load-slip behaviour of ten single nails was known for each specimen from a previous investigation. The expected maximum load of the nine-nail tests was determined with the model using the known load-slip data of the single-nail tests. Comparing the expected with the actual maximum loads yielded very good agreement on the average. A visual comparison of load-slip curves from -the tests with those from the model also showed good agreement in most cases.

The maximum load per nail for the single-nail tests did not differ significantly from the corresponding values from the nine-nail tests. Therefore, the maximum load of a multiple-nailed joint can be estimated as the sum of those for individual nails, provided joint failure is by nail yielding.

25-7-12 C J Mettem, A V Page
Load distribution in multiple-fastener bolted joints in European whitewood glulam, with steel side plates

Introduction
Draft Eurocode 5 requires as a principle that account should be taken of the fact that the load-carrying capacity of a multiple-fastener joint will frequently be less than the sum of the individual fastener capacities. The use of relatively thick metal plates, usually of mild steel, in conjunction with bolts or plain round dowels, is common in forming connections in timber engineering construction, particularly with glued laminated timber (glulam). Designs normally involve double shear planes, and the plates may be positioned as side plates, or as central inserts, or 'flitches'.

An accompanying paper presents a brief review of the current research and code status in relation to the topic of multiple-fastener modification factors for such joints. This paper describes recent tests, supported by a linear elastic orthotropic finite element analysis. The work has been conducted on both single and multiple bolted joints. An especially designed test apparatus, which could accept thin central members of European whitewood glulam, was used, to ensure failure in a pure embedment mode. Strain-gauged mild steel side plates were used on this test jig, following an established technique developed by previous researchers.

The results of the evaluation are compared with work previously reported by others to CIB W18A. They are also discussed briefly in relation to current design recommendations.

Conclusions

Tests and analytical investigations have been made on load distributions in multiple-fastener bolted joints, using European whitewood glulam, with steel side plates. The proportions of load carried by each bolt have been determined. These have been found to be very uneven, especially in the perpendicular to grain case.

For each of the three modes of loading which were investigated, namely compression parallel to the grain, tension parallel to the grain, and compression perpendicular to the grain, reduction factors have been determined, by test and from analysis. These reduction factors were related to the type of calculation which is based on summing the load carrying capacity of a multiple-fastener joint by taking the number of fasteners multiplied by the individual fastener capacity.

In compression parallel to the grain, the reduction factor determined by test was only about four per cent different from the corresponding value shown by analysis. In the tension case, agreement was within ten per cent. In compression perpendicular to the grain, agreement was less close, with a difference of twenty two per cent between test and theory. However, this is still considered quite reasonable, in view of the substantial influences tending to cause variability in such specimens.

The experimental measurements were conducted using a special test apparatus, which was designed to ensure that failure took place in a pure embedment mode. Other precautions were also taken in manufacturing the specimens. Single-fastener tests were conducted which were then compared with EC5 (draft) design procedures. These comparisons showed that the plastic embedment assumption was valid.

Both the tests and the analysis suggested that the load sharing amongst the fasteners was more even in the case of tension parallel to the grain, than in the case of compression parallel to the grain. The reduction factor for compression perpendicular to the grain was substantial, and should apparently be even more severe according to the finite element analysis than was determined by test. The test evidence suggested that a reduction factor of 0.50 should apply to four bolts in such a situation.

The reduction factors determined for the various four-bolt parallel to grain configurations in this research were in the range 0.66 to 0.89. A brief comparison with two other sets of tests was made in this paper, and in an accompanying paper, a selected review of research and codes was given. All the supporting evidence suggests that BS 5268:Part 2 and EC5 (April 1992 Draft) make insufficient allowance for this phenomenon..

30-7-5 A Jorissen
Multiple fastener timber connections with dowel type fasteners

Abstract

The load carrying capacity of a connection with a number of fasteners, a so called multiple fastener connection, does generally not equal the load carrying capacity of a single fastener multiplied by this number. Often the connection fails at a lower load because the timber splits. The result can be a brittle failure. The geometrical parameters, material properties and the number of fasteners determine the load carrying capacity and the type of failure.

To investigate the governing parameters a comprehensive research, both theoretical and experimental, started at the Delft University of Technology at the beginning of 1994.

About 950 tests on single and multiple bolted connections have been carried out. The tests will be presented and discussed.

Introduction

This paper deals with the load carrying capacity of multiple fastener timber connections, timber to timber, with dowel type fasteners in a row parallel to the grain. Symmetrical double shear connections are studied. The connection is in the grain direction.

The fact that the load carrying capacity of a multiple fastener connection, $F_{multiple}$, does not equal the load carrying capacity of a single fastener connection, F_{single} multiplied by the number of fasteners n is taken into account in many national design codes. An effective number of fasteners n_{ef} $< n$) has been introduced. $F_{multiple}$, is calculated according to equation:

$$F_{multiple} = n_{ef} F_{single}$$

In most countries F_{single} is calculated according to the so called European Yield Model, first described by Johansen, which allows n_{ef}, as defined in the national design codes, to be compared. Hardly any agreement on the design values for n_{ef} exists. Even if the background of the design rules is more or less the same there is no agreement. For instance, the design rules for Europe and Canada are both experimentally based. Nevertheless, n_{ef} for Europe is two to three times higher than n_{ef} for Canada.

The last fifty years research have been carried out to determine the load carrying capacity of multiple fastener connections.

Lantos, Cramerand, v d Put presented an analytical method for the calculation of the load distribution among the individual fasteners based on the assumption of identical linear elastic behaviour of all fasteners. Van der Put extended his model by a plastic analyses.

Wilkinson developed a numerical model where fabrication tolerances and variability of single fastener non-linear load-slip behaviour are taken into account. The result of this model is a randomly non uniform load distribution among the individual fasteners. Moss extended Wilkinson's' research by simulation of a large number of joints. Moss used a piecewise linear approximation of the actual fastener load-slip curve measured by Wilkinson.

Concluding remarks

For most configurations no significant difference has been found in the load carrying capacity between tests carried out in compression or in tension.

The differences in individual hole clearances can be of some importance if the end slip is rather low (smaller than 2 to 3 mm). In most cases the end slip is higher. As a consequence, in most cases the individual hole clearances does not affect the load carrying capacity.

The reduction factor R in load carrying capacity is calculated according to $R = n_{ef}/n$.

The effective number of fasteners n_{ef} is calculated according to

$$n_{ef} = \frac{\text{characteristic result multiple fastener}}{\text{characteristic result single fastener}}$$

This means that the results from tests on the single fastener connections are compared to the results from tests on the multiple fastener connections. The results from tests on single fastener connections are, however, for the connections with rigid bolts significantly below the calculated values according to the European Yield Model. Consequently, the reduction factors for connections with rigid bolts decrease if the strength values for the single fastener connections according to the European Yield Model are compared to the results from tests on multiple fastener connections. This results in a more significant influence of the timber thickness (a slenderness influence). This comparison has not been made, because this is not a comparison between equal quantities. Consequently, it must be investigated why the results from tests on single fastener connections differ so significantly from the theoretical values.

It is quite remarkable that the load carrying capacity is almost linear to the number of bolts. If this is true, the fact that the effective number of bolts n_{ef} does not equal the actual number of bolts n is explained by the spacing and the timber thickness of the middle member.

32-7-5 A Mischler, E Gehri
Strength reduction rules for multiple fasteners joints

Introduction

The load-carrying capacity of a multiple fastener joint is often significantly lower than the strength of one fastener times the number of fasteners. Therefore, strength reduction factors for multiple fastener joints had to be introduced. But there are large discrepancies between these factors proposed in structural codes such as the Canadian CSA 086.1 and the European ENV 1995-1-1 and ENV 1995-2.

Failure modes of multiple fastener joints

The strength reduction of multiple fastener joints is mainly caused by the failure modes which must not be the same as for the single fastener. In multiple fastener connections which are commonly used in timber structures, the failure is often caused by the timber parts and not by the fastener.

The following failure modes in the timber are possible:

- Splitting of the timber in a row of dowels
- Tensile failure of the timber in the reduced net section
- Combination of splitting, shear plug and tensile failure.

If premature timber failure is avoided by an adequate design of the connection or by reinforcing the joint area, the failure occurs in the connection itself. The load-carrying behaviour of a dowel type connection can be described by the Johansen's theory. A very important condition in the Johansen theory is that the joint allows large ultimate deformations in order to reach the plastic failure modes. In multiple fastener connections even small fabrication tolerances lead to an uneven load distribution among the fasteners. A certain amount of balancing of these unequal forces is possible by plastic deformations. Therefore the ultimate deformation capacity of a multiple fastener joint has to be even larger than the capacity of a single fastener connection.

As the timber fails in a brittle way, these deformations are typically only possible if the failure occurs after significant plastic deformations of the steel dowel. This failure mode (Type III, according to Johansen) can be reached when fasteners of effective slenderness ratio bigger than a limit slenderness ratio are used.

Conclusion

The tests on steel-to-timber connections show the influence of

- dowel slenderness ratio
- dowel strength
- end distance and spacing
- fabrication tolerances

on the strength reduction factor of multiple fastener connections.

The only difference to the research of Jorissen is caused by the fact, that his values for the single fastener connections with rigid dowels are 30 % lower than predicted by the Johansen theory. The Johansen theory has been validated by many researchers for single fastener connections. As long as there are differences of 30% in the resistances of the single fastener connection it is not possible to establish reliable rules for multiple fastener joints.

35-7-6 P Quenneville, M Kasim
Effect of row spacing on the capacity of bolted timber connections loaded perpendicular-to-grain

Introduction

When a wood member is loaded perpendicular to the direction of grain by a bolted connection, failure of the joint may result either by yielding of the bolts, crushing of the wood under the bolts or by splitting of the wood member. The first two failures are considered ductile, while splitting or fracture of the wood is brittle and may lead to catastrophic collapse. Most researchers agree that the bolt or line of bolts furthest from the loaded edge initially carries the biggest share of the load, and, therefore, splitting or fracture is more likely to occur at this level. The aim of any connection design is to ensure that brittle failure does not happen prior to yielding of the bolts or the wood, making it necessary to predict the splitting strength of the wood when the joints are subjected to a force perpendicular to grain.

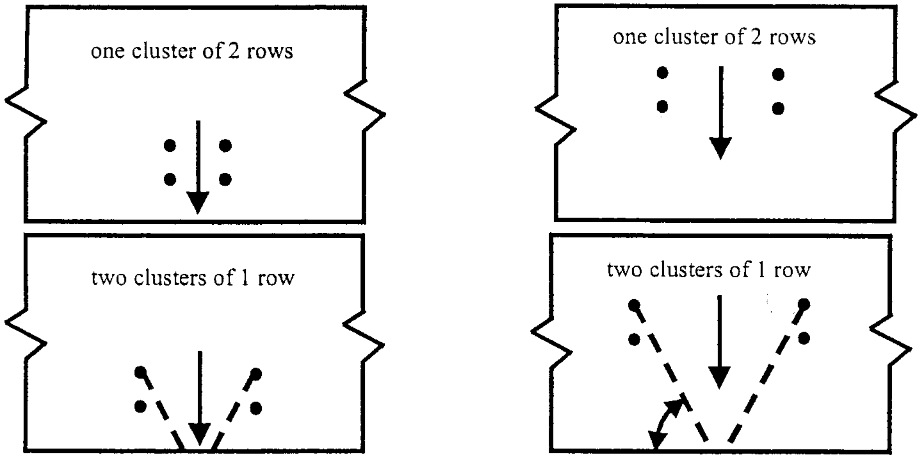
The Canadian design standard for wood structures (CSA 086) lays down minimum requirements for the bolt spacing, edge distance and end distance that define the layout of bolt groups. Tests conducted by many researchers showed that these minimum requirements do not guarantee ductile failure especially in multiple-bolt connections. In bolted connections, the manner in which the load is transferred from the bolts into the wood members affects to a great extent the capacity of the joints. Overlapping of the load paths of adjacent rows of bolts in a joint, for instance, can reduce the effectiveness of the bolts in the group, so that the joint capacity is less than the sum of the capacities of the individual rows.

