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A State-of-art of Experimental Research and Calculated Models of Dowel-type Timber Connections in Fire

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Abstract

Fire safety is one of the most significant issues for the design of mid-rise and high-rise timber structures. A large number of experimental tests were conducted during the last three decades to investigate the fire performance of the dowel-type timber connections. Many influenced parameters (e.g. the thickness of the side timber, the load ratio, the fasteners type etc.) were considered in those experiments. Relevant calculated models were proposed by previous researchers to estimate the fire resistance of the connections. In this study, a series of experimental programs of dowel-type connections in fire are collected. Then, empirical formulas proposed by EN 1995-1-2, Fire safety in timber buildings, and previous researchers are presented and analyzed. The accuracy of those formulas is checked by comparisons between the experimental data and estimated results. The collected experimental research and empirical formulas can be used as the reference for the fire design of dowel-type timber connections in the future.

Keywords: Dowel-type timber connections, fire design, timber structures.

1. Introduction

More and more engineers and architects around the world are trying to recast timber as the main construction material for mid-rise or even high-rise buildings. The fire performance of the connections has crucial effect on the design of the timber structures. Therefore, a large number of experimental tests of dowel-type connections in fire were conducted with varied connection configuration, load ratio and fasteners type etc. These research programs were aimed to investigate the mechanical behavior of connections in fire and the main influenced factors. Results shows that connections configurations, the thickness of timber members, load ratio, type and diameter of fasteners, and load direction have an impact on the fire resistance of dowel-type connections and need to be considered in the fire design.

The analytical approaches for timber connections in fire are still very complex due to the high anisotropic characteristics and brittle failure behaviours of timber materials. Two common methods were used by researchers to investigate the fire performance of timber connections. One is the numerical methods and another is the empirical formulas. Numerical methods that combine the heat transfer FE models and load-carrying models show good accuracy as compared with data from tests (Audebert et al. 2011; Moss et al. 2010). These methods require thermal properties of timber in heat transfer model to obtain the temperature field and some numerical calculations in load-carrying models to estimate the capacity of timber connections after a certain time of fire exposure. Besides, heat transfer model can also be combined with Johansen's yield model (Erchinger et al. 2010; Palma et al. 2017) and component model (Cachim et al. 2009). The numerical methods are quite time-consuming, while empirical formulas provide an easier way for engineers in fire design of timber connections. A common empirical formula known as "reduced load method" is proposed by EN1995-1-2 (ECS. 2004). However, there are limitations in the fire design rules of connections in EN1995-1-2 such as unsatisfactory accuracy of the results for connections with thicker side timber members (Palma. 2016) or loaded with an angle (Audebert et al. 2019). The fire design method in *Fire safety in timber buildings* (Östman B et al. 2010) can be considered as a complement to EC5. These studies are still at the research stage. In addition, new formula was proposed by Audebert et al. (Audebert et al. 2020) to predict fire resistance of dowel-type timber connections considering four main parameters including the thickness of the timber members, type and diameter of the fasteners and load ratio. Improvements of the EN1995-1-2 were also proposed by Palma et al. (Palma. 2016). In this improvement, parameter k is related not only with connection configurations but also with the timber side members.

In this study, a series of experimental programs of dowel-type connections in fire are collected. Then, empirical formulas proposed by EN 1995-1-2, *Fire safety in timber*

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buildings, and previous researchers are presented and analyzed. The accuracy of those formulas is checked by comparisons between the experimental data and estimated results.

2. Collected experimental tests

This section presents the collected experimental tests of timber dowel-type connections in fire during the last three decades.

2.1. Connection with nails and screws

Two experimental tests by Norén (Norén. 1996), Lau (Lau. 2006) and Chuo (Chuo. 2007) are presented in this section.

The experimental tests were performed by Norén (Norén. 1996) on nail connections loaded in tension parallel to grain in 1996. The wood member was spruce with density from 381 to 422 kg/m³. The nails were longitudinally grooved wire nails with thickness of 2.8 mm and 4.0 mm. Four series of tests were conducted on wood-to-wood nail connections. The configurations of tested specimens were shown in Fig. 1. The difference between series 1 and 4 was that tested specimens in series 4 were equipped with thermocouples. Tests in normal temperature were conducted for specimens in series 1 and 2. For series 1 and 2, different load ratios were selected from 10% to 60% in the subsequent fire tests. For series 3, the load ratio is determined by the applied load in fire divided by the ultimate capacity of series 1 in normal temperature.

The results of fire resistance for connections in series 1 and 2 are shown in Fig. 2. Fig. 2 also shows comparison between series 1 and 3. The thicker side member increases the fire resistance of connections under the same load ratio. Fig. 3 shows typical failure modes of nails connection in single shear. In normal temperature tests. the failure mode was No. 6 for specimens in series 1 and 2. Nails yielded in both central and side members. However, in fire tests, failure modes changed to No. 4 due to the charring of the side member at a load ratio larger than 0.35. When the load ratio was ranged from 0.35 to 0.2, No 4 and No. 1 failure modes were observed in tests. When the load ratio was less than 0.2, the failure mode for all tested specimens was No.1.

