New design model for brittle failure in the parallel-to-grain direction of timber connections with large diameter fasteners

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ABSTRACT

Timber connections may collapse in a brittle or in a ductile mode. The calculation models of timber joints are mainly focused on ductile failure mode, since it has been traditionally assumed that brittle failure was avoided by respecting a minimum spacing between fasteners. However, this assumption alone does not guarantee a ductile failure. This paper proposes a new design model dealing with brittle failure modes of timber connections with large diameter fasteners (those protruding the whole thickness of the timber member such as dowels or bolts). A comparison between the proposal and the existing models, using an extensive database of tests, is used to demonstrate the improved accuracy of the proposed design method.

1. Introduction

Environmental awareness, combined with politics that encourage the use of sustainable and renewable materials, clearly influence the building sector. The increased use of timber structures during the last years is thus expected to continue in the future.

Connections have a main role in the structural design, as they highly influence the behaviour, cost, and safety of structures. Connections were related with around 25% of recent collapses of timber structures, and around half of these cases involved connections with dowel-type fasteners [1,2].

A connection with dowel-type fasteners may fail in a ductile or a brittle manner. The European Yield Model (EYM), which is worldwide used to determine the capacity of a connection, considers only the ductile failure mode, namely embedment (Fig. 1a) and yielding of the fastener.

However, a timber connection may prematurely fail before achieving such ductile capacity, at a lower load, in a brittle manner: splitting (Fig. 1b), row shear (Fig. 1c), block shear (Fig. 1d) or net tension (Fig. 1e). Such early brittle failure may lead not only to a reduction of the intended capacity but, in a poorly designed building lacking robustness, to a possible sudden collapse of the building. However, as important as it may be, it is quite an unknown topic. As demonstrated by a survey made by the Working Group 3 of the Cost Action FP1402 [3], 30% of the participants (most of them European) ignored the existence of such kind of failure in connections.

Since reported in early works such as Fahlbusch [4], several authors have dealt with some or all of the possible brittle failure modes. For a detailed review, the reader is referred to Cabrero and Yurrita [5,6]. In the last 15 years, some of these models dealing with brittle failure have been included to a different extent in design standards, such as the Eurocode 5 [7], CSA Standard O86-09 [8] or the evolution of the latter one made by Quenneville and Zarnani [9] which will be included in the future chapter of the New Zealand standard dealing with connections.

This article presents a new design model for brittle failure of timber connections loaded in the parallel-to-grain direction with large diameter fasteners (such as dowels and bolts -defined in the Eurocode 5 [7] as the ones with a diameter larger than 6 mm- and which, by its configuration, usually protrude the whole timber thickness). The new model enables to calculate the brittle capacity of this kind of connections in a simple way, with an improved accuracy in comparison to the existing models.

This paper is organised as follows: Section 2 provides a brief description of the analysed type of structural connections, their brittle failure modes and existing models. Section 3 discusses the different assumptions and improvements of the new model in comparison to previous proposals. The full model is described in Section 4. Finally, in Section 5, an extensive data of experimental results is used to validate the new design model and compare its results with those obtained from other proposals.
2. State of the art

2.1. Geometry of the connection

Fig. 2 shows a typical configuration of a timber-to-steel connection (in this case a wood-steel-wood, wsw, joint with two shear planes) with large diameter fasteners, (in this case, dowels). The timber elements are defined by its main dimensions: width \(b\) and thickness \(t\) (\(t_2\) for inner timber members in multiple shear plane connections, \(msp\); the joint by its width \(b_c\) and length \(L_c\), and the steel plates by its thickness \(t_p\).

The connection depicted in Fig. 2 has a total of 12 fasteners with a diameter \(d\), which fit in pre-drilled holes of diameter \(d_0\) (which, in the case of bolts are oversized, \(d_0 = +d_{2m} mm\)), distributed in 3 rows in the parallel direction \((n_r)\) and 4 columns \((nc)\) in the perpendicular to grain direction. The spacing between columns is defined as \(a_1\); \(a_2\) is the distance between rows; \(a_3\) is the distance from the last column to the loaded edge and \(a_4\) is the distance from the outer rows to the lateral edge of the timber member.

<table>
<thead>
<tr>
<th>Nomenclature</th>
<th>(msp) Multiple shear plane connection</th>
<th>(ws) Wood-steel connection</th>
<th>(wsw) Wood-steel-wood connection</th>
<th>(ww) Wood-wood connection</th>
<th>(www) Wood-wood-wood connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower case</td>
<td>(a_1) Spacing between columns of fasteners</td>
<td>(a_2) Spacing between rows of fasteners</td>
<td>(a_3) Distance between the last column of fasteners and the the edge of the timber element</td>
<td>(a_4) Distance between the side row of fasteners and the edge of the timber element</td>
<td>(b) Width of a timber member</td>
</tr>
<tr>
<td>(b_{sw}) Width between the two lateral row of fasteners (gross and net distance, respectively)</td>
<td>(c) Rank correlation coefficient [42]</td>
<td></td>
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<tr>
<td>(d) Fastener diameter</td>
<td>(d_0) Diameter of the pre-drilled hole for a fastener</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>(f_s) Shear strength of a timber product</td>
<td>(f_{sh}, 0) Tensile strength in the parallel-to-grain direction of a timber product</td>
<td></td>
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<tr>
<td>(f_{t, 0}) Tensile strength in the parallel-to-grain direction of a timber product</td>
<td></td>
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</tr>
<tr>
<td>(k_1) Coefficient applied to the head tensile plane</td>
<td>(k_2) Coefficient applied to the lateral shear planes</td>
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<tr>
<td>(m) Slope of a linear fit passing through the origin of co-ordinates</td>
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</tr>
<tr>
<td>(n_c) Number of columns of fasteners</td>
<td>(n_{ef}) Effective number of fasteners</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(n_r) Number of rows of fasteners</td>
<td>(n_s) Number of shear planes of a connection</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>(sws) Steel-wood-steel connection</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(t) Thickness of the wood member</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>(t_1, t_2) Thickness of the side and middle timber member, respectively</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>(t_{ef}) Effective thickness of the connection</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(t_p) Steel plate thickness</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