Cluster Principle

A load transferred to a wood member perpendicular to grain by a bolt causes compression in the local area under the bolt. The ability of the member to spread the concentrated load and the manner in which the load is spread over the remaining depth can have a significant effect on the capacity of joints. Overlapping of the load path from the bolts in a row with those in adjacent rows within the same joint is not considered in design standards. However, the concept of fastener cluster has been used by many researchers, especially in Europe.

Foschi used the concept of cluster to predict the ultimate capacity of riveted joints. Later, other researchers used and applied the same concept to different types of fasteners. Görlacher used the concept of cluster in his design proposal for the new Eurocode 5. He recommended that several groups of fasteners can be considered as one cluster with one load carrying capacity if the distance between the groups is less than $0.5h$ (h being the depth of the member). On the other hand, they can be considered separate groups if the distance between the groups is more than $2h$. The assumed angle of distribution with the grain direction is therefore less than 45° (depending on the unloaded edge distance). A reduction factor is proposed if the distance is between $0.5h$ and $2h$. The $2h$ limit differs from that used by Ehlbeck, Görlacher and Werner where only h was assumed. The latter, however, gives a more consistent load distribution with the approach used for interior connections.

Quenneville and Mohammad used the cluster concept to arrive at the splitting capacity of bolted connections. For interior connections, they assumed that a group of bolts in a connection can be assumed as one cluster if the distance between the rows of bolts does not exceed the depth of the member less the unloaded edge distance (h-e_p). The assumed angle of distribution is therefore 63°. On the other hand, if the distance between the rows of bolts exceeds the depth of the member less the unloaded edge distance (h-e_p), the resistance of each row is determined and the connection resistance would constitute the sum of the resistance of each of the rows.



Load distribution in interior joints.

35-7-7 A J M Leitjen
Splitting strength of beams loaded by connections, model comparison

See Dowel-type fasteners loaded perpendicular to grain

35-7-2 K Komatsu, S Takino, N Nakatani, H Tateishi
Analysis on multiple lag screwed timber joints with timber side members

Abstract

Non-linear load-slip relationships of multiple lag screwed timber joints with timber side members were analyzed by making use of the classical Lantos theory which deal with load distribution in the members of general multiple timber joints under axial force.

Load-slip relationships obtained from single lag screwed joints were fitted by the three parameters exponential function. Then step-wise load incremental calculation method was applied on a series of the finite deferent equations which were obtained by applying Lantos theory to the multiple lag screwed timber joints under axial force.

A series of experimental studies was also conducted on the double sided timber to timber joints fastened by single raw lag screws. We prepared basically five different combinations of test specimens composed of single main member and double sided members connected by several lag screws located along one line.

From the comparisons between calculations and experimental result, it was recognized that the non-linear calculations could predict the non-linear load-slip behaviour at least up to the yielding point of each multiple lag screwed joint. So far as using the 'load incremental method', it was difficult to predict precisely on the ultimate stage of each multiple joint specimens.

Maximum strength per fastener tends to decrease as the number of fastener increases, and also this trend could be predicted theoretically.

Conclusions

In this article, we tried to apply the classical Lantos theory to the nonlinear analysis of the multiple lag screwed timber to timber joints. From the comparisons between calculations and experimental results, it was recognized that the theoretical calculations could predict precisely the nonlinear load-slip behaviours up to the point, which exceeded the yielding point, but before maximum load of each multiple lag screwed joint specimen. So-called multiple effect on the maximum strength per fastener was also recognized by experiment as well as theoretical calculation.

So far as using the 'load incremental method', it was difficult to predict precisely beyond the stage of ultimate load where local cracking or splitting had already occurred.

39-7-1 P Quenneville, M Bickerdike
Effective in row capacity of multiple-fastener connections

Introduction

It is well accepted that the effective in-row capacity of multiple-fasteners is potentially less than the sum of the capacity of each individual fastener in the row. Since most design procedures are based on principles developed for single fasteners failing in a ductile mode, multiple-fastener connection resistances could not be reasonably predicted for all cases. To overcome this shortcoming, design procedures would include modification factors that took various geometric variables into account, trying to cover all possible situations. In recent past, research observations have shown that this decrease from the optimal resistance is due to the brittle failures of the wood fibers surrounding the fasteners. Multiple-fastener connections would fail in row-shear, group tear-out, or splitting.

A numerical model based on the load-slip behaviour of single fastener connections was developed to study the behaviour of multiple fastener connections failing in row-shear. Variables taken into account include single fastener load-slip curves with varying resistances, stiffness and ultimate slip, fastener gaps, and number of in-row fasteners. The numerical model was developed using the finite element software ANSYS. Numerical predictions of the connection ultimate load were compared to available experimental results of the modeled connection configurations, showing good agreement. The model was then used to predict the ultimate resistances of connection configurations for various end distances and fastener spacings. From these predictions, a design equation for row-shear was developed.

This design equation is presented and compared to the Canadian, European and American design provisions for the in-row connection resistance.

Conclusions

A set of design equations to predict the resistance of dowelled connections failing in a brittle manner (row-shear, group tear-out, and net tension) for parallel-to-grain loading is proposed. These equations are based on experimental information along with the results from the numerical model of Bickerdike, and prove to be an effective method of predicting the strength of bolted timber connections loaded in the parallel-to-grain direction.

The relationship of effective number of fasteners to the number of bolts in-the-row was used to compare the proposed set of equations to existing design standards. The proposed design approach replicates the effective number of fasteners for the configurations that were used to develop the Eurocode and Canadian code.

Utilizing the minimum resistance determined from all possible failure modes provides the flexibility to consider all possible connection scenarios in the parallel-to-grain loading direction.

4.9 LOAD DURATION

22-7-4 A J M Leijten
The effect of load on strength of timber joints at high working load level

Introduction
In many publications load duration has received a considerable amount of attention. Tests have revealed that the strength of timber is time- and load level dependent. For timber, damage accumulation models have been developed to describe this phenomenon. A point of discussion is still, whether the strength decreases in time, independently of the load level, or that damage occurs after a certain load history. Some theoretical models backup this last approach. Although the attention is mainly focused on the load duration properties of timber, tests on timber joints seem to behave in the same way. In present timber codes long duration factors are used for timber and timber joints which range from 0,5 to 0,8

The prime objective of the present long duration tests on timber joints at the Stevin Laboratory is to get more insight at what load level and at what time damage occur. Although the research program will last another 5 years (started in 1983) already some results have become available and will be presented in this paper. The results indicate that there is indeed reason to believe that below the long duration threshold value no damage occurs and that therefore present long duration values might be to conservative. Acceptation of the idea that the strength will not be affected by loads below this threshold level would in its ultimate consequence lead to a total neglect of the load duration effect.

Tentative Results
In 1986 a number of joints loaded to 50 % were unloaded and tested to reveal any damage. No significant damage was recorded. In 1988 again a number of joints which had been loaded for 4,5 years were unloaded and SSD*-tested. During the first two years these joints were loaded to 40 % after which the load was raised to 50 % and continued for another 2,5 year. Prior to the SSD*-tests, the joints have been left unloaded for a 5 months period to record the creep recovery.

The most important conclusion is that at the present stage of the research program no strength loss is detected.

*SSD = Standard-Short-Duration

25-7-9 J W G van der Kuilen
Determination of k_{def} for nailed joints

Summary
In 1962 a comprehensive test program was started at the Stevin laboratory at Delft University of Technology to study load duration effects in timber joints. Nailed joints, tooth-plate and split-ring joints were incorporated in the program. The load levels chosen were 60, 65, 70, 75, 80, 85 and 90 % of the short duration strength. Meanwhile all specimens failed except the nailed joints at the 60 and 65 % load levels.
In 1983 the original test program was extended with specimens of the same type but at lower load levels. The load levels were chosen at 30, 40 and 50 %. The 30 % load level is considered as the level of a service load.

In the meantime a general creep and damage model has been developed based on reaction equations of plastic deformation in the molecular structure and on the transformation of stresses to surrounding elastic material. This model is used to determine parameters and deformation factors.

Conclusions from the study
Creep measurements of nailed joints have been analyzed using the creep and damage model of Van der Put

The creep results are analyzed as two processes. Both may be described by a simple In(t) formula with two parameters. The first parameter represents the steepness of the creep line on a logarithmic time scale. The second parameter represents the bending point in the creep line approximation.

In the evaluation the sudden increases in deformation which occur in the first years of the tests are not considered. After longer periods this effect seems no longer noticeable. It seems clear, however, that the second process is induced by climatical changes. The changes occur during a distinct change in relative humidity. This change introduces a change in the microstructure i.e. the number of flow units or the number of load-bearing molecular bonds.

The start of the second process depends on the season in which the creep test was started. Hence, to predict the deformation of a joint over a number of years may be done with an average delay time. Based on the above mentioned creep tests, this average delay time may be taken as half a year or 180 days. The accuracy of the prediction highly depends on the time of loading.

36-7-6 S Nakajima
Evaluation and estimation of the performance of the nail joints and shear walls under dry/humid cyclic climate

Abstract
As shear walls installed in timber houses usually experience dry and humid cyclic climate during their service life the effect of the humid and dry climate on the performance of the nail joints was evaluated. Lateral nail resistance tests were conducted after conditioning the test specimens in the dry/humid/dry cyclic climate. The tests were conducted for all possible combination of the surface grain direction of the studs and the sheathing materials and the loading direction. The strength reduction of the shear walls due to the dry and humid cyclic climate was estimated by the simplified model.

Not only the conditioning schedule but also the final condition, i.e. the condition of the test specimens during the test, affected the strength and stiffness of the nail joints. The condition of the test specimens, i.e. wet or dry, was supposed to be one issue that should be discussed in the process of evaluating the stiffness and strength of the joints and the shear walls that will be exposed to a certain humid climate.

Conclusion
The yield strength of the "plywood-lumber" nail joints was not reduced by the humid and dry conditioning process and was almost same or approximately 15 % higher than that of the nail joints conditioned at the standard climate. The reduction ratios of the initial stiffness of the "plywood - lumber" nail joints due to the humid and dry conditioning process ranged from 53 % to 60 %.

Basically the yield shear strength of the "OSB - lumber" joint was reduced to approximately 10 % by the humid and dry conditioning. The reduction ratios of the initial stiffness of the "OSB - lumber" nail joints due to the humid and dry conditioning process ranged from 48 % to 65 %.

Not only the conditioning schedule but also the final condition, i.e. the condition of the test specimens during the test, affected the strength and stiffness of the nail joints. The condition of the test specimens, i.e. wet or dry, is supposed to be one issue that should be discussed when evaluating the stiffness and strength of the joints and the shear walls that will be exposed to a certain humid climate.

40-7-7 J W G van der Kuilen, A P M G Dias
Creep of timber and timber-concrete joints

Abstract
According to the EC5, the additional deformation of joints at long term shall be calculated as the short term deformation multiplied by the deformation factor k_{def} The short term deformation shall be determined based on the slip modulus of the joint (K_{ser}). The slip modulus of the joints may be obtained either from the models given in Eurocode 5 for some types of joints, or alternatively, from experimental tests in accordance with EN26891. In experimental tests the long term deformation is related to the short term deformation through the creep factor of the joints, and is determined relative to the initial deformation. In this situation the deformation factor from Eurocode 5 and the creep factor of the joints have an equivalent meaning. An essential difference is the determination of the short term deformation. In EC5 it is based on the joint slip modulus, while in the creep tests it is usually taken the initial deformation. In the tests considered here, the initial displacement was taken after 10 minutes after the loads are applied.