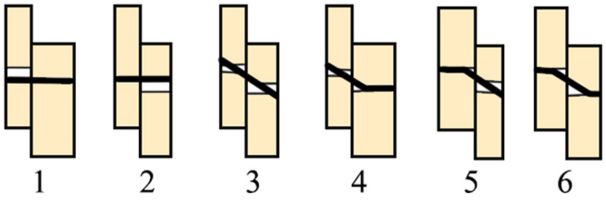


Figure 3. Typical failure modes of nails connection in single shear.

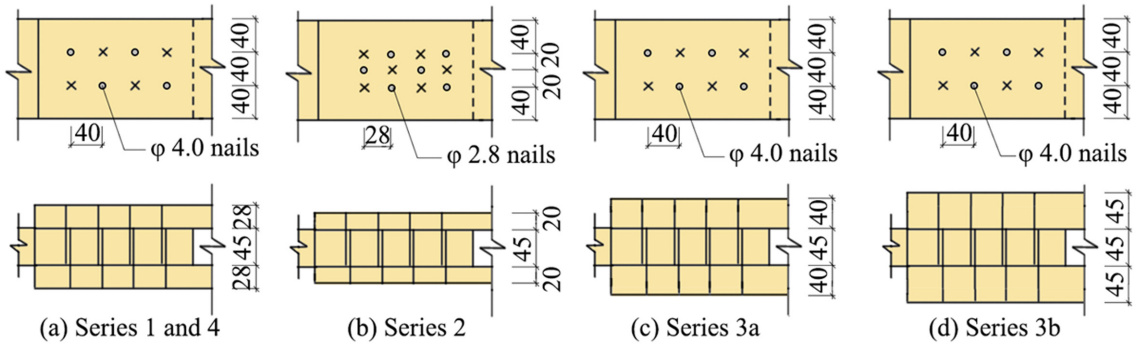


Figure 1. Configurations of the tested connections (Norén (1996)).

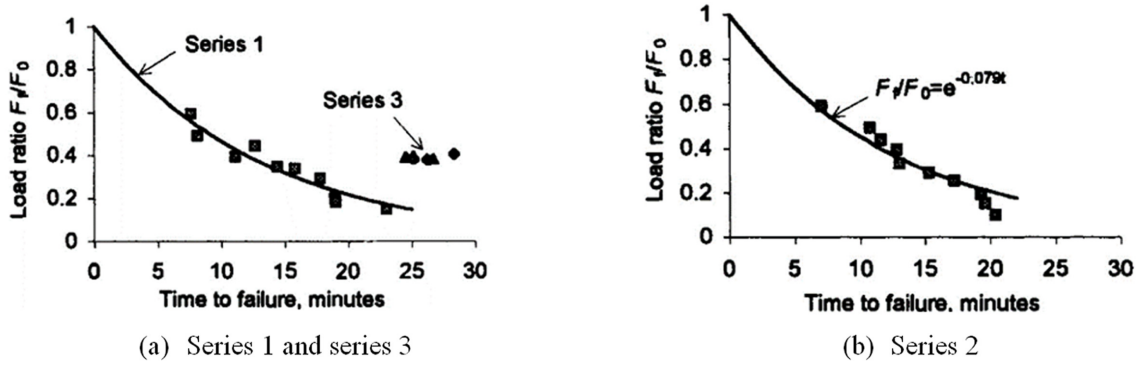


Figure 2. Results of fire resistance for connections in series 1, 2 and 3 (Norén (1996)).

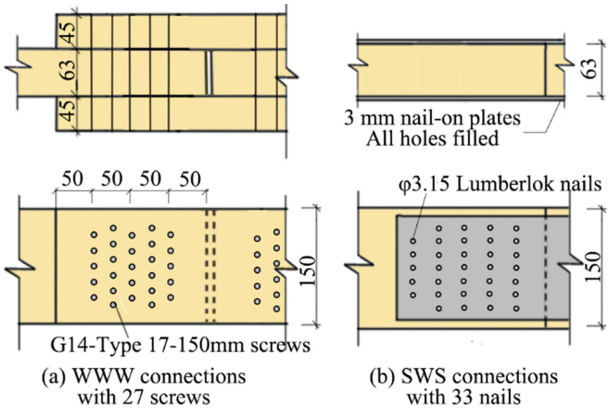


Figure 4. Arrangement of screws and nails connections (Lau and Chuo (2006&2007)).

Fire tests loaded in tension parallel to grain for connections with nails and screws were conducted by Lau (Lau. 2006) and Chuo (Chuo. 2007). Wood-wood-wood connection (WWW connections) and steel-wood-steel connections (SWS connections) were designed as shown in Fig. 4. LVL (Laminated Veneer Lumber) was selected as the wood member. The failure time in fire was determined when the applied load was no longer sustained. The applied load F in fire condition over the ultimate capacity of the connections in normal temperature N_u is defined as the load ratio η. Ultimate capacity in cold tests and fire resistance of the connections with nails and screws in this study are shown in Table 1.

2.2. Connection with bolts

A series of fire tests of bolted connections loaded in tension parallel to grain was conducted by Dhima (Dhima. 1999), Lau (Lau. 2006) and Chuo (Chuo. 2007), Peng (Peng. 2010) and Zhang et al. (Zhang et al. 2015). Table 2 summarizes fire-resistance results of connections with bolts. Typical tested connections with bolts are shown in Fig. 5.

