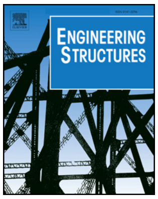
ARTICLE IN PRESS

Engineering Structures xxx (xxxx) xxx–xxx

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct



Performance assessment of existing models to predict brittle failure modes of steel-to-timber connections loaded parallel-to-grain with dowel-type fasteners

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ARTICLE INFO

Keywords:
Brittle failure parallel-to-grain
Splitting
Row-shear failure
Block-shear failure
Plug-shear failure
Timber connections

ABSTRACT

For safety reasons, ductile failure in timber connections with dowel-type fasteners is always recommended. It has usually been assumed that it can be achieved by fulfilling minimum spacing requirements between fasteners. However, recent works address the need to account for brittle failure modes (namely splitting, row-shear, and block and plug-shear) in connections loaded parallel-to-the-grain in an explicit manner, in order to evaluate them and achieve the desired ductility. This article describes the brittle failure modes and reviews the existing calculation models proposed by several authors – some of them included in standards. Finally, the performance of these models is assessed against an extensive database of tests gathered from the literature following a comprehensive methodology.

1. Introduction

It is well known that connections are of crucial importance in the behaviour of a structure, not only in terms of cost or influence on the global structural behaviour, but also in terms of safety. They have been reported to be involved in almost one quarter of recent collapses of timber structures, where more than half of the involved connections were with dowel-type fasteners [1,2].

The European Yield Model, included in the Eurocode 5 [3] dates back to early works by Johansen [4] and only provides the capacity for the ductile failure mode of joints, which is governed by the embedment of the timber or the bending of the dowel-type fasteners. It is assumed that no brittle failure occurs if the given minimum spacing requirements are met.

However, connections in construction practice include a number of fasteners larger than those currently investigated in the laboratories. As a consequence, the joint capacity could be governed by a brittle failure mode [5]. Nevertheless, designers are not aware of this fact, as shown by a survey conducted in the European area by the Working Group 3 of the COST Action FP1402 [6,7]: more than 30% of the participants (designers, engineers, constructors…) did not know about their existence (even up to 24% among those with more than 10 years of experience in the field of timber structures).

Some well-known building collapses were originated by a brittle failure of the connections, as the Siemens Arena and the Jyväskilä Fair roof [1,8]. In the case of the Utopia pavilion [5], a previous experimental campaign pointed out the resulting brittle failure, and collapse was prevented at the cost of reinforcing the connections on-site with glued-in-rods.

The prenormative version of the Eurocode [9] had been used in both the Jyväskilä Fair roof [8] and the Utopia pavilion [5]. It was demonstrated that it did not cover brittle failure in an adequate way [10,5]. Those experiences gave rise to a brief description in Racher [11], and a proposal from Ranta-Maunus and Kevarinmäki [10] of a supplement to the Eurocode 5 concerning the calculation of block shear failure. Both stand as the origin of the current Annex A of the Eurocode 5 [3].

Brittle failure modes had until then been grouped under the so-called group effect concept [12], which assumed that an interaction effect among the fasteners exists, and as a result the total capacity of the connection is reduced [13]. Nozynski [14], in 1980, was one of the first authors to notice fracture of wood along the row of nails, and proposed the introduction of an effective number of fasteners. Several similar design equations were suggested during the development of the Eurocode 5 [15–17], and were soon adopted by different countries in their design standards [18].

However, Smith and Steck [19] noticed already in 1985 the need for new theories to obtain the *“ultimate capacities of joints with brittle failures”*. Since then, several references introduced the concept of brittle failure. Among them, the STEP books, where Racher [11] provides a brief explanation of this concept for dowelled connections, and

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https://doi.org/10.1016/j.engstruct.2018.03.037
Received 28 November 2017; Received in revised form 21 February 2018; Accepted 14 March 2018
0141-0296/

Please cite this article as: Cabrero, J.M., Engineering Structures (2018), https://doi.org/10.1016/j.engstruct.2018.03.037

Nomenclature

Greek symbols

α	friction angle between the fastener and the timber in the hole
α_t	tensile stress coefficient [27]
β_t,β_s	stress coefficients (tensile and shear) based on nail spacing [27]
β_p	ratio of the perpendicular-to-grain wedging force to the parallel-to-grain fastener load
γ_h	stress coefficient depending on nail penetration [27]
Γ_i	additional expressions related to the relative stiffness of each failure plane [43,29–31,46]
Φ	factor function of fracture energy, location and geometry [35]