In this paper a detailed analysis is made on the determination of joints long term deformations in both situations: based on the creep factor determined from creep tests and based on the Eurocode method. These results are available from a large number of long term tests on timber-timber and timber-concrete joints. These tests have been performed at Delft University of Technology and at the University of Coimbra. Various types of fasteners have been used and both climate-controlled and uncontrolled conditions have been considered. The influence of climatic aspects on the long term deformations is also discussed and a proposal for creep factors for different duration of load classes is made.

4.10 NAIL PLATES

14-7-1 B Norén
Design of joints with nail plates

Introduction
For some time a small Nordic group has discussed guidelines for design of joints with nail plates in timber structures.
Their draft proposal contains these sections:
1. Definition and symbols
2. Design calculations
3. Material
4. Manufacture and control
5. Testing of plates and joints

Section 5 is principally a reference to the testing standard proposed by NORDTEST and based on the RILEM/CIB recommendations.

The paper presented here includes only Section 1 and 2 in which is given rules for design calculations. The outline is kept from the proposal:
2.1 Model for calculation of joints
2.2 Design against plate failure
2.3 Design against plate grip failure
2.4 Design against wood failure
2.5 Rules for specific joints
2.6 Calculation of slip

18-7-6 N I Bovim, B Norén
The strength of nail plates

Introduction
The strength of nail plates with respect to failure in the plate material have been examined by NTI as background for a Nordic proposal This proposal has been presented by B. Norén in **Paper 14-7-1**.
The present paper gives a summary of test results and proposed design method for design against plate failure.
Today, the design against plate failure is based on:

1) strength of plate with respect to forces parallel to the joint between connected timber members. This is called the shear strength of the plate.
2) strength with respect to forces perpendicular to the joint. This is called the plate strength of the plate.

The existing rules give no interaction formula for combination of "shear" and "plate"-forces. Consider Fig. 1. The example demonstrates clearly that existing rules do not take care of the orthotropic behaviour of nail plates. An interaction formula based on "plate" and "shear"-components would be still worse and in reality meaningless!
In many cases the rules are extremely unsafe (up to 200 percent for design values given in the Norwegian approval of the tested plate). This is the main reason for the work presented here.

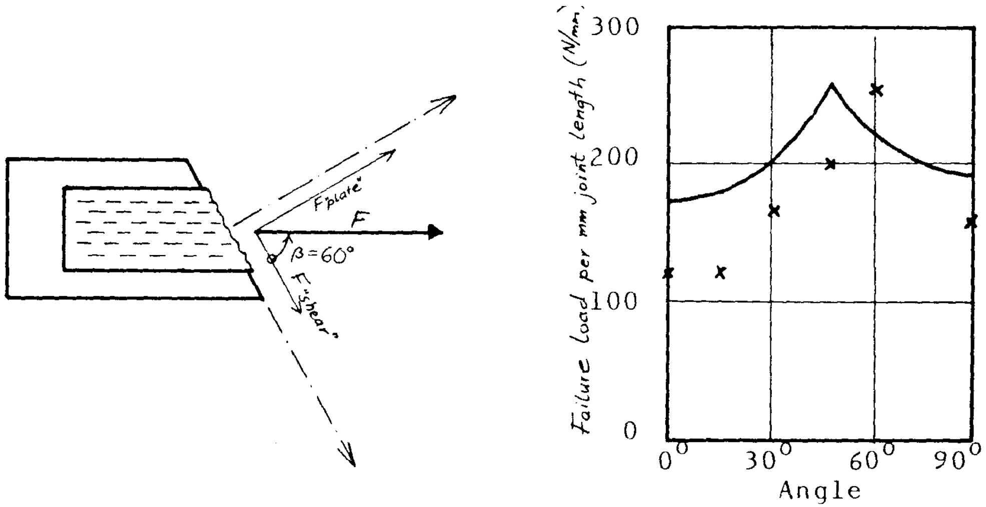
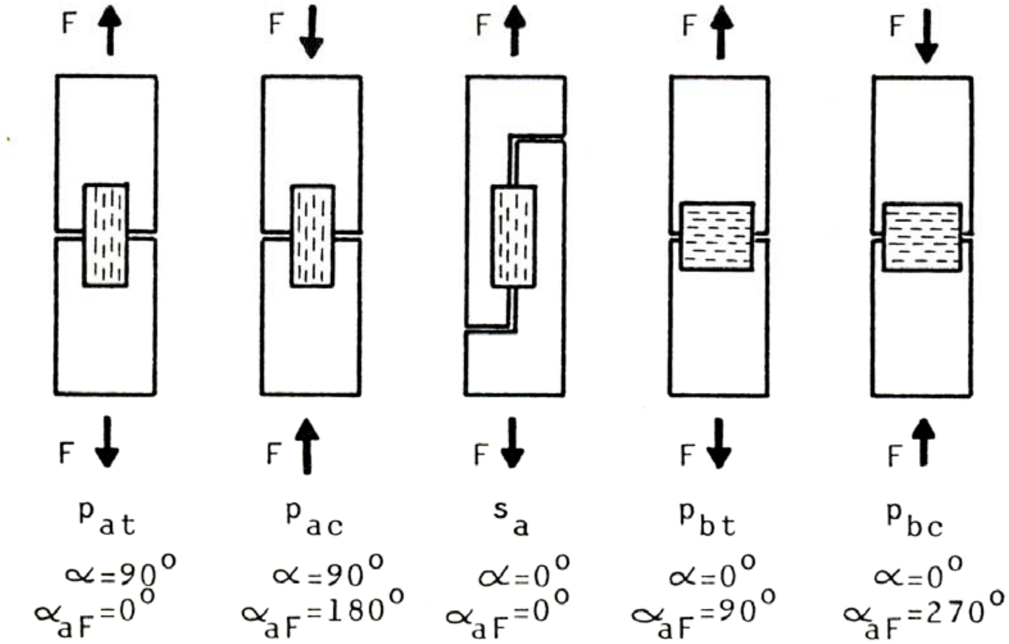


Figure 1. Example of nail plate joint. Existing design rules are drawn as a curve. Test results and common sense tell us that maximum strength is achieved for β = 60°.

Conclusion
The presented design method gives a good prediction of the plate strength of nail plates. The existing Nordic rules are partly unsafe and do not reflect the orthotropic behaviour of nail plates.

The amount of testing is considerably reduced – down to the 6 design values, see Fig. 2. Tests with skew plates often give confusing results – the need of these tests is eliminated.



19-7-7 B Norén
Design of joints with nail plates

Introduction

The following is a proposal for Annex to the CIB Structural Timber Design Code. The principles agree with those applied in **Paper 14-7-1**. That paper is referred to as the main paper.

21-7-3 T Poutanen
Nail plate joints under shear loading

Abstract

Some (15) shear tests with nail plate joints were conducted. The new thing was that the stress distribution (i.e. the moment distribution) was measured. It was found that the joint behaviour changes considerably if the plate has a plastic deformation. The present nail plate design and testing allows and utilizes steel plasticity: The characteristic values of the nail plate are defined after the steel plasticity limit and the design assumption for the stress distribution in the joint is assumed to be a plastic one. This brings some advantages e.g. the design values for shear are high and it is believed that the design is simple. But there are disadvantages too: the calculation actually becomes more complicated because the analysis model should take into consideration the degree of plasticity (or if plate plasticity is not considered the calculation is inaccurate and even unsafe); further the plasticity (at least in some common cases in practice) leads to an unfavorable stress distribution and to an excess timber volume.

The paper also considers two phenomena in a nail plate joint: lock action and gap constraint action and concludes that the lock action cannot be utilized in spite of its potential benefits and the gap constraint action is mainly harmful but in some cases it can be used for useful purposes.

16. In the author's opinion it is very obvious that linear joint behaviour leads to a better total reliability, quality and economy than the present concept based on the strength criterion and plastic joints.

Conclusions

1. Steel plasticity in the nail plate changes the joint behaviour considerably and according to the test the eccentricity increased on average by 16 %. This produces an excess load increasing the csi by app. 50 %.
2. Lock action brings many benefits to the nail plate joint but it apparently cannot be utilized in practice due to the requirement of a small gap or a big initial slip.
3. Gap constraint action is mainly harmful in nail plate joints but it can be utilized in some cases.
4. If steel plasticity occurs in a nail plate joint the degree of plasticity should be taken into consideration and if not the calculation includes a considerable error.
5. Nail plate tests should be conducted measuring the steel plasticity limit and the characteristic values of the plate should be fixed to this limit.
6. Nail plate shear tests should be conducted without contact.

24-7-1 B Källsner, J Kangas
Theoretical and experimental tension and shear capacity of nail plate connections

Background
The strength of nail plates with respect to the plate material has been studied at the Norweigan Institute of Wood Technology. This work served as a basis for a Nordic proposal for a design method and was presented by Norén in 1981.

In 1985 Bovim and Aasheim presented **Paper 18-7-6** where they made comparisons between measured and calculated values of plate strength. For all the tests a Gang Nail 18 plate was used. Their conclusion was that the design method presented gave a good prediction of the plate strength of nail plates.

During 1980-81 the Technical Research Centre of Finland (VTT) made an extensive investigation
of one type of nail plate to achieve an approval using different shapes and sizes of the nail plates.

Purpose and scope
At the Swedish Institute for Wood Technology Research the design method has been used for several years in connection with the evaluation of results from testing of nail plate connections. Since it is possible to derive rather simple expressions for the tension and the shear capacity one purpose with this paper is to present these expressions and to explain where they are valid.

It has often been questioned if the proposed design method can be used for any nail plate. In order to elucidate this, the results from testing of 6 different nail plates are compared with the theory. All the tests have been carried out at the Technical Research Centre of Finland.

Conclusions
The proposed design code is presented in a format easily applicable to results from standard tests. An evaluation of 6 different types of nail plates is made. The agreement between the measured and the theoretical capacity values in tension is somewhat varying but mostly rather good. The capacity values from the shear tests are often higher than the results from a theoretical calculation assuming no contact between the timber members. Special attention should be paid to the local buckling mode of the nail plate edge in the shear test. In the case of tension shear it might be possible to use a theory based on the assumption that there is contact but no friction between the timber members if the length-to-width ratio of the plate is high enough.

31-7-5 A Kevarinmäki
Timber contact in chord splices of nail plate structures

Introduction
In Eurocode 5 the contact pressure between timber members may be taken into account to reduce the joint force F in compression provided that the gap between the members has an average value not greater than 1 mm and a maximum value of 2 mm. In such cases the joint should be designed for a minimum compression force of $F/2$. However, EC 5 does not give any rules for utilization of the timber contact in the bending of chord splices.

In the Nordic countries the moment capacity of nail plate joints of chord splices with utilization of the timber contact has been used in practical design since the 1970s. In compression the design value of the joint normal force is 1/3 of the applied normal force of the chord, i.e. the timber contact reduction is 2/3. The design equations are based on the force couples: the moment, M_d, is analyzed as a tension force of the plate and a compression force of the timber contact located at a distance of $h/6$ from the chord edge. The interaction between the normal force and the moment is taken into account by the bi-linear capacity curve. Theoretically the design method is clearly on the unsafe side with a big tension force, because when the whole cross-section of the timber member is in tension there is no joint contact and no force couple for the moment.

The aim of this study was to develop new design equations suitable for use with EC5 for the chord splices with symmetrically placed nail plates, when the main direction of the nail plate is combined with the grain direction. The work may be done by theoretical calculations using previously developed models of the moment capacity and rotational stiffness of nail plate joints. The proposals for utilization of the timber contact in the chord splices are verified with bending and eccentric tension tests on different kinds of nail plate joints.