For tests conducted by Dhima (Dhima. 1999), The ultimate capacity of the connections in normal temperature was determined according to the EN 1995-1-1. Thus, η_{EC5} was used instead. For tests conducted by Lau (Lau. 2006) and Chuo (Chuo. 2007), it was noted that the temperature during the fire testing did not follow the ISO 834 curve. Some conversions were conducted to predict the fire resistance of the connections. A formula was developed by Harris (Harris, 2004) to convert the failure time in a nor-standard furnace to failure time in a standard furnace:

$$t_{ISO}=\frac{c_{cust}}{c_{ISO}}\times t_{cust} \quad (1)$$

where t_{ISO} is the fire resistance time in ISO furnace (min); $t_{cust}$is the time to fire in custom furnace (min); c_{ISO} is the char rate recorded in the ISO furnace (mm/min); c_{cust} is the char rate recorded in the custom furnace (mm/min).

Another method to determine the fire resistance in nor-standard furnace was developed by Nyman (Nyman. 2001). The fire resistance was determined by comparing the cumulative radiant heat exposure at any time in this fire testing with that in the standard fire. Two method were all used to calculate the fire resistance of the connections in this study (Lau. 2006; Chuo. 2007). The results shows that there was only a slight difference in fire resistance between those two methods.

For tests conducted by Peng (Peng. 2010), some of the specimens were protected by gypsum board or Douglas ply wood. For most of WSW connections, no bending of bolts occurred, but hole elongation and wood member splitting were observed after the fire tests. For SWS connections, bending of bolts, hole elongation and cutting through were observed after the fire tests.

2.3. Connection with dowels

Table 3 summarizes the fire-resistance results of connections with dowels. Typical tested connections with dowels are shown in Fig. 6.

Fire tests of connections loaded in tension parallel to grain was conducted in France by Dhima (Dhima. 1999). The ultimate capacity of the connections in normal temperature was determined according to the EN 1995-1-1, except for 2 groups, of which the ultimate capacity was determined from tests. After the fire tests, brittle cracks were observed along the row of fasteners for WWW connections. There are some factors influencing the fire resistance of the connections such as the fasteners type (dowel or bolt) and load ratio. However, the number of the fasteners have relatively little effect on the fire resistance for steel-to-timber connections.

A program (Racher. 2002; Laplanche. 2003; Ayme. 2003; Racher; 2010) was conducted as a supplemental experimental data of the tests by Dhima (Dhima. 1999). In addition to the minimum requirements of the spacing and size of connections in EC5, the edge and end distances were increased by a_{fi}, considering the charring of the wood. The results shows that the thickness of the timber elements and the load ratio have large influence on the

Table 1. Ultimate capacity in cold tests and fire resistance of the connections

Ref.	Type	Fasteners	N_u(kN)	η	$t_{fi,test}$ (min)
Lau (2006) and huo (2007)	WWW connections	Screws	255.66	0.11	30.3
	SWS connections	Nails	91.5	0.32	9

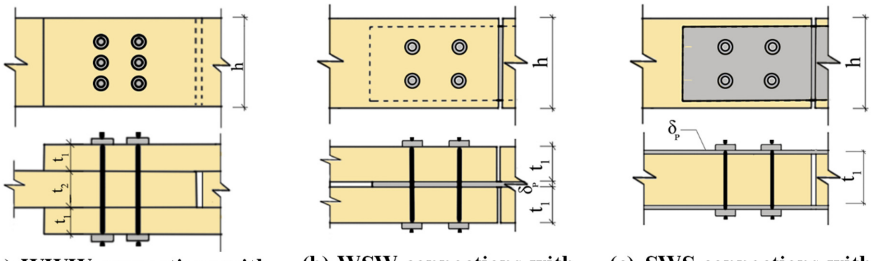
Table 2. Fire-resistance results of bolted connections

Ref.	Type of connections	Wood	t_1 (mm)	t_2 or δ_p (mm)	h (mm)	Fasteners		N_u (kN)	η	$t_{fi,test}$ (min)
						Type	φ (mm)			
Dhima (1999)	WWW connections	GL28h glulam	60	100	240	Bolts	20	235	0.47(η_{EC5})	24
			60	100	240				0.94(η_{EC5})	15
			50	80	170	Bolts	12	110	0.36(η_{EC5})	22
			50	80	170				0.84(η_{EC5})	13
	WSW connections	GL28h glulam	60	6	240	Bolts	20	262	0.37(η_{EC5})	22
									0.75(η_{EC5})	15
			60	6	400	Bolts	20	523	0.37(η_{EC5})	23
									0.73(η_{EC5})	16
			50	6	170	Bolts	12	66	0.46(η_{EC5})	17
									0.91(η_{EC5})	10
			50	6	170	Bolts	12	123	0.41(η_{EC5})	18
									0.83(η_{EC5})	11
			50	6	240	Bolts	12	246	0.40(η_{EC5})	18
									0.80(η_{EC5})	13
Lau(2006) and Chuo (2007)	WWW connections	LVL	45	63	150	Bolts	12	200.22	0.14	20.5
	WSW connections	LVL	45	6	150	Bolts	12	216.07	0.13	16.5
			45	6	150	Protected bolts	12	216.07	0.13	19.7
	SWS connections	LVL	63	6	150	Bolt	12	161.6	0.18	8.8
			63	6	150	Protected bolt	12	161.6	0.18	18.7
Peng (2010)	WSW connections	No.2 SPF Lumber	38	9.5	140	Bolts	12.7	58	0.10	14.5
									0.10	15
									0.29	8
									0.30	8.5
		20f-EX S-P Glulam	60	9.5	190	Bolts	12.7	115	0.11	28
									0.18	22.5
									0.29	17.5
		20f-EX S-P Glulam	60	9.5	190	Bolts	19.1	65	0.11	27
									0.32	15
		20f-EX S-P Glulam	60	9.5	190	Bolts	19.1	227	0.10	26
									0.30	14
									0.30	51*
		20f-EX S-P Glulam	80	9.5	190	Bolts	19.1	243	0.10	36.5
									0.10	43**
									0.29	19
									0.29	34***
	SWS connections	24f-EX D.fir	80	9.5	190	Bolts	12.7	169	0.10	14
									0.30	8.5
									0.30	41.5#
		24f-EX D.fir	130	9.5	190	Bolts	19.1	404	0.10	23.5
									0.19	15.5
									0.19	22.5##