\(AT_{b, ef}, AT_{n}\) Head tensile area related to block shear and net tension, respectively

\(BOEF\) Beam on elastic foundation

\(CCC\) Concordance correlation coefficient [43,41,44]

\(COV\) Coefficient of variation

\(EYM\) European Yield Model

\(F_{b}\) Brittle load capacity of a connection

\(F_p\) Theoretical load capacity of a connection predicted by a model

\(F_{b,hi}\) Brittle load capacity of a timber element for block shear

\(F_{b,li}\) Brittle load capacity of a connection

\(F_{b,ni, i}\) Brittle load capacity of a timber element for net tension

\(F_{b,ri}\) Brittle load capacity of a timber element for row shear

\(F_{l, i}\) Load capacity of a single lateral shear plane

\(F_{l, i}\) Load capacity of the head tensile plane

\(F_{l, i}\) Load capacity of the head tensile plane for net tension

\(L_{c}, L_{net}\) Length of the connection. Gross and net length, respectively

\(M_y\) Yield moment of the fastener

\(MRE\) Mean relative error

\(Q^2\) Coefficient of correlation [40,41]

\(R^2\) Coefficient of determination

\(SD\) Standard deviation of the mean relative error

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\(R^2\) Coefficient of determination

\(SD\) Standard deviation of the mean relative error

**Fig. 1.** Possible failure modes of a timber connection with dowel-type fasteners: embedment is the only ductile failure mode, the rest are brittle.

2. State of the art

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The loaded timber area $b \times L$ (Fig. 2a) is defined from $b_c = a_2(n_r - 1)$ and $L_c = a_1(n_r - 1) + a_3$, and the corresponding net dimensions (which consider also the protruded holes of diameter $d_0$), are $b_{net} = b_c - [(n_r - 1)d_0]$, and $L_{net} = L_c - [(n_r - 0.5)d_0]$.

2.2. Failure modes of connections loaded parallel to the grain

From the possible failure modes of a timber connection loaded parallel-to-grain with large diameter fasteners shown in Fig. 1, only embedment (Fig. 1a) is a ductile failure mode, as considered by the European Yield Model (EYM). The other possible failure modes, all of them brittle, are:

- Splitting (Fig. 1b): it is formed by a longitudinal crack along the row of fasteners. This failure might locally occur in the outer row, and the final global failure of a connection with multiple rows might not be related to it. It is, therefore, dismissed by authors such as Quenneville and Zarnani [9].
- Row shear (Fig. 1c): shear stresses generate two parallel cracks along each row of fasteners.
- Block shear (Fig. 1d): it is generated by the tear out of the loaded timber area, $b \times L_{c}$, defined by two lateral cracks along the exterior rows of fasteners and a head crack.
- Net tension (Fig. 1e): it is determined by a crack produced on the net cross sectional area at the beginning of the connection.

2.3. Existing brittle failure design models

Only three models consider somehow all the brittle failure modes in the parallel-to-grain direction: the Eurocode 5 [7], the model from Hanhijärvi and Kevärinniemi [10,11] and the one proposed by Quenneville and Zarnani [9]. However, as shown in previous works [12,5] they lack a consistent approach. A brief description of them is given below, and further details regarding the capacity of the involved failure planes will be described within Section 3. For a more detailed description and analysis, please refer to Yurrita and Cabrero [12].

Among the described brittle failure modes, the Eurocode 5 [7] only deals explicitly with block shear (Fig. 1d). The corresponding model is described in the informative Annex A, where the brittle capacity is obtained as the maximum of the capacities of the involved failure planes. Splitting (Fig. 1b) and row shear (Fig. 1c) are considered indirectly in the ductile calculation by introducing the factor $n_f$. This value, derived from the work developed by Yurrita and Cabrero [13], reduces the ductile capacity of the connection by reducing the number of considered fasteners. Even though net tension failure is not explicitly considered for connections, it is said in Section 6.1.2 [7] that the wood members must withstand the loads (tension in this case) at any point of them, and this should be also accomplished in the connection.

Hanhijärvi and Kevärinniemi [10,11] also consider all the possible failure modes (including splitting) by a comprehensive calculation model which accounts for all the possible failure modes at once. The capacity of the timber element is obtained by adding the minimum of the brittle failure planes or the ductile mechanism. In consequence, the obtained capacity might not reflect the actual failure mechanism.