Conclusions

In addition to the semi-rigid anchorage stiffness, also the stiffness of the plate in the joint line (plastification), the timber contact and the effect of joint gap on them could be taken into account already in the determination of the member forces and moments of the nail plate structure. However, this would lead to a heavy iterative design procedure and the influence of these factors on the member forces and moments would not be significant. But very big differences would be observed in the force and moment values acting on the plate if the analysis were done without taking account of the timber contact and the plate plastification (allowed buckling). The effects of these factors may be calculated theoretically with general plate and wood properties and with certain allowed joint gaps. Based on the results of the theoretical calculations it is possible to derive simplified equations for the determination of plate forces and moments.

The results of the analysis of the effect of the timber-to-timber contact in chord splices is shown and the general simplified method of determining the force and moment components acting oil the plate in both tension and compression loaded chord splices with the bending moment has been derived. The simplified design method developed was verified with the altogether 148 bending and eccentric tension tests of different kinds of nail plate joints. The simplified method corresponds well with the test results of anchorage failure, and it is on the safe side when the plate capacity of the joint line is the critical factor. The conservatism of the calculation method is due to the higher plate tension stresses developed in bending than determined in axial tension tests.

Eurocode 5 does not give any rules for the utilization of the timber contact in bending and, together with the weak compression strength of nail plates, this leads to a very conservative design results in bending joints. The failure loads of the bending tests were 2,4..5,3 times higher than the capacities calculated by EC5 with the mean strength values, and the difference is further emphasized in combined compression and bending. The very simple assumption that 50 % of the bending moment of joint transfers through the contact couple and 50 % by the moment stresses of the nail plate leads to much better results. However, EC5 is conservative also in combined tension and bending, if the bending moment is so high that there is contact pressure on the compression side of the joint, which has been shown also by the analysis of the eccentric tension test results of Wolfe (1990).

4.11 NON-METALLIC DOWELS

22-7-3 J Ehlbeck, O Eberhart
Design of double shear joints with non-metallic dowels. A proposal for a supplement of the design concept

Introduction

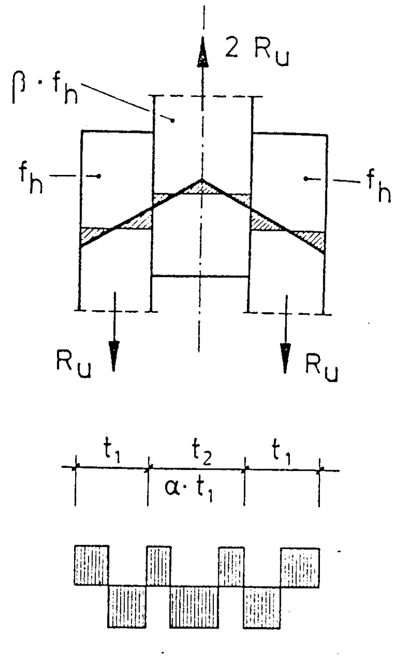
At the University of Karlsruhe a research programme was undertaken to determine the load-carrying capacity and the deformation behaviour of glulam joints with non-metallic dowels of resin-impregnated compressed wood. This material consists of multi-layered densified 2 mm beech veneers glued together and fully impregnated with phenolic resins. One of the objectives of this research work was to find out if Johansen's theory for determining the ultimate load-carrying capacity of joints with dowel-type fasteners can also be applied for such type of material having pronounced brittle properties.

The test data in principle confirmed the applicability of this design model which was introduced in the CIB-Code as well as the draft Eurocode 5 for limit state design calculations. It turned out, however, that an additional failure mode may occur which makes it advisable to introduce a supplementary design formula.

Conclusions

Tests with brittle non-metallic dowels in glulam joints proved an additional failure mode compared to those used in the "classic" Johansen theory which is -the basis for calculating the ultimate load-carrying capacities of joints with dowel-type fasteners in the CIB-Code and the draft Eurocode 5.

It is proposed to add to the design formulae another one describing this failure mode where there is only one brittle bending failure in the middle member. Furthermore, in case of brittle dowel materials it is proposed to substitute the yield strength by the bending strength of the material.



b)static model of the failure mode

32-7-11 R D Drake, M P Ansel, C J Mettem
Non-metallic, adhesiveless joints for timber structures

Abstract

The construction of wide-span timber structures is strongly dependent on the effective design of joints, which are usually fabricated from steel plates, bolts and dowels. Such joints tend to be heavy, costly, prone to corrosion and vulnerable to fire with poor aesthetic qualities. This paper investigates the potential for replacing steel fasteners with nonmetallic elements based on fibre-reinforced plastic pultrusions for the design of medium-to large-scale timber structures. Techniques have been developed for calculating the load bearing capacities of a range of non-metallic joints including stress analysis using finite element methods. Non-metallic joints for wood composites have been manufactured using friction-fitted, reinforced plastics in the form of dowels and plates. The reliability of these techniques has been confirmed by full scale laboratory testing of components and jointed structures under static and dynamic loads in double

shear, bending, tension and moment-resisting configurations.

Non-metallic fasteners and fittings for laminated veneer lumber (LVL) joints have been found to embody significant advantages compared with standard steel fasteners. The performance of glass-reinforced plastic (GRP), pultruded dowels and steel dowels have been compared and finite element- (FE-) modelled in double shear. Failure loads for GRP pultruded dowels are marginally higher than those for steel dowels and in the former case damage occurs in the dowel and the LVL rather than in the LVL alone. Moment-resisting joints based on friction-fitted, GRP dowels are capable of absorbing more energy than joints secured by steel dowels and this ductility is a key outcome of the research. Eurocode 5 design equations for steel dowels have been modified for pultruded GRP dowels and the concept of a cross breaking strength has been developed for design purposes.

Conclusions

The research programme has made a significant contribution to the advancement of better construction techniques for timber buildings by developing jointing techniques based on non-metallic connectors. Whilst these joints offer structural integrity, high aesthetic appeal, ease of manufacture, and low weight, their key attributes are excellent ductility, low cost (similar to steel per unit length) and commercial acceptability. The research has place these advances in a Eurocode 5 context (design modifications) and provided a foundation for current EPSRC- and EU-funded research into novel joints based on bonded-in pultruded GRP rods.

4.12 STATISTICAL TREATMENT

37-7-13 A Kevarinmäki
Behaviour of fasteners and glued-in rods produced from stainless steel

See: Glued-in rods

37-7-12 A J M Leijten, J Köhler, A Jorissen
Review of probability data for timber connections with dowel-type fasteners

Introduction
In the European design standard, Eurocode 5 (EN1995-1-1) the Johansen model is presented to predict the strength of connections with dowel-type fasteners. This model contains besides a number of geometrical parameters two material parameters; the embedment strength of the timber and the yield moment of the fastener. This study focuses on the main influencing independent parameters of the embedment strength being; the timber density and diameter of the fastener. The embedment strength expressions in Eurocode 5 are based on a comprehensive study by Whale and Smith (1986b) and Ehlbeck and Werner (1992). The influence of the timber density and the fastener diameter was derived using regression analyses. Expressions for the lower 5 %-Fractile value were assumed to be the same as for the mean value. This was achieved by simply exchanging in the regression formula the mean value of the density by the lower 5%- Fractile value of the density.

In the present study embedment test results from the above-mentioned research and test data from later investigations are considered to create a probabilistic framework for the prediction of the embedding strength. The presented framework is applied in a reliability analysis of a simple dowel connection. The lower 5%-Fractile of the embedding strength is introduced to characterize the evaluated probability distribution of the embedding strength. This information can be used to be incorporated in probabilistic design codes to feed design models of timber connections.

Conclusions regarding the results
From the evaluation of the analyses results it can be concluded that:

- Caused by differences in definition of the embedment strength perpendicular to grain a large portion of the available database was unsuitable for evaluation.
- A framework for the evaluation of the probability distribution of the embedding strength is presented for given 5%-Fractile value of the density and the diameter of the fastener is presented. In doing so it is distinguished between timber family, loading direction and fastener type.
- The 5%-Fractile value of the probability distribution is quantified to characterize the distribution.
- Applying the method in a reliability analysis with a simple connection, acceptable results for the failure probability can be obtained.

38-7-2 J Köhler
A probabilistic framework for the reliability assessment of connections with dowel-type fasteners

Introduction
For timber structures, the structural performance depends to a considerable part on the connections between different timber structural members; connections can govern the overall strength, serviceability and fire resistance. Assessments of timber structures damaged after extreme events as storms and earthquakes often point to inadequate connections as the primary cause of damage. Despite their importance codes and regulations for the design of timber connections, however, are not based on a consistent basis compared to the design regulations of timber structural components.

In the daily practice the engineering codes and regulations form the premises for the use of timber as a structural material. Code regulations in North America, Australia and Europe are based on the limit states design (LSD) approach which is implemented via load and resistance factor design (LRFD) formats. Originally, LRFD methods where converted as so called "soft conversions" of allowable stress design (ASD), the design method which was commonly used in code regulations before LRFD was introduced and which is usually based to a major part on experience, tradition and judgment. In the last decade this situation was changed by the use of structural reliability concepts as the basis of design of timber members. This progress was possible by means of collecting a huge amount of experimental data to characterize the statistical variation in timber material properties which facitilitates the formulation of probabilistic models for the behaviour of timber structural members. The resistance factors for members are calibrated employing probability based calibration techniques. Therefore the design of structural members is based on the same consistent foundation as for other building materials as concrete or steel. Codes for timber connections have been adjusted in recent years to reflect additional data and information on connections, but reliability based design for joints is not yet implemented in the code formats. LRFD for joints is in general still soft-converted from the traditional ASD format.

Explanations for this imbalance in advancement of design provisions for members and connections can be found in the relative simplicity of characterising mechanical behaviour of members, as compared to connections. A diversity of connections types is used in practise and these types have infinite variety in arrangement. This usually precludes the option of testing large numbers of replicas for a reliable quantification and verification of statistical and mechanical models.

For commonly used connections, a distinction is made between carpentry joints and mechanical joints that can be made from several types of fasteners. The mechanical joints are divided into two groups depending on how they transfer the forces between the connected members. The main group corresponds to the joints with dowel type fasteners. Connections with dowels, nails, screws and staples belong to this group. The second type includes connections with fasteners such as split-rings, shear-plates and punched metal plates in which the load transmission is primarily achieved by a large bearing area at the surface of the members.

In the course of this paper it is discussed how a probabilistic model for connections can be derived. It is focused on connections with dowel type fasteners, more precisely on timber to timber double shear parallel loaded connections with single dowel type fasteners.

Summary and conclusions

A probabilistic modelling framework for the load bearing capacity of single dowel type fastener connections is derived based on the design framework utilized in the present version of the Eurocode 5. Reliability analysis for connections with different fastener slenderness ratio is performed and the following conclusions can be drawn:

– The reliability index is an order of magnitude too high.
– System model assumptions have minor consequence on the result of the reliability calculation.
– The relative importance of the basic variables is quantified by sensitivity factors. The model uncertainty and the variable load are found to be most relevant.

Based on these findings the model uncertainty is quantified based on test data of connections. The following can be observed:

– The parameters of the model uncertainty are considerably different for cases where different Johansen failure modes are relevant.
– Especially the model bias is different for different failure modes.

Model verifications are proposed, whereas the introduction of an additional spitting mode and the utilisation of an alternative embedding strength model where derived independently of the considered data set. Friction factors for Johansen modes II and III are introduced. It is found that:

– A significant model improvement is reached by introducing the splitting mode.
– A minor improvement is reached by using the alternative embedding strength model.
– The improvement which is obtained by introducing friction factors has to be seen conditional on the data base which is considered.