Table 2. Continued.

Ref.	Type of connections	Wood	t_1 (mm)	t_2 or δ_p (mm)	h (mm)	Fasteners		N_u (kN)	η	$t_{fi,test}$ (min)
						Type	φ (mm)			
Zhang et al. (2015)	SWS connections	Douglas fir glulam	100	10	210	Bolts	14	280	0.10	27'12"
									0.20	13'08"
									0.30	11'03"
									0.30	13'56"!
									0.30	18'43"!!
		Douglas fir glulam	100	10	210	Bolts	14	280	0.30	10'13"

*This specimen was protected by a single-layer of 15.9 mm Type X gypsum board;
**This specimen was protected by 19.1 mm plywood on the lower and upper face;
***This specimen was protected by a double-layer of 12.7 mm Douglas fir plywood;
#This specimen was protected by a single-layer of 15.9 mm Type X gypsum board;
##This steel plate and bolts were protected by 2 mm thick intumescent coating;
!Steel plate was protected by fire retardant coating;
!!Protected by fire retardant coating.



(a) WWW connections with 6 bolts (Lau and Chuo (2006&2007))

(b) WSW connections with 4 bolts (Peng (2010))

(c) SWS connections with 4 bolts (Peng (2010))

Figure 5. Typical tested connections with bolts.

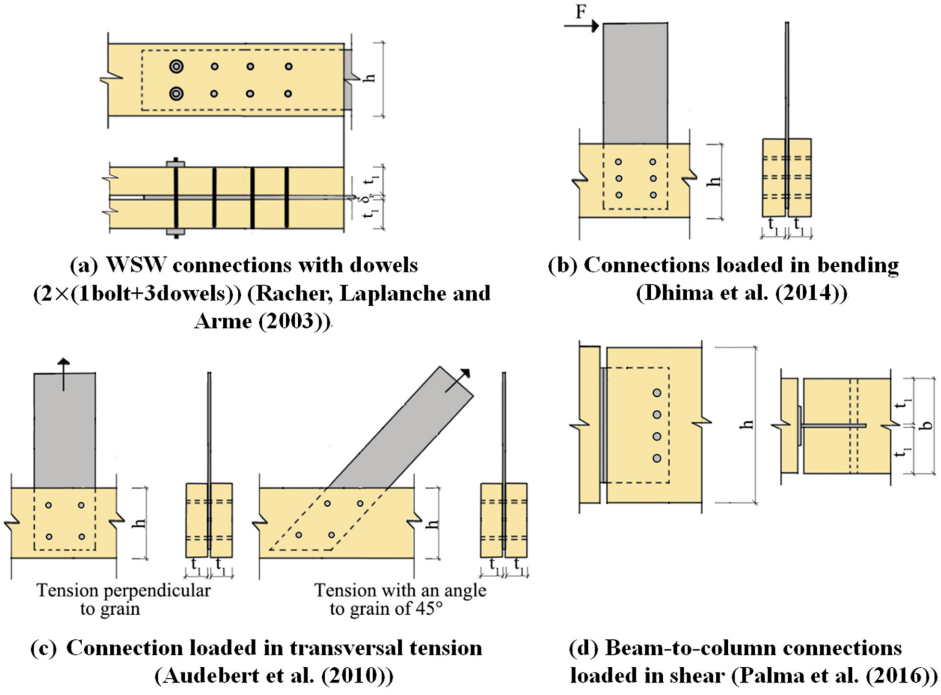
Table 3. Fire-resistance results of doweled connections

Ref.	Type of connections	Wood	t_1 (mm)	t_2 or δ_p (mm)	h (mm)	Fasteners		N_u (kN)	η	$t_{fi,test}$ (min)
						Type	φ (mm)			
Dhima (1999)	WWW connections	GL28h glulam	60	100	240	Dowels*	20	360($N_{u,test}$) 230($N_{u,EC5}$)	0.31(η_{test}) 0.48(η_{EC5})	35
									0.61(η_{test}) 0.96(η_{EC5})	7**
			60	100	240	Protected dowels*	20	211	0.47(η_{EC5})	41
									0.94(η_{EC5})	22
			60	100	400	Dowels*	20	460	0.31(η_{EC5})	38
									0.62(η_{EC5})	23
			50	80	170	Dowels*	12	117	0.39(η_{EC5})	32
									0.77(η_{EC5})	13
			50	80	170	Protected dowels*	12	129($N_{u,test}$) 103($N_{u,EC5}$)	0.30(η_{test}) 0.36(η_{EC5})	27
									0.60(η_{test}) 0.76(η_{EC5})	9
Racher, Laplanche and Arme (2003)	WWW connections	GL24h glulam	64	112	254	Dowels*	16	267.9	0.10	59.5
									0.20	45.5
									0.30	39.5
			84	160	254	Dowels*	16	269.4	0.10	79
									0.30	54

Table 3. Continued.