The model from Quenneville and Zarnani [9], which is the proposal for the future standard from New Zealand, is an upgrade of the model included in the Canadian code CSA Standard O86-09 [8], originally derived from the work from Quenneville and Mohammad [14], and Mohammad and Quenneville [15]. The capacity for each brittle failure mode is obtained by adding the corresponding capacities of the
involved failure planes. Only splitting is not considered, as it is not an expected failure mode in a connection with two or more rows of fasteners.

3. Considerations for the new design model

3.1. Considered failure planes

All the existing models are intended for design purposes, as the one herein presented. Hence, they aim at a simple model which is easy to understand by the designer. For that reason, the brittle capacity is obtained from the capacity of those planes where the failure may take place, and which are clear to the practitioner: the shear lateral planes \( L \) (related to row shear -Fig. 3a- and block shear -Fig. 3b-) and the head tension plane \( H \) (related to block shear -Fig. 3b- and net tension -Fig. 3c-). Such capacity is obtained by multiplying their area by the corresponding strength, either tensile parallel-to-grain \( f_{\theta}^{L} \) or shear \( f_{S}^{L} \). This section presents the different assumptions for the consideration of the capacity of these planes, discusses their adequacy and presents how they are considered in the new proposal.

3.1.1. Head tensile planes

All the models consider the net area for these tensile planes \( H \) (Fig. 3), as described below. No modifications are introduced in the proposal.

**Head tensile plane for block shear.** The area of the head tensile plane related to block shear \( A_{T,b} \) is the net area comprised within the two lateral shear planes of the outer rows of fasteners, where the pre-drilled holes are not considered:

\[
A_{T,b} = (a_{2} - d_{0})(n_{c} - 1)t
\]  

(1)

Head tensile plane for net tension.

The area of the head tensile plane (or net cross-section) related to net tension \( A_{T,n} \) is the result of discounting the pre-drilled holes to the gross cross-sectional area of the timber element:

\[
A_{T,n} = (b - d_{0}n_{c})t
\]  

(2)

3.1.2. Lateral shear planes

The existing models differ on how to consider these lateral planes \( L \), which are activated in block shear (Fig. 3b, two planes) and in row shear (Fig. 3a, two planes per each row of fasteners). They assume different length and width, this latter related to the effective thickness of the timber member.

The number of considered lateral shear planes depends on the failure mode: two shear planes for block shear (Fig. 3b) or two shear planes per each row of fasteners in the case of row shear (Fig. 3a).

Length of lateral shear planes.

The models from Hanhijärvi and Kevarinmäki [10,11] and Eurocode 5 [7] consider the total length of the connection, \( L_{c} = a_{2}(n_{c} - 1) + a_{1} \). On the other hand, Quenneville and Zarnani [9] propose a more conservative value, \( L_{c} = n_{c}\min(a_{2}, a_{1}) \), obtained by multiplying the number of columns of fasteners by the minimum value between the parallel spacings, \( a_{2} \) and \( a_{1} \).

Experimental results suggest that considering the whole length of the connection might closer reflect the actual response. Fig. 4 shows two different connections tested by Yurrita et al. [12], in which the \( a_{1} \) distance was the only difference between them: \( LVLb3.a3 \) had \( a_{1} = 3d = 36 \text{ mm} \) (Fig. 4a), while \( LVLb9.a3 \) had \( a_{1} = 9d = 108 \text{ mm} \) (Fig. 4c).

Fig. 4b and d show the load-slip behaviour of the performed tests. As a reference, the horizontal dashed line shows the theoretical load capacity, 159.20 kN, predicted by Quenneville and Zarnani [9]. It is the same for both configurations, because it is obtained from the minimum between \( a_{2} \) and \( a_{1} \), which in both cases is \( a_{1} = 36 \text{ mm} \).

However, the test results show an increased capacity in those connections with higher end distance. The \( LVLb3.a3 \) configuration reaches an average maximum load of 131.50 kN while the \( LVLb9.a3 \) obtains a mean capacity of 191.68 kN, 45% higher. The difference of length between both configurations is around 35%, similar to the observed increase in load capacity.

It may be concluded that those models which consider the whole length of the lateral plane more closely reflect the actual behaviour of the connection. Therefore, the considered length of the connection in the new proposal is defined as \( L_{c} = a_{2}(n_{c} - 1) + a_{1} \).

Width of the lateral planes: effective thickness.

The width of the lateral shear planes is usually defined by means of an effective thickness of the timber element, which allows for a simple way to consider the non-homogeneous stress distribution along the fastener and the resulting deformation.

The Eurocode 5 [7] bases its proposal in the yielding mechanism of the fastener. However, brittle failure may happen before yielding [16]. On the other hand, Quenneville and Zarnani [9] propose to determine the effective thickness based only on the position of the wood member in the connection: outer members can be found in wood-steel (ws), wood-steel-wood (www), and the exterior members of multiple shear plane (msp, Fig. 5d) connections. Inner timber members of a connection are used in steel-wood-steel (sws, Fig. 5c) connection and also as interior elements of msp joints. Depending on their relative position, they propose \( t_{f} = 0.65t \) and \( t_{f} = 1.0t \) for outer and inner timber members, respectively. Hanhijärvi and Kevarinmäki [10,11] also consider the relative position of the timber member, but additionally take into account geometrical and material properties of the fastener and the timber product.