43-7-1 T Tannert, T Vallée, F Lam
Probabilistic Capacity Prediction of Timber Joints under Brittle Failure Modes

See: Glued joints

43-21-1 J Munch-Andersen, J D Sørensen, F Sørensen
Estimation of load-bearing capacity of timber connections

Introduction

Experimental determination of the characteristic value of the properties used to calculate the load-bearing capacity of timber connections involves 3 steps. There is a standard which covers each step:

1. Selection of timber specimens (ISO 8970 = EN 28970)
2. Performing the tests according to the standard relevant for the property

3. Deriving the characteristic value (EN 14358)

This paper is discussing step 1 and 3 which are general for estimation of all properties. The standards are not referring to or in accordance with the principles in EN 1990, Basis of design.
Examples are given for withdrawal of threaded connector nails, where the tests should be carried out according to EN 1382. It is assumed that the withdrawal load-bearing capacity can be expressed as

$$F_{ax} = b\left(\rho/\rho_0\right)^c dl_{tr}$$

where

d	nail diameter
l_{thr}	length of the threaded part of the shank
ρ	timber density
ρ_0	a reference density
c	a power determining the dependency on the density
b	the withdrawal strength f_{ax} at density ρ_0

The parameters b and c should be estimated from the test results.

Conclusions
The present method for estimating strength properties of fasteners based on ISO 8970:1994 suffers from several drawbacks. It aims at determining the characteristic strength for timber with the characteristic density, which is too safe. For Method 1 it is quite difficult to find test specimens with sufficiently low density, and for Method 2 it is not clear which power should be used to correct for variations in density. Both methods are quite dependent on how the specimens are chosen, even though the standard gives no guidance except for the density.

The new ISO 8970:2010 focuses on determining the strength property at the mean value of the density but allows the test specimens to have the same density, so the influence of variation of the density is ignored. This causes unsafe values when estimating the characteristic strength as before using EN 14358.

Instead it is proposed to select test specimens so their densities are evenly distributed over the relevant range of densities, perhaps even requiring that timber from different sources is used. The observed values then have to be corrected to a reference density proposed as 420 kg/m^3 (mean value for C24). The correction should preferably take place using a power c fixed in a standard, but if c is to be determined from the observations, the estimate will be much better when the observations represent a wide range of densities. If fixed values of c are used, they shall of course be identical in the test standards and in Eurocode 5 when the strength for another strength class than C24 is determined by calculation.

There is no single safe choice for c. When shifting to a higher strength class a lower bound is the safe value, but when shifting to a lower class it should be the upper bound. That might be usilized when sufficient information to fix a single value is not available.

When a model for the mean value of the load bearing capacity is established, the parameters including the variation of the model error can be estimated using Annex D in EN 1990. The Annex also offers a method to include the effect of the natural variation of the density (and other parameters such as dimensional tolerances). Simplified equations are presented and their use illustrated for two test series with connector nails.

The examples suggest that the proposed method is quite robust but the selection of test specimens still is important. The characteristic withdrawal strength becomes higher when using the present method but lower if the new ISO 8970 is used together with EN 14358. This is desirable as the old and new ISO 8970 were believed to be too safe and unsafe, respectively.
It is demonstrated that a constant withdrawal strength f_{ax} for different nail lengths can be obtained only if it is determined using a threaded length not including the point. The tests also suggest that the minimum length of threaded nails required by Eurocode 5 should be taken as the real penetration length, not the threaded length as it might be read.

4.13 SCREWS

ESSAY 4.4 H J Larsen
Design rules for screws

A distinction is made between traditional smooth shank screws, where the outer thread diameter is equal to the shank diameter, see Figure 1, and ”modern” self-drilling screws, see Figure 2, with a geometry as shown in Figure 3.

Figure 1. Smooth shank screws. To the left: Lag screws. To the right slotted screw

Figure 2. Top: Self-drilling spun screw. Bottom: SFS- screw with thread both under the head and at the end, (the pitches are a little different resulting in the parts being drawn tight together).

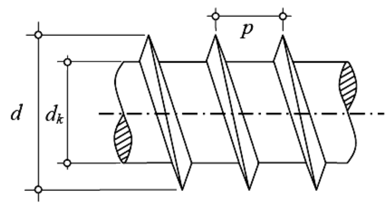


Figure 3. Detail of a screw thread: denotations nominal diameter d (= nominal screw size), thread-root diameter d_k and thread pitch p.

Laterally loaded screwed joints

The load-carrying for screws is assumed to be the same as for dowels, but with an effective diameter d_e, i.e. the characteristic embedding strength should be calculated as:

$$f_{h,k} = 0,082\rho_k d_e^{-0,3} \tag{1}$$

For smooth shank screws, the effective diameter d_e is taken as the smooth shank diameter. For other screws d_e should be taken as $1,1d_k$. Assuming $d_k/d = 0,65$:

$$f_{h,screw,k} = 0,082\rho_k\left(1,1d_e\right)^{-0,3} = 0,082\rho_k\left(1,1\cdot 0,65d\right)^{-0,3} = 0,091\rho_k d^{-0,3} \tag{2}$$

Tests to determine $f_{h,screw,k}$ are reported in Blass, H J, Bejtka, I and Uibel, T: “Tragfähigkeit von Verbindungen mit selbstbohrenden Holzscrauben mit Vollgewinde”, Karlsruhe Berichte zum Ingenieurholzbau, 2006.

Based on the tests the following expression for the embedding strength is proposed:

$$f_{h,screw,k} = 0,022\rho_k^{1,24}d^{-0,3} = 0,022\rho_k^{0,24}\rho_k d^{-0,3} = k\cdot\rho_k d^{-0,3} \tag{3}$$

The factor k is shown below as a function of the strength class/characteristic density.

	C14	C18	C24	C30
ρ_k kg/m^3	290	320	350	380
k	0,086	0,088	0,090	0,092

It is concluded that the Eurocode 5 simple rule seems reasonable.

Axially loaded screwed joints

In Eurocode 5:2004 the characteristic withdrawal load-carrying capacity at an angle α to the grain was originally given as

$$R_{ax,\alpha,k} = n_{ef}\left(\pi d l_{ef}\right)^{0,8} f_{ax,\alpha,k} \tag{4}$$

where

n_{ef} is the effective number of screws
d is the outer diameter measured on the threaded part
l_{ef} is the pointside penetration length of the threaded part minus one screw diameter

$f_{ax,\alpha k}$ is the characteristic withdrawal strength at an angle α to the grain.

The characteristic withdrawal strength at an angle α to the grain should be taken as:

$$f_{ax,\alpha,k} = \frac{f_{ax,k}}{\sin^2\alpha + 1,5\cos^2\alpha} \tag{5}$$

with

$$f_{ax,k} = 3,6 \cdot 10^{-3} \rho_k^{1,5} \tag{6}$$

ρ_k is the characteristic density in kg/m^3.

The background for these expressions, taken from DIN 1052:2004, is not given in any CIB W18-paper.

Without any explanation these rules were changed in the 2008 amendment A1 to Eurocode 5. For screws according to EN 14592 and with

6 mm $\leq d \leq$ 12 mm

$0,6 \leq d_k/d \leq 0,75$

where

d is the outer thread diameter;

d_k is the inner thread diameter

Eq. (4)-(6) were replaced by the following:

$$R_{ax,k} = \frac{n_{ef} f_{ax,k} d l_{ef} k_d}{1,2\cos^2\alpha + \sin^2\alpha} \tag{7}$$

where

$$f_{ax,k} = 0,52 d^{-0,5} l_{ef}^{-0,1} \rho_k^{0,8} \tag{8}$$

$$k_d = \min\begin{cases} d/8 \\ 1 \end{cases} \tag{9}$$

n_{ef} is the effective number of screws

l_{ef} is the penetration length of the threaded part, in mm

ρ_k is the characteristic density, in kg/m^3

The statistical treatment of the test results is described in **Paper 42-7-3** Models for the Calculation of the Withdrawal Capacity of Self-tapping Screws - M Frese, H J Blass.

It is assumed that the angle, β, between the screw axis and the grain direction, see Figure 4, is greater than 30º.

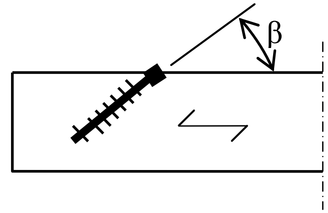


Figure 4. Angle between screw axis and grain direction.

For angles between 30º and 90º, $f_{ax,\alpha,k}$ should be multiplied by

$$k_\beta = \frac{1}{2,5\cos^2\beta + \sin^2\beta} \tag{10}$$

It is assumed that the penetration length is as a minimum 6*d*. For smaller lengths, Eurocode 5 gives no load-carrying capacity.

42-7-1 G Pirnbacher, R Brandner, G Schickhofer
Base Parameters of self-tapping Screws

Introduction

According to EN 1382:1999 the axial withdrawal strength of fasteners in timber is determined by means of a standardised test under tightly regulated loading conditions and exact rules for the moisture content of the specimens (thereby also limiting the possible climatic conditions during preparation and/or storage of the test pieces). The axial resistance is the primary mechanism defining connections that employ axially loaded screws as load carrying members – in general the dowel-type effect of the screw is not taken into account for the design of the connection. Variations of the moisture content, the temperature at "screw-in" and/or "pull-out" and wether the screw is pre-drilled or not are not considered in the design rules at all. Other parameters like the effective length and the angle between the screw axis and the grain are taken into account in the different rules available in the diverse codes like EN 1995-1-1:2004-1 and DIN 1052:2008 and technical approvals.

Conclusion

The parameters researched show significant effects on the withdrawal resistance of screws. Most prominent is the effect of embedment with an increase of at least 15% starting from only 15 mm embedment of the thread into the wood.

Additionally the effect of embedment covers other effects – e.g. the effect of the angle to the grain (earlier research without consideration of embedment indicated no effect between 90° and even below 45°). The inclusion of the embedment depth is proposed in the following form:

$$k_{emb} = 1{,}15 \text{ if } l_{emb} \geq 2d$$

The moisture content shows an influence of 0.65% per percent of moisture content. The
proposed correction takes the form of:

$$k_{mc} = \begin{cases} 1.00 & \text{for } 8\%\ m.c \leq 12\% \\ 1.00 - 0.0065 \cdot (u[\%] - 12) & \text{for } 12\%\ m.c \leq 20\% \end{cases}$$

The diameter has an effect of about -12.5% between the diameters of 8 mm and 12 mm. The relation included in the modelling of f_{ax} as function of density is:

$$k_{diam} = 2.44 d^{-0.428} \quad d \text{ in } [mm]$$

The length can be considered by deducting the tip from the thread length, where the correction is diretly related to the diameter and proposed in the form of:

$$l_{korr} = l_{thread} - k_{length} d \quad \text{with}$$

$$k_{length} = 1.17$$

The temperature does not exert a quantifiable effect on the withdrawal resistance.

$$k_{temp} = 1.00 \quad \text{for } -20^\circ \text{ to } 50^\circ\text{C}$$

A modified Hankinson relation optimized to describe the 5% fractile can be formulated by an adjustment of the exponents from the original value of 2.0 to 2.2 as follows:

$$k_{hankinson,modified} = \frac{1}{\sin(\alpha)^{2.2} + 1.30\cos(\alpha)^{2.2}}$$

The dependency from density shows clearly in the regression analysis. An example of the obtained mean value model shall be given here:

$$f_{ax,mean,90} = 0.01353\rho_{test} - 0.28147 \cdot 2.4d^{0.572} + 2.18888$$

with

$$\rho_{mean} = 450 \text{ kg/m}^3 \text{ and } d = 8 \text{ mm}$$

$$f_{ax,mean,90} = 0.01353 \cdot 450 - 0.28147 \cdot 2.44 \cdot 8^{0.572} + 2.18888$$

$$f_{ax,mean,90} = 6.09 - 2.26 + 2.18 = 6.02 \text{ N/mm}^2$$

The investigated effects show clear trends and can be normalized across material and angle to the grain variations. Summing all single effects up and considering observations during testing an optimized screw for load carrying joints with steel plates and screws under an angle of 45° can be derived. This screw includes a strengthened shaft of about 3*d* length under the head in order to minimize the curb effect of the steel plate's edge. Then an additional free length of about 1,5*d* is added to reach the optimal embedment depth of 2*d* below the surface. Finally a thread length of about 18-20*d* is applied to the screw. As lower bound 18*d* are suggested because the critical length that marks the transition of withdrawal failure to screw failure is at around 16*d* (depending on steel strength). If the effect of embedment is taken into account this length could even be reduced to 14 – 16*d*.