Ref.	Type of connections	Wood	t_1 (mm)	t_2 or δ_p (mm)	h (mm)	Fasteners		N_u (kN)	η	$t_{fi,test}$ (min)
						Type	φ (mm)			
Racher, Laplanche and Arme (2003)	WSW connections	GGL24h glulamL24h glulam	76	8	254	Dowels*	16	374.0	0.30	36
			75	8	252	Dowels*	16	335.1	0.10	55.5
									0.20	41
									0.30	36
			75	10	292	Dowels*	20	456.3	0.10	52
									0.30	37
			75	6	212	Dowels*	12	198.4	0.10	54
									0.30	39
			95	10	288	Dowels*	20	450.0	0.10	90
									0.30	45
Lau (2006) and Chuo (2007)	WSW connections	LVL	45	6	150	Dowels	5	249.9	0.12	25.8
						Protected dowels	5	249.9	0.12	29
Audebert et al. (2010) (Connection loaded in tension perpendicular to grain and with angle of 45°)	WSW connections	GL24h glulam	77.5	8	267	Dowels	16	82.6	0.10	71
									0.30	48
			105	10	320	Dowels	20	129.1	0.30	69
									0.45	62
			77.5	8	267	Dowels	16	128	0.10	69
									0.30	35
			105	10	320	Dowels	20	180	0.30	62
Dhima et al. (2014) (Connections loaded in bending)	WSW connections	GL24h glulam	77.5	8	445	Dowels	16	33.6	0.12	88
									0.38	61
									0.38	55#
			77.5	8	445	Dowels	16	38.8	0.38	58
									0.38	52#
									0.45	50
Palma et al. (2016) (beam-to-column connections loaded in shear)	WSW connections	GL24h glulam	77	5	260	Dowels	12	51.5	0.30	38.7
									0.30	44.1
			77	5	260	Dowels	12	51.5	0.30	32.2
									0.30	34.0
			77	5	260	Dowels	12	51.5	0.30	48.0
									0.30	46.8
			77	5	260	Dowels	12	39.3	0.39	37.8
			117	5	340	Dowels	12	58.5	0.27	76.6
									0.26	82.8
			77	5	260	Dowels	8	33.9	0.28	49.0
									0.28	43.4
			77	5	260	Dowels	8	46.7	0.31	43.3
									0.30	44.5
			77	5	260	Dowels	8	68.6	0.30	44.4
									0.30	41.8
			117	5	340	Dowels	8	89.3	0.23	77.6
									0.23	72.8

*2 × (1bolt+3dowels)
**Failure due to defects in timber member.
#Cyclic loading was applied before fire tests;



(a) WSW connections with dowels (2×(1bolt+3dowels)) (Racher, Laplanche and Arme (2003))

(b) Connections loaded in bending (Dhima et al. (2014))

(c) Connection loaded in transversal tension (Audebert et al. (2010))

(d) Beam-to-column connections loaded in shear (Palma et al. (2016))

Figure 6. Typical tested connections with dowels.

fire resistance of the connection. However, the thickness of the steel plate, the diameter of the connections and the increase of the spacing value of the fasteners have little influence on the fire resistance of the connection as long as the they have satisfied the minimum requirements of EC5.

For tests conducted by Audebert et al. (Audebert. 2010; Audebert et al. 2014), connections were loaded in tension perpendicular to grain and with angle of 45° under standard fire. According to EN 1995-1-2, additional thickness a_{fi} was added to the end and edge distance of the connections for fire safety. The failure modes in fire situation in these tests were large deformation of the embedment area and splitting along the fastener's rows.

Fire tests in bending was conducted by Dhima et al. (Dhima et al. 2014) on steel-to-timber connections in 2014. For each configuration, two specimens were tested in cold temperature and three specimens were tested in fire. In order to investigate the influence of the clearance between the fasteners and the wood on the fire resistance of the connection, cyclic loading was also applied to some specimens before fire tests. Displacement control were applied to the cyclic loading procedure. The specimens were loaded in cyclic tests until 75% of the value of u_y. u_y is the limit slip of the connections at failure in cold tests. The failure modes of the specimens in fire tests were large embedment deformation of the wood near the fastener area. No brittle failure was observed in fire tests. The post-cyclic loading which resulting in clearance of between the fasteners and wood seems to have little influence on the fire resistance of the connections.

Over 30 full-scale beam-to-column connections were tested by Palma et al. (Palma et al. 2016). It was assumed that the beam-side of the connection were more vulnerable to the shear load. Thus, some of the tests in cold temperature were conducted on full connections, while some of the tests were conducted on beam-side of the connections. It was shown from the tests in cold temperature that the beam-side of the connections were failed first even in the tests of full connections. Thus, the column member was partially protected to avoid the burning from the back and side of the column that might promote a premature failure. Both embedment deformation and splitting cracks were observed after the fire tests. The gap between the column and beam affected the fire resistance of the connections. An increase of the gap from 10 mm to 20 mm led to an average reduction from 44 min to 33-34 min. However, a reduction of the gap from 10 mm to 0 mm only led to a increase of 3-4 min in fire resistance. It was mainly because the larger gap between the column and beam leads to charring of the column side.