In a previous study, Yurrita and Cabrero [16] analysed this parameter, and developed a new approach to determine it. The new proposal is based on the elastic deformation of the fastener, modelled by means of a beam-on-elastic-foundation. Only the slenderness \( t/d \) of the fastener (ratio between timber thickness \( t \) and fastener diameter \( d \)) is considered to be the main influencing parameter. The following expressions for thin \( (t_{f} \leq 0.5d) \) and thick \( (t_{f} > 1.0d) \) steel plates were developed by Yurrita and Cabrero [16]. The distinction made in the equations below between thin and thick steel plates is taken from the Eurocode 5 [7], which additionally suggests an interpolation of the obtained value for intermediate cases.

![Fig. 3. Loading planes (lateral shear \( L \), and head tensile \( H \)) related to each failure mode.](image-url)
Fig. 4. Geometry and test results of the LVLb3.a3 and LVLb9.a3 test series performed by Yurrita et al. [12].

Fig. 5. Possible configurations of a timber connection combined with steel plates.
3.2. Coefficients associated to each failure plane

All the existing models use numerical coefficients to modify the capacity of both failure planes. These parameters hide several assumptions which simplify the different complex phenomena involved in the connection behaviour, such as non-homogeneous stress distribution, stress interaction, or the probability of defects in the stressed timber volumes.

An overview of these different values is given in Table 1. The capacity of the lateral planes is reduced to around 70%, while that of the head tensile plane is increased.

The Eurocode 5 [7] and Quenneville and Zarnani [9] propose a fixed numerical value in both planes, while Hanhijärvi and Kevarinmäki [10,11] proposed different coefficients for different timber products.

Following this latter model, the proposal presents coefficients which are different for each timber product, and defined by the corresponding modulus of elasticity parallel-to-grain and the shear modulus (Table 1). This $E_0/G$ ratio allows for a simple way to additionally consider the interaction between both types of stresses, as pointed out by Hanhijärvi and Kevarinmäki [11] and Sjödin and Johansson [17]. This approach will be validated below in Fig. 10 and the related discussion.

3.3. Determination of the brittle capacity from the failure planes

Once the capacity of the different failure planes is calculated, the total brittle capacity of the timber member must be obtained. Most of the proposals share the same principle, that is, the brittle capacity is obtained as the sum of the capacities of the involved planes. Only the Eurocode 5 [7] proposes a more conservative approach, in which the total load capacity is determined as the maximum between the capacity of the lateral shear planes and the head tensile plane. This latter procedure has been shown to be excessively conservative, and it is not related to the failure observed in the experimental tests [12]. Therefore, the new proposal proposes the addition of the capacities of the lateral shear and head tensile planes to obtain the brittle load capacity for block shear.

3.4. Considerations for the case of multiple shear planes connections

The models for brittle failure, including the new proposal, provide the load capacity per timber element of the connection. Most of the connections have one (wood-steel, Fig. 5a) or two shear planes, (wood-steel-wood, Fig. 5b, and steel-wood-steel, Fig. 5c). In these cases, the total load capacity of the connection will be the sum of those of the timber members which share the load (1 element in the cases of Fig. 5a and c, or 2 symmetrical members for Fig. 5b).

However, in practice, connections with multiple shear planes are also used (Fig. 5d shows an example of a connection with four shear planes). In these cases, the simple addition of the load capacity of each timber member might not be adequate, since different timber sections might be used, and they therefore might not be simultaneously loaded at the same rate. As previously explained by Yurrita and Cabrero [18], a stiffness-based model is more adequate. The total capacity of the connection $F_b$ can be then obtained from the load capacities $F_{b,1}$ and $F_{b,2}$ of the outer and inner timber members, respectively:

$$F_b = \min \left( F_{b,1}, F_{b,2} \right)$$

where $t_1$ and $t_2$ are the thickness of the outer and inner wood members of the connection, and $n_2$ is the number of inner timber members in the connection. This expression assumes that the two outer elements are equal, and same consideration is taken for connections with several inner members.

4. Summary of the new model proposal

4.1. Brittle capacity of the connection

The brittle load capacity $F_b$ of a connection may be obtained by adding the load reached by each of the timber members which share the load:

- Connections with one or two equal timber members (one or two shear planes):

$$F_b = n F_{b,1}$$

where $F_{b,1}$ is the capacity of a timber element and $n$ is the number of timber elements.

- Connections with more than two timber members (usually referred as multiple shear planes):

Table 1

<table>
<thead>
<tr>
<th>Model</th>
<th>$k_o$</th>
<th>$k_1$</th>
<th>$k_{n,t}$</th>
<th>Block shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eurocode 5 -Annex A- [7]</td>
<td>0.7</td>
<td>1.5</td>
<td>1</td>
<td>tension</td>
</tr>
<tr>
<td>Quenneville and Zarnani [9]</td>
<td>0.9</td>
<td>1.5</td>
<td>1</td>
<td>shear</td>
</tr>
<tr>
<td>Hanhijärvi and Kevarinmäki [10,11]</td>
<td>1</td>
<td>1.25</td>
<td>1</td>
<td>shear</td>
</tr>
<tr>
<td>Proposal</td>
<td>0.4 + 1.4 $\sqrt{2}$</td>
<td>0.9 + 1.4 $\sqrt{2}$</td>
<td>1</td>
<td>shear + tension</td>
</tr>
</tbody>
</table>

a Row shear is not considered in Eurocode 5 -Annex A- [7], so this coefficient is only applied to block shear.

b This is just a simplification, as this model is more complex and considers more interactions.
\[ F_F = \min \begin{cases} \frac{F_{B,1}}{2} + \frac{k_{1} n_{1}}{2} & \text{failure of outer members,} \\ \frac{F_{B,2}}{2} + \frac{k_{2} n_{2}}{2} & \text{failure of inner members;} \end{cases} \]  

where \( F_{B,1} \) and \( F_{B,2} \) are the capacities of the outer and inner timber members respectively, and \( n_{2} \) is the number of inner timber members.