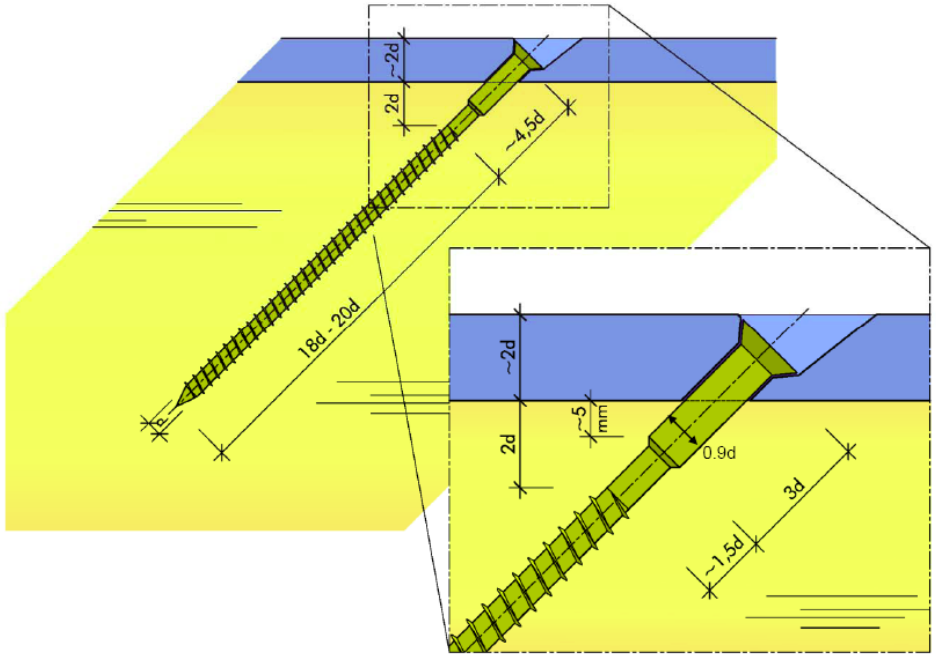


Figure 12: Optimized screw geometry for load carrying steel plate connections

Based on the research concerning moisture content and feedback from practical use it is proposed to limit screws to applications inside the service class 1 and 2.

42-7-2 H Krenn, G Schickhofer
Joints with inclined Screws and Steel Plates as outer Members

Introduction

The load-carrying capacity of connections with dowel-type fasteners and steel plates as outer member of a single shear connection, determined on the basis of Johansen's yield theory, is limited by the embedding strength of the timber member and the bending capacity of the fastener. The use of self-tapping screws with continuous threads with the screw axis perpendicular to the axis of the timber member is very inefficient because of the limited diameter and the relatively small bending capacity of the screws.

The application of inclined screws that are arranged under an angle of 45° between screw axis and member axis was presented by Bejtka & Blass as well as Kevarinmäki in 2002. They showed that the load-carrying capacity and the stiffness of timber-to-timber connections in these joints is a lot higher compared to common shear connections due to the high withdrawal capacity of the screws. Going one step further, the additional use of steel plates with special holes, incorporating the geometry of the screw head, leads to a further increase in load-carrying capacity and stiffness of this steel-to-timber connection. The ultimate load of these joints is then mainly limited by the withdrawal capacity and the tensile capacity of the screw as well as the friction between the steel plate and the timber member. These parameters are relatively well known, whereas the influence of the number of fasteners in such a connection is in question.

This paper will present the results of tests of joints with inclined screws in glued laminated .timber and steel plates as outer members. Some aspects of the influence of the number of fasteners will be discussed and a proposal for design rules for single-shear joints with inclined screws will be presented. General considerations about the joint geometry and the screw arrangement will will conclude the publication.

Conclusions

The test results of this study showed that steel-to-timber joints with inclined self-tapping screws yield a very high load-carrying capacity and an even more impressive stiffness. A simple truss-model, incorporating the axial resistance of the screw and the friction in the interface between timber member and steel plate, is sufficient for reliable design of these joints (although the dowel effect could be observed to some extent). In the ultimate limit state, the effective number of screws shall be taken into account with $n_{ef} = 0.9 \cdot n \cdot m$ with the number of screws n in a row and the number of rows m. In the serviceability limit state, the group effect might be accounted for with $n_{ef,ser} = (n \cdot m)^{0.8}$. All of these results are only valid, if the personnel for the fabrication of these joints is well trained and the deviations from the plan are minimised.

For structural detailing, the geometry of the steel members (e. g. rigid end plate) shall be taken into account. Furthermore an overlap of the screws in the axis of the timber member of at least 4 *d* is essential for symmetrical tension joints – otherwise a tension failure perpendicular to the grain could occur. In addition, it is recommended to choose an embedment length of the threaded part in the wood with at least 16 - 20 *d*, so the withdrawal capacity exceeds the tensile capacity of the screw sufficiently, and the screws fail in tension. The width of the steel plates depends on the applied screws, in particular on the head diameter of the screws with a countersunk head. For a screw diameter of 8 mm, a width of the steel plate of 15 mm shall be the minimum.

42-7-3 M Frese, H. J Blass
Models for the calculation of the withdrawal capacity of self-tapping screws

Abstract
Equations to calculate the withdrawal capacity of self-tapping screws, arranged in softwood, were determined. Based on results of 1850 withdrawal tests and by means of a regression analysis one equation for the withdrawal resistance and two further equations for the withdrawal parameter (resistance over nominal screw diameter and penetration depth) were derived. Nominal screw diameter, penetration depth and characteristic density of the softwood used are the independent variables in the equations, which are valid for an angle from 45° to 90° between the screw axis and the grain.

The equation for the withdrawal resistance delivers, also in comparison with code rules, the most favourable values. It enables, more accurately than until now, the estimation of the withdrawal resistance for wood screws with geometrical properties similar to those of the screws used for the tests. Hence, the benefit of this study is the reduction in testing, which is currently necessary for technical approvals.

Conclusion
Based on results of about 1850 withdrawal tests with self-tapping screws, arranged in softwood, one equation for the withdrawal resistance and two further equations for the withdrawal parameter were derived. Nominal diameter, penetration depth and characteristic density of a strength class in EN 338 are the independent variables in the equation for the withdrawal resistance. In the equations for the withdrawal parameter the independent variables are nominal diameter and characteristic density. The equation for the withdrawal resistance delivers, compared with the equations for the withdrawal parameter, the most favourable values concerning the withdrawal capacity. In comparison with the equations in DIN 1052 and Eurocode 5 the values of this equation are on average 30% higher. Further criteria: The equation is valid for nominal diameters between 4 and 14 mm, for penetration depths between 20 and 140 mm and is applicable independent of the angle between 45° and 90°. The results of the withdrawal tests are based on different types of screws from various manufacturers, therefore the new equations are suitable to calculate the withdrawal capacity of standard self-tapping wood screws. The new equation for the withdrawal resistance increases the load-carrying capacity in a considerable way, with the consequence that the application of self-tapping wood screws for connections inserted at an angle α to grain might be even more attractive in the future.

42-7-7 D M. Carradine, M P Newcombe, A. H. Buchanan
Using Screws for Structural Applications in Laminated Veneer Lumber

Abstract
There is a large body of research currently underway in New Zealand investigating the behaviour and design of multi-storey and long-span structures utilizing laminated veneer lumber (LVL) as beam, column and wall components. Screws have significant potential for use as fasteners for a myriad of connections throughout these buildings. While considerable research has been conducted on the use of screws in solid timber and glue laminated timber (glulam), there is currently a lack of data on the behaviour of screws when used with LVL. In many cases screws are installed parallel to the laminations (and glue-lines) and subjected to withdrawal loading, so it is necessary to determine appropriate configurations of screws that can be safely used in these situations. Monotonic testing on double shear screwed connections in LVL has been performed on specimens with varying screw configurations, LVL thickness and member depth. Direct withdrawal tests of screws installed into the edge grain of LVL, parallel to the glue lines were conducted using varying screw penetration depth, screw spacing and numbers of screws. Comparisons are drawn between existing standards for determining screw connection capacity typically used for solid timber and glue laminated material and the

capacities of the screwed connections in LVL. Recommendations are made for calculating LVL connection strength using screws as well as the limitations of existing design code predictions when using screws installed parallel to glue lines in LVL.

Conclusions

Experimental testing of screwed connections in LVL designed for rocking timber building systems has shown that design methods in NZS 3603 (SNZ, 1993) provide acceptable predictions of connection failure when loaded vertically. Subsequent testing on a 2/3 scale 2-storey demonstration building at the University of Canterbury will assess the lateral load behaviour of the screwed connections and provide data for the design of these structures. NZS 3603 provided conservative values for these connections due to inherent uncertainty of the fastening method reflected in the standard as well as the assumptions that the designers made in order to predict the behaviour of connections that have never been tested or used in practice. Development of connections similar to these and also for gravity resisting systems that do not require the ability to rock are currently underway and will be presented in future papers.

Direct withdrawal testing of self-drilling 14 gauge Type 17 screws installed in the edge of LVL members parallel to the glue lines provided data indicating that Eurocode 5 (EC5, 1994) and NZS 3603 (SNZ, 1993) predictions of withdrawal strength tended to be unconservative particularly when more than a single screw was installed in a row along the length of the LVL. It was observed that a possible failure mode where a plug of timber between the screws was removed as withdrawal loads were applied is a concern for designers and needs to be investigated because the spacing requirements from both model building codes did not keep this from occurring. Designers should exercise caution when designing these types of connections unless very conservative assumptions are used for the distribution of forces to screws.

4.14 SLOTTED-IN STEEL PLATES

30-7-3 E Aasheim
Cyclic testing of joints with dowels and slotted-in steel plates

Background and objective
In this paper tests performed in the project "Joints for Timber Bridges" are described. This project is a part of the "Nordic Timber Bridge Program".

The objectives of the project were to evaluate and develop mechanical fasteners with regard to dynamic actions and moisture conditions. Different types of fasteners were considered, and it was soon clear that the project had to be limited to one main type of joints. The project team chose to concentrate the work to testing of joints with steel dowels and slotted-in steel plates exposed to dynamic loads. The main reasons for this were that these joints are heavily used for timber bridges and that this type of joints is typical for all kinds of dowel type fasteners.

Few test results concerning dynamic testing, of joints are available. In the draft for Eurocode 5 - Design of timber structures - Part 2: Bridges, $k_{fatigue}$ for joints are introduced. $k_{fatigue}$ is a factor to transform characteristic short term static strength to characteristic strength for dynamic loads, $k_{fatigue}$. The aim of the project was to obtain more detailed information about $k_{fatigue}$ than those indicated in Eurocode 5. By studying draft Eurocodes and national design rules, the maximum number of load cycles during testing was set to 10^7. The tests were performed to determine the load level where the dynamic loading resulted in rupture at this number of cycles.