3. Approaches to predict fire resistance of timber connections

Simplified analytical approaches were proposed by researchers to predict the fire resistance of the wood connections with dowel-type fasteners. Four methods are presented in this section.

3.1. Calculated models by EN1995-1-2:2004

Fire design rules of timber connections in EN1995-1-2

are based on experimental programs conducted before or around its publication time. Two common empirical approaches that were called the simplified rules and the reduced load method were recommended in EN1995-1-2. These methods were applied to connections with timber side members. For simplified rules, the fire resistance of the connections was attained based on the thickness of the side members or protection measures (e.g. addition of gypsum plasterboard). For the reduced load method. The estimated ultimate capacity of the connections after fire was attained based on load ratio of the connections.

3.1.1. Simplified rules

For unprotected wood-to-wood connections where dimensions of side members, spacings, end and edge distance satisfy the minimum requirements of EN 1995-1-1, the minimum fire resistance are set as 15 min for nails, screws, bolts, and connectors or 20 min for dowels as shown in Table 4.

For connections with dowels, nails or screws with non-projecting heads, fire resistance periods $t_{d,fi}$ greater than the values in Table 4, but not exceeding 30 min, was achieved by increase the dimensions such as the thickness of side members, the width of the side members, the end and edge distance of fasteners. The increased dimensions a_{fi} is determined by:

$$a_{fi} = \beta_n \times k_{flux} \times (t_{req} - t_{d,fi}) \tag{2}$$

where β_n is the charring rate; k_{flux} is the coefficient taking into account heat flux through the fastener, $k_{flux} = 1.5$; t_{req} is the required standard fire resistance period; $t_{d,fi}$ is the fire resistance period of the unprotected connection given in Table 4.

For wood-steel-wood connections where the steel plate does not project beyond the wood surface, the fire resistance is determined based on the width b_{st} and thickness t_s of the steel plate as shown in Fig. 7 and Table 5. No limitation of fastener type (e.g. dowels or bolts) are made in this rule. The prescribed minimum width of the steel plate b_{st} for connections with unprotected edges in general under R60 class is recommended to be 400 mm instead of 280 mm by some researchers (Scheer et al. 2009; Kordina et al. 1995).

Table 4. Fire resistance of unprotected connections with side members of wood

	Time of fire resistance $t_{d,fi}$ (min)	Provisions*
Nails	15	$d \geq 2.8$ mm
Screws	15	$d \geq 3.5$ mm
Bolts	15	$t_1 \geq 45$ mm
Dowels	20	$t_1 \geq 45$ mm
Connectors according to EN 912	15	$t_1 \geq 45$ mm

*d is the diameter of the fastener and t_1 is the thickness of the side member.

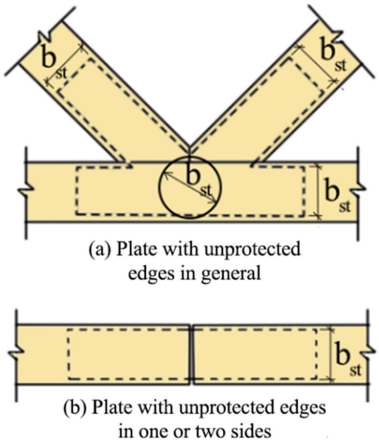


Figure 7. Steel-to-timber connection with an internal steel plate.

Table 5. Fire resistance classes of steel-to-timber connections with an internal steel plate

		b_{st}
Unprotected edges in general	R30	≥ 200 mm
	R60	≥ 280 mm
Unprotected edges on one or two sides	R30	≥ 120 mm
	R60	≥ 280 mm

3.1.2. Reduced load method

Load ratio is not considered in the simplified rules. However, it certainly has influence on the fire resistance of the connections according to experimental tests. The reduced load method gives the ultimate capacity of the connection under a given standard fire exposure or the maximum fire exposure time under a given load ratio. For unprotected connections, the reduced load method for bolts and dowels are valid where the thickness of the side plate is equal or greater than t_1 (mm):

$$t_1 = max\begin{cases} 50 \\ 50 + 1.25(d - 12) \end{cases} \tag{3}$$

where d is the diameter of bolt or dowel (mm).

The characteristic ultimate capacity of a connection $F_{v,Rk,fi}$ with fasteners in shear should be calculated as:

$$F_{v,Rk,fi} = \eta \cdot F_{v,Rk} \tag{4}$$

with

$$\eta = e^{-kt_{d,fi}} \tag{5}$$

where k is the parameter given in Table 6; $t_{d,fi}$ is the design fire resistance of the unprotected connection.