### 4.2. Brittle load capacity of a timber member

The brittle capacity \( F_{B,i} \) of a single timber element \( i \) is obtained as the minimum of the capacities for the different possible failure modes.

The row shear capacity of a timber member \( F_{R,i} \) is determined by adding the capacity \( F_{L,i} \) of the two lateral shear failure planes generated by each row of fasteners:

\[ F_{R,i} = 2 n_{i} F_{L,i} \] (9)

where \( n_{i} \) is the number of rows and \( F_{L,i} \) is the shear capacity of the lateral plane given in (11).

The block shear capacity \( F_{B,bl,i} \) is defined by the onset of failure of two lateral shear planes and a head tensile plane, and it is therefore obtained by adding the corresponding capacities:

\[ F_{B,bl,i} = 2 F_{L,i} + F_{T,i} \] (10)

where \( F_{L,i} \) is the shear capacity of the lateral (longitudinal, \( L \)) shear plane given in (11) and \( F_{T,i} \) is the tension capacity of the head (transverse, \( H \)) plane given in (12).

The net tension capacity of a timber member \( F_{B,nl,i} \) is determined by the load capacity of the net cross sectional area of the timber element, and obtained from the capacity of the tensile plane of the net transverse section of the connection \( F_{T,nl,i} \) defined in (13).

### 4.3. Plane Capacities

#### 4.3.1. Lateral shear plane, \( L \)

The capacity of a single lateral longitudinal plane \( L \) of the timber member \( i \) may be defined as:

\[ F_{L,i} = k_{L} t_{LV} L \] (11)

where the shear factor is \( k_{L} = 0.4 + 1.4 \frac{t_{LV}}{r_{L}} \) and \( L \) is the length of the shear plane defined as \( L_i = a_{i}(n_{i} - 1) + a_{i} \) and \( t_{LV} \) is the effective thickness defined as:

- Thin plates (\( t_{p} \leq 0.5d \)): $\frac{t_{LV}}{r_{L}} = 1 - \frac{h_{w}}{d}$

### Table 2

Summary of the tests used for the validation of brittle failure.

<table>
<thead>
<tr>
<th>Author</th>
<th>Number of</th>
<th>Joint scheme</th>
<th>Fastener Type</th>
<th>Timber product</th>
<th>Fastener pattern</th>
<th>Failure mode</th>
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<tr>
<td>Massé et al. [19]</td>
<td>5</td>
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<td>3</td>
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<td>Mischler [24]</td>
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<td>37</td>
<td>0</td>
<td>73</td>
<td>70</td>
<td>17</td>
</tr>
<tr>
<td>Quenneville and Mohammad [14]</td>
<td>37</td>
<td>37</td>
<td>37</td>
<td>0</td>
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<td>14</td>
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<td>40</td>
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<td>Sjödin and Johansson [17]</td>
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<td>Leivo et al. [29]</td>
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<td>SP [30]</td>
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<td>43</td>
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<td>Kevarinmäki [31]</td>
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<tr>
<td>Hanhijärvi and Kevarinmäki [33]</td>
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<td>118</td>
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<td>Legras [22]</td>
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<td>Hübner [26]</td>
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<td>15</td>
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<tr>
<td>Misconel et al. [27]</td>
<td>2</td>
<td>10</td>
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<tr>
<td>Ottenhaus et al. [23]</td>
<td>6</td>
<td>30</td>
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<tr>
<td>Yurrita et al. [12]</td>
<td>39</td>
<td>118</td>
<td>16</td>
<td>13</td>
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<td>39</td>
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<tr>
<td>Total brittle tests</td>
<td>236</td>
<td>1193</td>
<td>72</td>
<td>2</td>
<td>246</td>
<td>0</td>
</tr>
<tr>
<td>%</td>
<td>-</td>
<td>72.9%</td>
<td>30%</td>
<td>0.8%</td>
<td>19.5%</td>
<td>0%</td>
</tr>
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</table>

#### 4.2.2. Brittle load tests

<table>
<thead>
<tr>
<th>Author</th>
<th>Number of</th>
<th>Joint scheme</th>
<th>Fastener Type</th>
<th>Timber product</th>
<th>Fastener pattern</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ehlbeck and Werner [34]</td>
<td>45</td>
<td>146</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Ehlbeck and Werner [35]</td>
<td>47</td>
<td>141</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Jorissen [13]</td>
<td>8</td>
<td>121</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Ballerin [36]</td>
<td>8</td>
<td>24</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Total brittle + ductile tests</td>
<td>344</td>
<td>1625</td>
<td>72</td>
<td>2</td>
<td>10</td>
<td>46</td>
</tr>
<tr>
<td>%</td>
<td>-</td>
<td>20.9%</td>
<td>6.0%</td>
<td>2.9%</td>
<td>13.4%</td>
<td>29.1%</td>
</tr>
</tbody>
</table>