Recommendations and conclusions.
No guidelines for fatigue testing of timber joints are found in Eurocode 5 or in other documents known to the project team. The tests described in this report may be regarded as a first proposal from the project team about how to test and evaluate joints with regard fatigue actions. The experience gained during the testing shows that many factors should be discussed and that further research and tests are desirable. Some of the factors that are not investigated in this project are:

- Different failure modes
- The difference between reversed and non-reversed cycling
- The influence of the steel quality of the dowels
- Other dowel diameters.
- Other angles between the load and the direction of the grain
- The effect of several dowels in line with the load direction
- The effect of the frequency of the load cycles
- The effect of temperature and moisture content in the specimen.

The load levels in the range from 10^6 to 10^7 load cycles are slightly higher than $k_{fatigue} = 0,25$ from Eurocode 5. In other words, Eurocode 5 seems to give values on the safe side.

30-7-4 E Gehri
A steel-to-timber dowelled joint of high performance in combination with a high strength wood composite (parallam)

Introduction
A lot of research work has been done and is still going on for the development of high strength wood composites like Parallam. The main problem consists now in the transfer of larger forces in such materials, i.e. the development of adequate connections of high performance and reliability. The Blumer system (BSB), a steel-to-timber dowelled joint, developed and mainly used for connections in glued laminated timber of spruce, showed to be also adequate with Parallam.

BSB-connections
Two types of BSB-connections were used (see figure 1): with 3 and 4 steel plates. The number of dowels in a row varied between 3 and 5. Besides the normal BSB-dowel (diameter 6,25 mm and nominal tensile strength of 550 N/mm^2) a serie was made with diameter 6,0 mm and ETG-steel (nominal tensile strength of 1000 N/mm^2).

Conclusions
The BSB connections are meanwhile (the tests with Parallam were made in 1991) made with a distance of the steel parts of 60 mm. Using normal BSB-dowels a slightly higher load-carrying capacity is achieved.

The connection length is – independent of the section – when using 4 dowels in a row only about 220 mm, which is a very short connection.

The reliability is high; the variation of the connection load-carrying capacity is small, since the failure is connected to the formation of plastic hinges in the steel dowels.

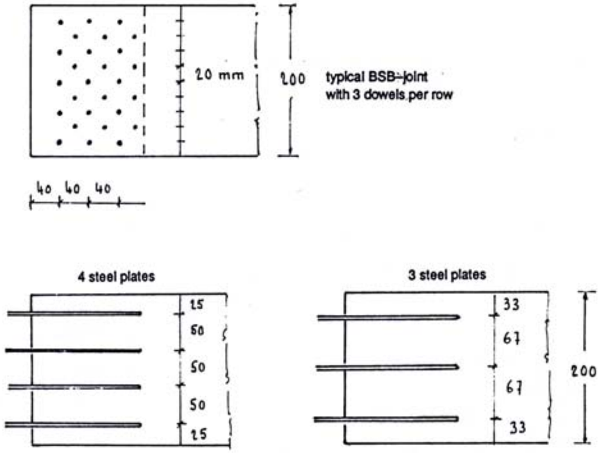


Figure 1: Type of BSB–connections used

The performance of the connected wood composite is high. Based on the net section (for 3 steel plates about 2/3 of the total section) the tensile strength of Parallam (f_t = 45 N/mm^2) has been reached. Therefore the performance of the joint – based on the total section – reaches about 2/3.

The group effect is directly considered, since the test were made with full size joints and corresponding high number of dowels (about 36 dowels) or more than 200 shear planes per joint.

32-7-8 M U Pedersen, C O Clorius, L Damkilde, P Hoffmeyer, L Eskildsen
Dowel type connections with slotted-in steel plates

Introduction

In the Eurocode the strength of dowel type connectors is determined according to the theory of plasticity. When the loading is in the grain direction the strength is well predicted by the plasticity theory. However, when the loading is in the transverse direction splitting may supervene plastic failure, as is shown in test series on smaller specimens with slotted-in steel plates. The scope of the present investigation is to trace the effect of eccentric transverse loading of full scale connections with slotted-in steel plates primarily loaded in the grain direction.

Conclusion

In order to perform a better modelling of the results laid forward the authors would like to state and discuss the following points:

1. Plastic failure criteria can successfully be applied for dominating axial load and a 10% additional frictional load bearing capacity before final splitting is obtainable.
2. Plastic failure criteria are less successful for dominating transverse load and utilising 10% additional frictional load bearing capacity seems to be optimistic.
3. The mechanical part of the plastic model may need refinement to include eccentricity introduced in the slot in order to model plastic failure accurately.
4. The material part of the plastic model may need refinement to include strain hardening for fh,90. This may explain why the severest loaded dowels in an eccentric loaded joint locally introduces splitting stresses in excess of what is expected by a perfect plastic model.
5. Dowel holes have to be tight fitting when the failure mode utilises rotational restraints of the outer part of the dowel.
6. No experimental evidence is found to substantiate the apparently sound idea of increasing the non-plastic capacity by prescribing slender dowels.

38-7-7 B Murty, I Smith, A Asiz
Design of timber connections with slotted-in steel plates and small diameter steel tube fasteners

Introduction

Finding combinations of member materials and fasteners that produce ductile timber connection responses is challenging and research toward that end is being conducted by the authors. Others have pursued the same aim but their techniques resulted in solutions that are labour intensive and also expensive in other ways, e.g. involving localised reinforcement of the members and grouting the fasteners in with epoxy. The approach taken here is to use 3 mm thick steel plate link-elements that slot into ends of wood members and circular cross-section steel tubes of relatively small external diameter (up to 12.7 mm), Figure 1. Solid spruce and Laminated Strand Lumber (LSL) were used as representative diverse 'wood' member materials. LSL is an advanced type of engineered wood product common in North America. While spruce is splitting prone, LSL is not. Tested connection arrangements had one or two slotted-in link-elements and one or four tube fasteners. Specimens were subjected to axial tensile static load until failure. Ductile load-deformation responses were expected to occur through combinations of wood crushing beneath fasteners, bending induced plastic hinges in fasteners, and plastic distortion of fastener cross-sections. Distortion of fastener cross-sections was the result of them being hollow, with the extent of those distortions controlled by the ratio of inner and outer tube diameters and the yield stress. Especially in cases where the fasteners were not slender, that mechanism compensated for loss of the other sources of ductility.

The remainder of this paper is focussed on test and results, examination of the acceptability of closed form Johansen type yield models for design level predictions of strengths of connections with small diameter steel tube fasteners, and the format of design rules. Yield models, or European Yield Models (EYM) as North Americans like to call them, are very widely accepted for making strength predictions for connections with dowel fasteners. However, EYM as currently implemented in many national and model international timber design codes, can over predict a connection's capacity by a considerable margin. Errors occur mostly when failure is due to splitting of the wood member(s), rather than creation of plastic hinges in fasteners and localised crushing of members by fasteners. Further, it is erroneous to suppose that if codes specify use of large fastener spacings and large fastener end distances then splitting of members can be reliably avoided.

Conclusions

Small diameter steel tube fasteners are an effective means of achieving strong and ductile structural wood connections. This is especially true if steel tube fasteners are used in conjunction with slotted-in steel plate link-elements, and join members manufactured from one of the newer generation of engineered wood materials (e.g. Laminated Strand Lumber).

4.15 SPACING

35-7-5 M Schmid, R Frason, H J Blass
Effect of distances, spacing and number of dowels in a row and the load-carrying capacity of connections with dowels failing by splitting

Introduction
Joints in timber structures often fail in one of the two brittle modes shown in figure 1'

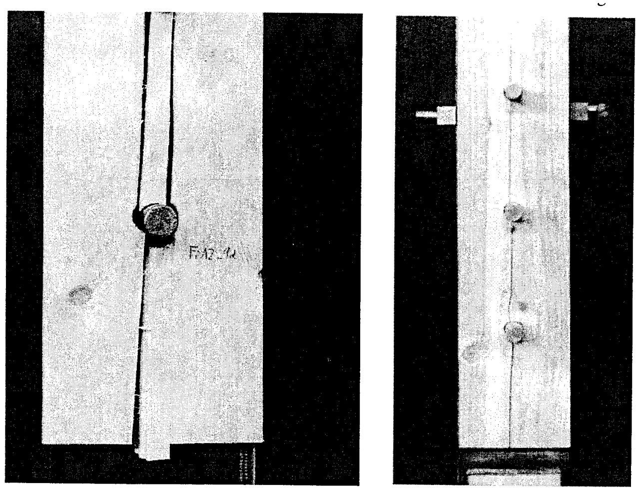


Fig. 1: plug shear failure   splitting failure

In order to avoid these brittle failure modes, most timber design codes contain rules based on the experience of craftsmen and results of connection tests in laboratories. These rules mostly consist of prescribed minimum dimensions, such as fastener end and edge distances, fastener spacing, or timber thickness. Regarding these minimum dimensions, no distinction is made between different timber softwood species in many codes. Recent research results e. g. by Jorissen (1998) showed brittle failure modes also in cases where the minimum dimensions were respected. In order to study the influence of the timber species on the splitting tendency, a research project was carried out at Karlsruhe University.

As for economical reasons it is not possible to test all types of fastener using different species and different joint geometry, a mechanical model based on fracture mechanics was developed. In this paper the model for splitting, that was frequently observed in the tests performed both, by Blass and Schmid (2002) and Masuda (1998), is presented. In terms of fracture mechanics it is a mode I crack extension.

Conclusions
The influence of geometry and material properties on the splitting tendency in the connection area of timber members was studied using a fracture mechanics approach. Based on the results of this approach, the model developed by Jorissen (1998) was modified. The predictions of the load-carrying capacity of multiple fastener joints show a good agreement with the test results of Jorissen. The effect of joint geometry was also studied using the model. The major influencing parameter on the splitting tendency of timber in the connection area is the fastener spacing a, parallel to the grain, while a_{3t} and a_{4c}, are of minor influence for joints with more than one fastener. For similar geometry and the same fastener slenderness the absolute diameter has a significant influence as well. Joints, where failure mode 3 according to Johansen's yield theory governs the design should hardly fail by timber splitting.

38-7-6 A Kevarimäki
Nails in spruce – Splitting sensitivity, end grain joints and withdrawal strength

Introduction
The new Eurocode 5 (EN 1995-1-1:2004) allows the national choices for the rules for nails in end grain and for the rules of timber thickness and edge distance of nailed joints with the species sensitive to splitting. Spruce (Picea abies) is named as an example of species sensitive to splitting. Experimental research (VTT 2004) was done for the justification of these national choices and for the development of the next version of Eurocode 5.

The withdrawal strength of nails was also studied. In Finland, the withdrawal failures of machine driven nails have caused several collapsing of secondary structures of ceilings in public buildings.

Conclusions
Splitting sensitivity of spruce
The clause 8.3.1.2(7) of Eurocode 5 (EN 1995-1-1) given for timber of

species especially sensitive to splitting is recommended to omit at all or to limit only for the silver fir (Abies alba) and Douglas fir by National annexes. With the Nordic spruce, the allowed minimum thickness of timber for nailed connections without predrilled holes may be safely defined as for the common species of timber. For the next version of Eurocode 5, the clause 8.3.1.2(7) shall be corrected: spruce should be eliminated from the list of species of sensitive to splitting and the real splitting sensitivity of silver fir (Abies alba) and Douglas fir should be verified. The rules given for "timber of species especially sensitive to splitting" seems to be over-conservative: the minimum timber thickness or the minimum edge distance is required to double.

Nails in end grain

In National annexes, the clause 8.3.1.2(3) of Eurocode 5 (EN 1995-1-1), "Smooth nails in end grain should not be consider capable of transmitting lateral forces", should be recommended to replaced by clause 8.3.1.2(4). The clause 8.3.1.2(4) with the limitation of the load-carrying capacity of nails in end grain to 1/3 of the values for nails installed at right angles to the grain may be safely applied.