Table 6. Values of parameter k

Connection with	k	Maximum period of validity for parameter k in an unprotected connection (min)
Nails and screws	0.08	20
Bolts wood-to-wood with d ≥ 12 mm	0.065	30
Bolts steel-to-wood with d ≥ 12 mm	0.085	30
Dowels wood-to-wood with d ≥ 12 mm	0.04	40
Dowels steel-to-wood with d ≥ 12 mm	0.085	30

Table 7. Values of parameter k_{fi}

Connection type	k_{fi}
Connections with fasteners in shear and side members of wood	1.15
Connections with fasteners in shear and side members of steel	1.05
Connections with axially loaded fasteners	1.05

The design fire resistance of unprotected connection $t_{d,fi}$ under the design effect of load action is calculated as:

$$t_{d,fi} \le -\frac{1}{k} ln \frac{\eta_{fi}\eta_0 k_{mod}\gamma_{M,fi}}{\gamma_m k_{fi}} \tag{6}$$

where η_{fi} is the reduction factor for design load in fire; η_0 is the degree of utilisation at normal temperature; k_{mod} is the modification factor; $\gamma_{M,fi}$ is the partial safety factor for timber; γ_m is the partial factor for the connection; k_{fi} is the parameter given in Table 7.

3.2. Calculated models in *Fire safety in timber buildings: Technical guideline for Europe*

The fire design method in *Fire safety in timber buildings: Technical guideline for Europe* (Östman B et al. 2010) can be considered as a complement to EC5. These studies are still at the research stage, but they still can be used as a basis for designing the connections under a certain dimension range. The application results in safe design.

The equation for timber-to-timber connections is valid for glulam members with strength class of GL24h and GL28. The equation for steel-to-timber connections is valid for glulam members with strength class of GL24h.They could also be used for connections composed of other wood species when the charring rate is equal of less than the charring rate of the tested glulam member. Connections should be designed according to EN 1995-1-1. The design value of load-carrying capacity of the connection $F_{d,t,fi}$ in shear under standard fire is determined by:

$$F_{d,t,fi} = \eta \cdot F_{v,Rk} \tag{7}$$

$$\eta \cdot = e^{-kt_{fi}} \tag{8}$$

where $F_{v,Rk}$ is the characteristic lateral ultimate capacity of the connection with fasteners in shar in normal temperature; k is defined in Table 8; t_{fi} is the required fire resistance period.

3.3. Calculated models by Audebert et al. (2020)

Three factors (diameter of the fasteners, thickness of the timber and load ratio) were considered in the formula proposed by Audebert et al. (Audebert et al. 2020). A polynomial function is fit considering the selected parameters. Then, a transformation was done to get a similar formula similar to the EC5. So the equation was transformed as:

$$\eta_{fi} = e^{\frac{\frac{t_{d,fi}}{k} - \ln\left(\alpha_1 \alpha_2^d \alpha_3^{t_1}\right)}{\ln\left(\alpha_4 \alpha_5^d \alpha_6^{t_1}\right)}} \tag{9}$$

where $t_{d,fi}$ is the design fire resistance of the connection; k is

Table 9. Values of coefficients k

Connection configuration and loading type	k
Steel-to-timber connections in tension parallel to grain	1
Timber-to-timber connections in tension parallel to grain	1.35
Steel-timber-steel connections in tension parallel to grain	0.40
Steel-to-timber connections in tension with an angle to grain of 90°	1.60
Steel-to-timber connections in tension with an angle to grain of 45° (steel plates are visible)	1.30
Steel-to-timber connections in bending	2.10
When only bolts are used without protection, k factor can be multiplied by 0.75	

Table 8. Values of coefficients *k*

Connection configuration and fastener type	Timber element	k	Maximum period of validity for parameter k (unprotected connections)
Timber-to-timber connection, bolts d ≥ 12 mm	t1 ≥ 45 mm	0.060	$t_{fi} \le 30$ min
Timber-to-timber connection, dowels d ≥ 12 mm	t1 ≥ 60 mm	0.035	$t_{fi} \le 60$ min
Steel-to-timber connection, bolts d ≥ 12 mm	t1 ≥ 45 mm	0.070	$t_{fi} \le 30$ min
Steel-to-timber connection, dowels d ≥ 12 mm	t1 ≥ 60 mm	0.045	$t_{fi} \le 60$ min

the modification factor considering the different connection types (Table 9); the coefficient $\alpha_1 \sim \alpha_6$ are given in Table 10; d is the diameter of the bolt (or dowel); t_1 is the thickness of the timber member; η_{fi} is the load ratio.

3.4. Calculated models by Palma et al. (2016)

The reduced load method in EC5 was improved by Palma et al. (Palma et al. 2016) considering not only the connection typology and load ratio but also the thickness of the timber side members. In Palma's method, the parameter k is dependent on the connection type and the thickness of timber side members. Therefore, the ultimate capacity of a connection R_{fi} after a given standard fire exposure t_{fi} is determined by:

Table 10. Values of coefficients $\alpha_1 \sim \alpha_6$

α_1	α_2	α_3	α_4	α_5	α_6
$7.327*10^{11}$	0.221	2.915	$5.476*10^{-4}$	1.632	0.787

Table 11. Values of coefficients c_1 and c_2

Connection		Requirements	k	Maximum fire resistance (min)
Nails and screws	Timber-to-timber	50mm ≥ t_1 ≥ 20 mm d ≥ 2.8 mm	0.1308-0.00195 t_1	min{13.3 + 0.33t_1;30}
Bolts	Timber-to-timber	60 mm ≥ t_1 ≥ 45 mm d ≥ 12 mm	0.0717-0.00039 t_1	min{–10.0 + 0.67t_1;30}
	Steel-to-timber	80 mm ≥ t_1 ≥ 45 mm d ≥ 12 mm	0.2089-0.00218 t_1	min{1.4 + 0.36t_1;30}
Dowels	Timber-to-timber	100 mm ≥ t_1 ≥ 50 mm d ≥ 12 mm	0.0447-0.00026 t_1	min{20.0 + 0.40t_1;60}
	Steel-to-timber	124 mm ≥ t_1 ≥ 45 mm d ≥ 12 mm	0.0711-0.00039 t_1	min{–15.9 + 0.61t_1;60}