* As there is only two configurations of \( w w \) and \( w s \), they have been gathered with the \( w w \) configurations for the validation in Section 5.2.2
* All the multiple shear connections have 4 shear planes, except three configurations with 6 and one with 8 shear planes.
* Only the test with two or more rows of fastener have been considered.
* Part of the results gathered by Kevarinmäki [28].
* One configuration has been discarded as the properties of the timber were not provided.
* Only the tests assembled and tested at normal moisture conditions are included.
Outer members: 
\[ t_\theta = \begin{cases} 
0.66t & \text{if } \frac{t}{d} \leq 0.76 \\
\max\left(0.76 - \frac{t}{md}, 0.2\right)t & \text{if } \frac{t}{d} > 3 \quad (3 \text{ revisited})
\end{cases} \]

Inner members: 
\[ t_\theta = \begin{cases} 
0.76t & \text{if } \frac{t}{d} \leq 7 \\
\max\left(1.7 - \frac{t}{md}, 0.5\right)t & \text{if } \frac{t}{d} > 7 \quad (4 \text{ revisited})
\end{cases} \]

- Thick plates \( (t_p \geq 1.0d) \):

Outer members: 
\[ t_\theta = \begin{cases} 
t & \text{if } \frac{t}{d} \leq 3 \\
\max\left(1.17 - \frac{t}{md}, 0.35\right)t & \text{if } \frac{t}{d} > 3 \quad (5 \text{ revisited})
\end{cases} \]

Inner members: 
\[ t_\theta = \begin{cases} 
t & \text{if } \frac{t}{d} \leq 11.5 \\
\max\left(1.95 - \frac{t}{md}, 0.65\right)t & \text{if } \frac{t}{d} > 11.5 \quad (6 \text{ revised})
\end{cases} \]

In the case of multiple shear plane connections, same formulae may be used. However, as the influence of the adjacent parts is not considered in the original, a reduction factor of 0.85 of the obtained effective thickness is recommended [16].

4.3.2. Head tensile plane, \( H \)

The capacity of the head tensile plane \( F_{T,i} \) of the timber member \( i \) may be obtained as:
\[ F_{T,i} = k_i A_{T,0} f_0 \]
where the tensile factor is \( k_i = 0.9 + 1.4 \sqrt{\frac{t}{d}} \) and \( A_{T,0} \) is the net area of the head tensile plane defined as \( A_{T,0} = (a_2 - d_b)(n_l - 1)\).

4.3.3. Tensile plane for net tension

The capacity of the tensile plane for net tension \( F_{T,n,i} \) is defined as:
\[ F_{T,n,i} = A_{T,n,0} f_0 \]
where \( A_{T,n} \) is the net tensile plane area defined as \( A_{T,n} = (b - d_b)n_l\).

5. Validation of the proposed model

5.1. Tests database

A total of 22 sets of tests (18 dealing with brittle failure and 4 focused in ductile behaviour) have been gathered for the validation of the model: seven of them were performed in North America (Massé et al. [19], Quenneville and Mohammad [14], Mohammad and Quenneville [15], Dodson [20], Reid [21] and Legras [22]), two in New Zealand (Ottenhaus et al. [23] and Yurrita et al. [12]), and the rest in Europe (Mischler [24], Anderson [25], Sjödin and Johansson [17], Hübner

Fig. 6. Comparison between the load capacity values obtained from the tests \( F_T \) and the corresponding theoretical values \( F_p \) predicted by the models from Eurocode 5 [7], Quenneville and Zarnani [9], Hanhijärvi and Kevarinnmäki [10,11], and the proposal.
A brief summary of the compiled tests is given in Table 2. Only those connections with two or more rows of fasteners have been included, as row shear and block shear are not expected to happen in connections with a single row of fasteners. In this latter case, splitting is the expected failure mode.

The validation process considers two different stages: first, an analysis of the accuracy of the brittle failure models to predict the brittle capacity (where only those research campaigns dealing with brittle failure are considered); second, assessment of the discrimination ability of each of the four models to determine whether a connection fails in a ductile or brittle manner (here also the tests failing in a ductile manner are considered).

At the first stage, after discarding some particular cases indicated in Table 2, a total of 1193 single tests distributed in 236 different configurations have been used for the validation. Three main joint schemes are the most represented: 30.5% of the configurations are steel-wood-steel (sws) connections, whereas 19.5% are wood-steel-wood (sww) connections and the rest, 48.3%, are connections with multiple shear planes (msp), most of them with 4 shear planes (some with 6 and 8 shear planes). Two are the most represented types of fasteners, dowels (72.9%) and bolts (27.1%). Regarding the timber product, the most representative is glulam (80.5%), followed by LVL (15.3%) and with a symbolic presence of solid wood and CLT. Considering the fastener pattern, 75.8% were designed following a grid pattern, while the rest are staggered.