Lower cases

a_1	spacing between columns of fasteners
a_2	spacing between rows of fasteners
a_3	distance to the parallel-to-grain edge
a_4	distance to the perpendicular-to-grain edge
$a_{L,min}$	minimum of a_1 and a_3
b	width of the wood member
b_c	width of the connection
b_{net}	net width of the connection
c	rank correlation coefficient [68]
d	fastener diameter
d_r	rivet short diameter
$\overline{f}$	average predicted values
f_i	predicted values
$f_{h,0}$	embedment strength in the parallel-to-grain direction
$f_{r,h,0}$	embedment strength for rivets in the parallel-to-grain direction
$f_{t,90}$	tensile strength parallel-to-grain
$f_{t,90}$	tensile strength perpendicular-to-grain
f_v	shear strength
f_y	yield strength of the fastener
k_{con}	factor of stress concentration [22]
k_{ef}	geometric coefficient for determining the n_{ef} of nails in Eurocode 5 [3]
$k_{t,cnctr}$,$k_{v,cnctr}$	stress concentration factors depending on the timber product [22]
k_v	factor depending on the load distribution [22]
k_{int}	interaction factor in Hanhijärvi and Kevarinmäki [22]
ℓ	penetration length of a small fastener in the wood
m	slope of a linear fit passing through the origin
n	number of tests
n_c	number of fastener columns of the connection
n_{ef}	number of effective fastener columns of the connection
n_r	number of fastener rows of the connection
n_s	number of shear planes of the connection
n_w	number of wood members of the connection
r_m^2	coefficient correlation based on the slope of different fitting procedures [75–77]
$s_{t,90,i}$	geometric parameters for splitting [22]
t	thickness of the wood member
t_{ef}	effective thickness of the connection
t_p	steel plate thickness
$\overline{y}$	average of experimental values
y_i	experimental values

Upper cases

CCC	concordance correlation coefficient, defined in (3) [71,73,72]
E_0	modulus of elasticity in the parallel-to-grain direction
G	modulus of rigidity
G_f	fracture energy value
J_r	factor depending on the number of rows[23,40]
K_H,K_B,K_L	stiffness of head, bottom, and lateral planes [43,29–31,46]
K_t,K_s	coefficients (tensile and shear) depending on the n_c and n_r [27]
k_{LS}	factor depending on the load distribution along the fastener[43]
L_c	length of the connection
L_{net}	net length of the connection
$M_{r,y}$	rivet yield moment.
M_y	fastener yield moment.
MRE	mean relative error, defined in (4)
Q^2	coefficient of correlation defined in (2) [66,73]
R_5	over-prediction coefficient when characteristic properties values are applied
SD	standard deviation of the mean relative error
X_s,X_t	parameters function of the timber product [43]

Kevarinmäki [20] describes it for nailed connections in trusses.

Several model proposals for the different types of brittle failure have been made: for splitting [3,21,22], row-shear [23,22] block-shear models for dowelled [23,24], nailed [25,26] and riveted connections [27–33]; some of them are fracture-mechanics based models, mainly for splitting and row-shear [34,16,35–37]. Most of them will be reviewed in this paper.

Brittle failures, such as block and row-shear models were introduced in the early 2000s in the Canadian Code O86 [38,24,39–42]. In the case of the Eurocode [3], splitting and row-shear failures are implicitly taken into account by means of the effective number of fasteners based on the work by Jorissen [16]. A model for block and plug-shear is included as Annex A [3], dating back to the previously referred proposals [11,10]. Currently, the subject is under consideration in the New Zealand Standard draft [43] and in the future Eurocode 5. Within the COST Action FP 1402 [7], which aims to prepare background documents for the future Eurocode 5, Working Group 3 has been in charge of the review of the different proposals for this type of failure, which this article summarizes.

This work provides insight into the different brittle failure modes of steel-to-timber connections with dowel-type fasteners loaded parallel-to-grain. It compiles the different available models in an ordered and coherent way, and benchmarks them against experimental tests compiled from the literature.

Special attention is given to those models which aim at providing a complete and consistent set of equations to discriminate among ductile and brittle failures. Such a complete method is nowadays provided in the New Zealand Standard draft [43], and the method for dowelled connections by Hanhijärvi and Kevarinmäki [44,22]. It may be argued that also a complete model is given in the Eurocode 5 [3], although some failure modes are implicitly taken into account.

The paper is organised as follows: first, the different failure modes and parameters of connections loaded parallel-to-grain are described in Section 2. Section 3 reviews the different existing models for each failure mode. Section 4 provides information about the experimental data set, and the methodology used to compare and benchmark the different models. Special attention is given to the different possible metrics to assess the performance of the models. The results concerning