Long-term lateral load-carrying capacity tests of nails in end grain should be done for the future development of Eurocode 5. According to the results of short-term tests, the lateral load-carrying capacity for nails in end grain may be increased at least to 1/2 from 1/3 of the values for nails installed perpendicular to the grain. Also the exceptional and unclear rule for the restriction on use of smooth nails only for secondary structures given in EN 1995-11:2004 should be cancelled from the next versions of Eurocode 5.

Withdrawal strength

It is proposed that the test standard for withdrawal strength EN 1382:1999 is modified so that the conditioning of wood is done to RH85% before nailing and after it is conditioned to RH40% before testing at a temperature of 20°C. This requirement could also be mentioned in the product standard for nails EN 14592. In case the nail type specific withdrawal capacity is determined according to EN 1382:1999 conditioned to RH65%, the experimental withdrawal strength should be reduced by a factor of 0,4 for plain shank nails and at least by a factor of 0,7 for profiled nails.

The reduction of the short-term withdrawal strength due to the previous long-term loading should be taken into account in design. The next version of Eurocode 5 (EN 1995-1-1) could be supplemented for example as follows: In case the share of the permanent and long-term loads is over 1/3, the withdrawal capacity of nails is calculated with a modification factor k_{mod} equal or lower than 0,7.

42-7-8 E Gehri
Influence of fastener spacings on joint performance experimental results and codification

Introduction

Engineer's first aim is to connect timber pieces together in the most efficient way; that includes performance, economy and reliability. Starting from the known behaviour of a joint – for a given geometry and fasteners arrangement – the performance can be established in relation to load-carrying capacity, stiffness and ductility.

For the evaluation of the above mentioned properties more or less standardized procedures may be applied. Due to the complexity of such a joint (different parameters involved, different failure modes possible) and the lack of reliable strength models the analysis of the influence of fasteners (and of fasteners spacings) is here quite impossible.

Therefore evaluation of fasteners starts mostly from the behaviour of a single or individual fastener; in many cases test procedures are only directed to a single property like the embedding strength (case for dowel-type fasteners) or the withdrawal strength (case for axially loaded screws) for a determined species and wood density. In a second step the interaction in a multiple fastener connection is than considered.

Basically such derived values are only valid for the test geometry and configuration used. The

application for actual joint design may be problematic, if the behaviour of the fastener in a joint does not follow the assumed behaviour of an individual fastener.

The load-carrying capacities of fasteners are in the Codes generally given in function of fasteners spacings, edges and end distances. In certain cases they are furthermore linked to the number of fasteners acting in a row or in a group. The stiffness is implicitly assumed to be independent of fasteners spacing (which is not true). The important and safety relevant property ductility has even not found recognition in the Codes; fasteners spacings may highly affect the ductility of a joint.

The weak points in a timber structure are often the joints. Generally the joint presents a lower load-carrying capacity than the two parts to be jointed. Highest performance is linked to an undisturbed (smooth) flux of forces in direction of the fibres. This is achieved the nearest by finger-jointing. More indicated – since a rather smooth flux of forces is possible – are glued in rods. A near steady flow of shear forces is originated around the rod.

Conclusion
In view of the great importance of a correct formulation of the spacing requirements for the performance of connections with axially loaded screws and glued-in rods inserted parallel to the grain the actual Codes have to be adapted.

43-21-2 T Uibel, H J Blass
A new method to determine suitable spacings and distances for self-tapping screws

Introduction
In recent years self-tapping screws have been increasingly used for connections or reinforcements in timber engineering. Most self-tapping screws can be arranged maintaining only small spacings and distances without risking a consequential splitting failure of the timber member. To avoid significant crack growth and splitting failure minimum values for spacings, end and edge distances as well as for the corresponding minimum timber thickness have to be determined. These requirements are important for the design of joints with self-tapping screws and have to be defined in technical approvals or to be examined regarding structural design codes [1] [2]. Fig. 1 shows typical splitting failure due to too small spacings and distances. The determination of spacing, edge and end distance requirements for self-tapping screws requires numerous and comprehensive insertion tests. Yet the results of such tests cannot be transferred to other types of screws or even to screws of different diameter because of differences in shape or geometry. To reduce the effort of insertion tests a new method was developed which allows the
estimation of required spacings, distances and timber thickness.

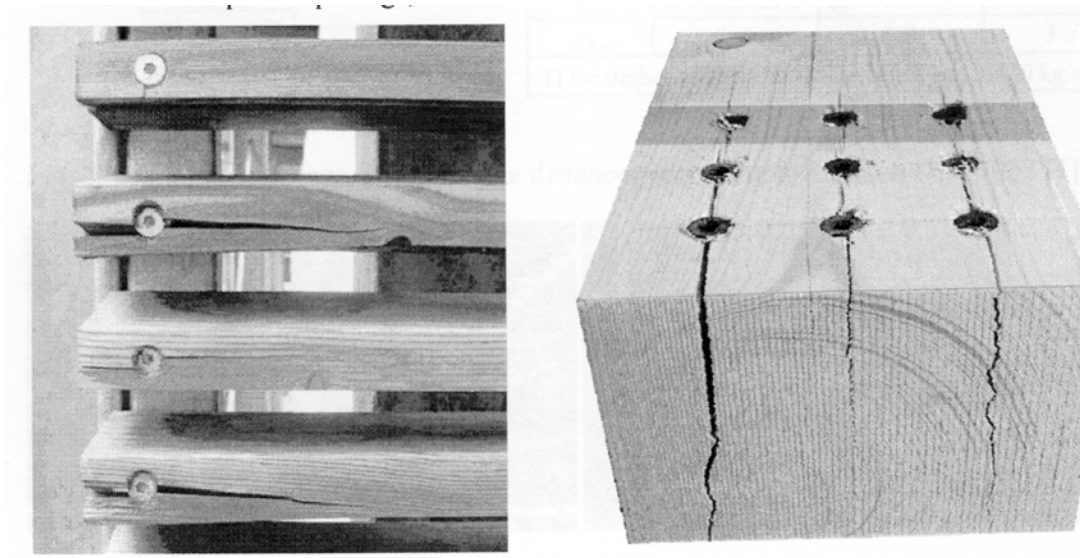


Fig. 1: Typical splitting failure caused by the insertion of self-tapping screws

Conclusions
To estimate the splitting behaviour of timber during the insertion process a new calculation method was developed. It represents a combination of a FE calculation and a new test method. The FE model allows the calculation of the resulting crack area for screws in different end distances as well as for different cross-sections of timber. In the Finite Element model the tensile strength perpendicular to the grain that represents a relevant factor for splitting was simulated by using non-linear spring-elements whose material behaviour was determined on the basis of tests using CT specimens. In order to determine the fastener-specific influences on the splitting behaviour a new test method was developed for measuring forces affecting the member perpendicular to the grain during the insertion process. This method also allows a direct evaluation of a screw's effect on the splitting behaviour by comparing it with the results of parallel tests involving reference screws whose influence on the splitting behaviour has already been established. For the calibration and verification of the model the crack area was visualized in insertion tests by means of dyeing the relevant areas. The simulated crack areas mostly proved to correspond with the test results.

Using the new method reduces the effort of conventional insertion tests and offers a basis for a realistic calculation of the load carrying capacity of joints in the case of failure by splitting.
In the continuation of the research project the parameters influencing the splitting behaviour like e.g. the angle between screw axis and tangent to the annual rings will be determined in greater detail. Besides the influences of the angle between screw axis and grain direction will be examined. In addition, parameters to facilitate the evaluation of the splitting behaviour and their limits (e.g. limits for the dimension of split areas) will be derived.

4.16 STIFFNESS

32-7-6 A Jorissen
The stiffness of multiple bolted connections

Introduction
The slip characteristics of multiple bolted connections were analysed experimentally and theoretically. It was found that the stiffness values obtained by tests were considerably lower than those suggested in ENV-1-1:1993 (Eurocode 5). This can be explained by the different hole clearances of each bolt. An alternative equation for the stiffness of multiple bolted connections is proposed in this paper based on about 850 short term tests on full-scale multiple bolted connections loaded parallel to the grain and on results obtained by a load distribution model.

Conclusions and recommendations
The research described in this paper show that the determination of the stiffness of multiple bolted connections according to ENV-1-1:1993 (Eurocode 5) result in too high values for this connection stiffness. On the other hand, the initial slip of the multiple bolted connections is negligible, even if the initial slip of the individual bolts is not. Therefore it is recommended to change the design rules for the stiffness of multiple bolted connections:

(1) the slip values should not be increased by 1 mm in order to take the hole clearance into account.
(2) the connection stiffness parameter can be calculated according to
$k_{ser} = k_{bolt} \frac{\rho_k^{1,5}}{20}$ with $k_{bolt} = 0,3$

32-7-9 J Vesa, A Kevarinmäki
Creep of nailplate reinforced bolt joints

Introduction
The effect of nail plate reinforcement on timber joints has been researched in Helsinki University of Technology since 1992. It was found that in short duration loading the reinforcement performs well: stiffness at low load levels is high, ultimate strength is high and the joint slip in failure is large. The presented creep tests were started to verify the function of the nail plate reinforced bolted joints under varying climate conditions during long period of time.

The long-term tests started at autumn 1992. A test of 44 nail plate reinforced bolt joints was carried out. In the test series the type of nail plates, the loading direction of the nail plate as well as the direction between grain and the applied force were varied. The test series consisted of three pairs: three series were tested in a covered but unheated building (a granary) and similar series in cyclically changed humidity and temperature conditions with rapidly changing humidity. The duration of the tests in cyclically changed conditions was 200 days and the tests in the natural covered conditions are still continuing after 6.5 years loading period.
The results of the measurements between the tests series from the natural and cyclically changed conditions were compared. The joint slip after 50 years loading of the tests pieces in the natural covered conditions was estimated with a logarithmic function. These results were compared to the joint slips according to EC5. Also the joint slip from other joint types of other previous tests found from literature was compared to the nail plate reinforced bolt joints and to joint slips according to EC5.

Conclusions
Nail plate reinforced bolt joints are very stiff compared to other joint types that can be assembled on building site. This is emphasized in longer time periods. The creep of nail plate reinforced bolt joints increases with the angle between force and nail plate main axis.

The calculation methods in EC5 give too small joint slips. This applies both to initial deformations and deformations after given time.

Joint slips after 200 days in the tests conducted under rapidly changing humidity are generally larger than those estimated to happen in naturally covered conditions after 50 years loading.

4.17 TRADITIONAL JOINTS

41-7-4 C Faye, P Gatcia, L Le Magorou, F Rouger
Mechanical behaviour of traditional timber connections: Proposals for design, based on experimental and numerical investigations. Part I: Birds mouth

Introduction
Due to the development of CNC (Computer Numerically Controlled) machines, traditional timber connections become economically competitive. Consequently, the use of these wooden connections in construction is increasing.

In this context, it is necessary to provide appropriate design rules taking account of the specific geometry of these connections.

Indeed, failure of the birds mouth connections occurs according to two phenomena:
- shear failure in the notch, for low skew angles. In this case, criteria using the average shear stress are not adequate because high peak stresses are not taken into account;
- failure in compression with an angle to grain in contact zones for higher skew angles. In this case, non linear distribution of the compression stress has to be determined.

In order to solve these issues, an experimental and numerical study on the instantaneous mechanical behavior of birdsmouth connections was investigated.

Summary
This paper describes:
1/ an experimental study: birdsmouth connections were tested for glued laminated beams in eight geometric configurations varying the values of skew angle 0, notch depth *t* and notch length *l*. The experimental and designed values were compared.
2/ a finite element modeling taking account of contact with friction between timber surfaces. The model allows the determination of shear and compression stresses with angle to grain profiles respectively in the notch and contact wooden zones.
3/ formulation of design proposals on the basis of experimental and numerical results.