The former percentages vary considerably when considering the second stage, as most of the ductile tests were solid timber wood-wood-wood (www) connections. In this case, 344 configurations with a total of 1625 single tests were used. Brittle failure was reported in 60.2% of the tests, while the rest failed in a ductile way.

5.2. Comparison of the brittle failure accuracy of the models

The first part of the validation compares the test results of each configuration of the database that failed in a brittle way with the load capacity predicted by each of the three existing models (Eurocode 5 [7], Hanhijärvi and Kevarinmäki [10,11] and Quenneville and Zarnani [9]) and the proposal.

The small number of replicates for each configuration in the existing campaigns does not allow to obtain an accurate characteristic load and, hence, the validation is performed at the mean level. The required mean properties are obtained from the characteristic values following the procedure explained by Jockwer et al. [37] and Cabrero et al. [38] which is based on the probabilistic model for timber proposed by the Joint Committee on Structural Safety [39], and previously used in other works [5,18,12].

5.2.1. General analysis of the prediction accuracy

Fig. 6 provides an overview of the relation between the predicted theoretical values (ordinate axis) and the experimental ones (abscissa axis). The ideal correlation, i.e. 1, is plotted as a dashed reference line. The figure includes a linear fitting passing through the origin of coordinates, and its corresponding slope $m$ and correlation coefficient $R^2$.

A statistical analysis considering different metrics is additionally performed, and its results shown in Table 3. The different considered metrics are the coefficient of determination $Q^2$ (best values are the closest to 1), the mean relative error $MRE$, and its corresponding standard deviation $SD$ (lower values are the best ones), the slope $m$ of the linear fitting (already given in Fig. 6), the correlation coefficient $R^2$ (values closer to 1 are the best) and the Concordance Correlation Coefficient $CCC$ (another time values closer to 1 are the best ones, with a recommended threshold value of 0.85). For a more detailed description, the reader is referred to Cabrero and Yurrita [5].

The Eurocode 5 [7] (Fig. 6a) is clearly the least accurate and most conservative model, with a slope $m = 0.767$. Moreover, it reaches consistently the worst results in all the metrics. On the opposite side, Hanhijärvi and Kevarinmäki [10,11] (Fig. 6c) tend to slightly overestimate the capacity of the connections ($m = 1.062$). The model from Quenneville and Zarnani [9] (Fig. 6b) obtains an almost ideal slope of $m = 1.011$. However, this model obtains a high dispersion of results, as demonstrated by parameters such as the correlation coefficient $R^2$ or the standard deviation $SD$. Both models can be considered to obtain quite similar results, as proved by the similar obtained $CCC$, which can be considered as a summary of the rest of the studied metrics.

Finally, the proposal (Fig. 6d) gets the best performance, since it commands all the metrics gathered in Table 3. It slightly improves the slope of Quenneville and Zarnani [9] in 1% and the $R^2$ from Hanhijärvi and Kevarinmäki [10,11] in 5%.

The boxplot from Fig. 7, which evaluates the ratio between the predicted $F_{pred}$ and the tested capacity $F_{exp}$, summarizes the previous discussion. All the box (25th to 75th percentile of the results) from Eurocode 5 [7] falls below the ideal ratio $F_{pred}/F_{exp} = 1$. This additionally proves its conservative trend. The observed overestimation of results from Hanhijärvi and Kevarinmäki [10,11] can be clearly noticed; it additionally features almost no outliers, and all of them close the whiskers. Quenneville and Zarnani [9] has a more reduced box, which is located closer to 1 than the previous model but, in this case, several outliers are far from the whiskers (some of them even with a ratio close to $F_{pred}/F_{exp}$ = 3). The proposed model shows the most reduced box and whiskers (least scatter), and both its median (depicted by the central line within the box) and average (cross inside the box) values are closer to the ideal ratio $F_{pred}/F_{exp} = 1$.

5.2.2. Detailed analysis of the involved parameters

An additional analysis is conducted by classifying the tests into three groups, according to the three most representative joint configurations in the database: steel-wood-steel (sws), wood-steel-wood (sww), and multiple shear planes (msp). Fig. 8 plots this analysis in a similar way as previously done in Fig. 6. An ideal model should show the same slope $m = 1$ for the three joint configurations.

The proposal is the closest model to that ideal situation, with the three slopes ranging from 0.98 to 1.00, and a $R^2$ always higher than 0.89. The second best performance is reached by Quenneville and Zarnani [9] with slopes close to 1.00. However, this model obtains a low $R^2 = 0.65$
for the wsw connections, which is related to the determination of the effective thickness of the lateral plane, as will be seen later. In the case of Hanhijärvi and Kevarinmäki \cite{10, 11}, it shows how it mainly overestimates those cases of msp joints ($m = 1.108$). Finally, the Eurocode 5 \cite{7} gets different slopes for each joint configuration, and it is less conservative in the case of wsw connections.

**Analysis of the influence of the fastener slenderness.**

As the effective thickness of the timber member is one of the main

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**Fig. 7.** Boxplot considering the accuracy of the predicted ratio between the predicted failure load $F_{pred}$ and the tested failure load $F_{exp}$.

**Fig. 8.** Comparison between the values obtained from the tests and the corresponding theoretical values predicted by the models from Eurocode 5 \cite{7}, Quenneville and Zarnani \cite{9}, Hanhijärvi and Kevarinmäki \cite{10, 11}, and the proposal. The results are divided by the joint configuration (wsw, wsw or msp).
differences introduced in the new model, an analysis of its influence has been conducted, and its results are given in Fig. 9. The ratio $F_{\text{pred}}/F_{\text{exp}}$ is given in the ordinate axis and the slenderness in the abscissa axis. The perfect correlation 1:1 is plotted by a horizontal dashed line. The more horizontal and closer to 1 a fitting line is, the more accurate the predicted values are. In the case of the $\text{msp}$ connections, the shown slenderness $t/d$ for these cases is the average of those calculated for each of the timber elements.

The model from Quenneville and Zarnani [9] presents the most scattered graphic (Fig. 9b). Its results worsen as the slenderness increases, specially for the cases of $\text{wsw}$. The shown outliers in Fig. 7 correspond to cases with high slenderness, as already demonstrated by Yurrita and Cabrero [16]. The Eurocode 5 [7] also presents big differences among the different joint configurations, and it does not reach a horizontal slope for any case (Fig. 9a). The behaviour by Hanhijärvi and Kevarinmäki [10,11] (Fig. 9c) provides better general results, but again the best trend is achieved by the new model.

Analysis of the influence of the ratio $E_0/G$.

The new model proposes to define the associated coefficients to the tensile, $k_t$, and lateral planes, $k_v$, by means of the ratio between the modulus of elasticity parallel-to-grain $E_0$ and the shear modulus $G$ of the timber product (see Table 1). The herein described analysis shows its suitability, according to the shown results in Fig. 10.

The two models which provide fixed values (Eurocode 5 [7] –Fig. 10a– and Quenneville and Zarnani [9] –Fig. 10b–) obtain the least consistent behaviour. In the case of Hanhijärvi and Kevarinmäki [10,11], who proposed different values depending on the timber product (Fig. 10c), there is a similar tendency for the three joint configurations: the average ratio $F_{\text{pred}}/F_{\text{exp}}$ slightly decreases as the ratio $E_0/G$ increases. The proposal (Fig. 10d) shows an improved accuracy. Therefore, establishing the coefficients $k_v$ and $k_t$ for the shear and tensile planes as a function of the ratio $E_0/G$ seems to be the most adequate strategy.

5.3. Discrimination ability between brittle and ductile failure modes

The last stage of the validation process, as previously explained, evaluates the discrimination ability of each model to determine if a connection will fail in a brittle or ductile way. Such discrimination ability is plotted in Fig. 11, where the dark colours are related to the percentages of correct predictions (true ductile and true brittle) while the light ones depict those cases where the predictions were wrong (false ductile or false brittle).

The ductile capacity for both the model of Quenneville and Zarnani [9] and the new model corresponds to the EYM, but considering the actual number of fasteners, instead of the effective number of fasteners $n_{ef}$. In the case of the Eurocode 5 [7], two options are shown, as it does not explicitly provide the information of the failure mode when the $n_{ef}$
Therefore, the option named "Eurocode 5 Brittle", considers all the cases where \( n_f \) is different from the actual \( n_c \) as brittle failure, whereas the second option ("Eurocode 5 Ductile") only considers brittle failure when the minimum capacity is that obtained by applying the Annex A model (therefore, all the cases where the minimum capacity is different from the actual capacity are considered as ductile failure).

Fig. 10. Comparison of the prediction accuracy of the studied models, considering the stiffness ratio between the Modulus of elasticity parallel-to-grain \( E_0 \) and the shear modulus \( G \).

Fig. 11. Discrimination ability. Comparison between Eurocode 5 [7], Hanhijärvi and Kevärinnäki [10, 11], Quenneville and Zarnani [9] and the proposal.
The best discrimination ability is achieved by the new model (85% of right predictions). The “brittle” option from Eurocode 5 [7] appears as the worst one, with only around 50% of correct predictions. The “ductile” case, with almost 70% of correct failure modes, slightly improves the discrimination ability from Hanhijärvi and Kevarinmäki [10,11] (65%). The second position is for Quenneville and Zarnani [9] (around 75%).

6. Conclusions

A new model dealing with all the brittle failure modes for timber connections with large diameter fasteners is proposed. The new model tries to obtain a better prediction ability and, at same time, provides a calculation process with a limited complexity.

The new proposal takes the simplicity of the model from Quenneville and Zarnani [9] as a basis, and it additionally considers different assumptions. Some of the modifications include the effective thickness of the timber member, the length of the lateral shear planes, a different way to account for the interaction between shear and tensile stresses, or the special considerations for connections with multiple shear plane connections.

An extensive database of experimental results is used for the validation of the proposal, which is compared with the existing models. The model from the Eurocode 5 [7] can be considered as the worst one, as it tends to a clear underestimation of the load capacity of timber connections. Even if the model from Hanhijärvi and Kevarinmäki [10,11] contains a better discrimination ability between brittle and ductile failure modes.

The new proposal improves both the accuracy and discrimination ability of all the previous models, while being still quite a simple model to use and understand for practitioners. Therefore, it can be concluded that the new model improves all the existing models, reaching the best balance between accuracy and simplicity.

As a future work, a new model dealing with timber connections with small diameter fasteners (the ones which do not fully penetrate the whole timber thickness such as nails, screws or rivets) and plug-shear failure, similar to the one proposed herein, is required.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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