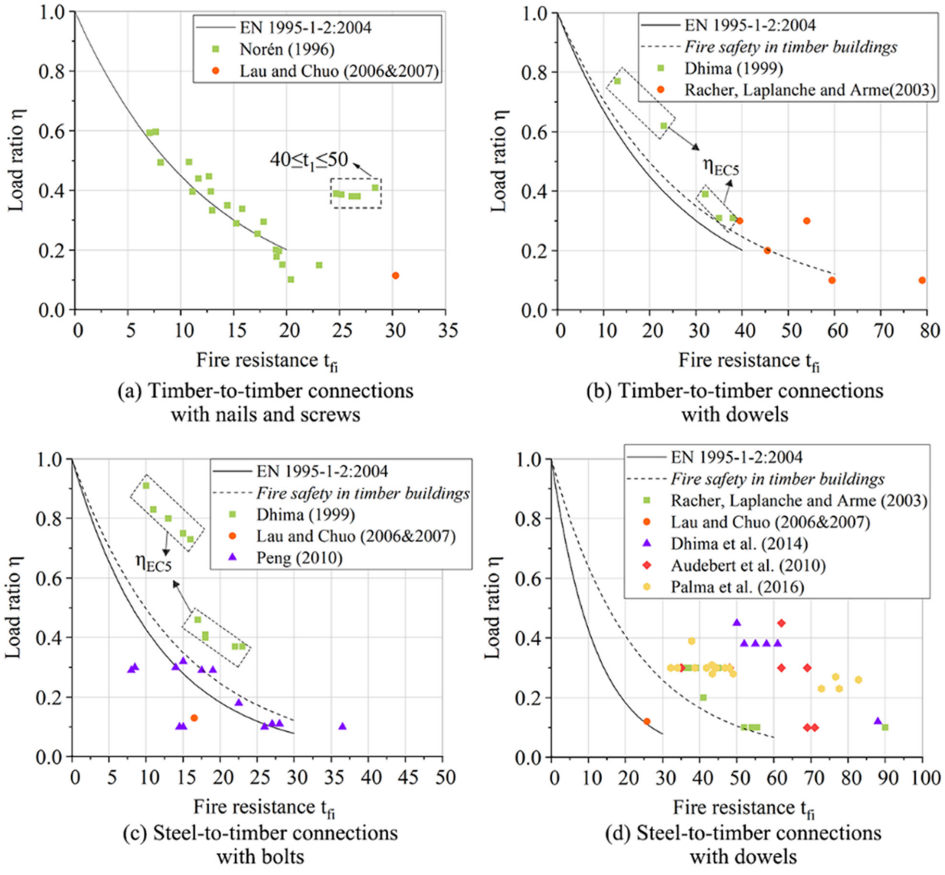


Figure 8. Two calculated models and experimental results.

$$R_{fi}=e^{-kt_{fi}}\cdot R_{20^{o}C} \quad (10)$$

where R_{20oC} is the ultimate capacity of the connection in normal temperature; parameter k is determined by:

$$k=c_1+c_2\cdot t_1 \quad (11)$$

where c_1 and c_2 are regression parameters considering connection typology and thickness of the timber side members. Vales of coefficients c_1 and c_2 are shown in Table 11.

3.5. Discussion

Two calculated models from reduced load method in EN 1995-1-2 and *Fire safety in timber buildings* are shown in Fig. 8. Experimental results from timber-to-timber connections with nails and screws, timber-to-timber connections with dowels, steel-to-timber connections with bolts and steel-to-timber connections with dowels are selected for comparison. Experimental results from timber-to-timber connections with bolts are not presented here due to little experimental data. For timber-to-timber connections with nails and screws, good agreement was achieved between the calculated models and experimental results except experimental results from connections with thicker timber side members (e.g. $40 \le t_1 \le 50$ in this test). Although the calculated model in EN 1995-1-2 provides conservative results for such connections with thicker side member. It is suggested to consider the thickness of the timber members as well to obtain more accurate results. For connections tests by Dhima in 1999, the real load ratio was unknown for most of the connections due to the absence of the experimental tests in normal temperature. Thus, η_{EC5} was used instead. η_{EC5} is determined by the ratio between the applied load in fire and the calculated load-carrying capacity in normal temperature according to EN1995-1-1. η_{EC5} is usually higher than the real load ratio as the calculated load-carrying capacity of the connections by EN1995-1-1 is always lower than those from real tests. In addition, for steel-to-timber connections with bolts, the calculated models from EN 1995-1-2 and *Fire safety in timber buildings* often overestimate the results. For bolts connections, these calculated models are needed to be improved. For steel-to-timber connection with dowels, the calculated results from EN 1995-1-2 and *Fire safety in timber buildings* are too conservative for connections loaded with an angle to the grain (Audebert et al. 2010), or in bending (Dhima et al. 2014), or in shear (Palma et al. 2016).

To improve the fire design rules of timber connections from EN 1995-1-2, two new calculated models were proposed by Audebert et al. (Audebert et al. 2020) and

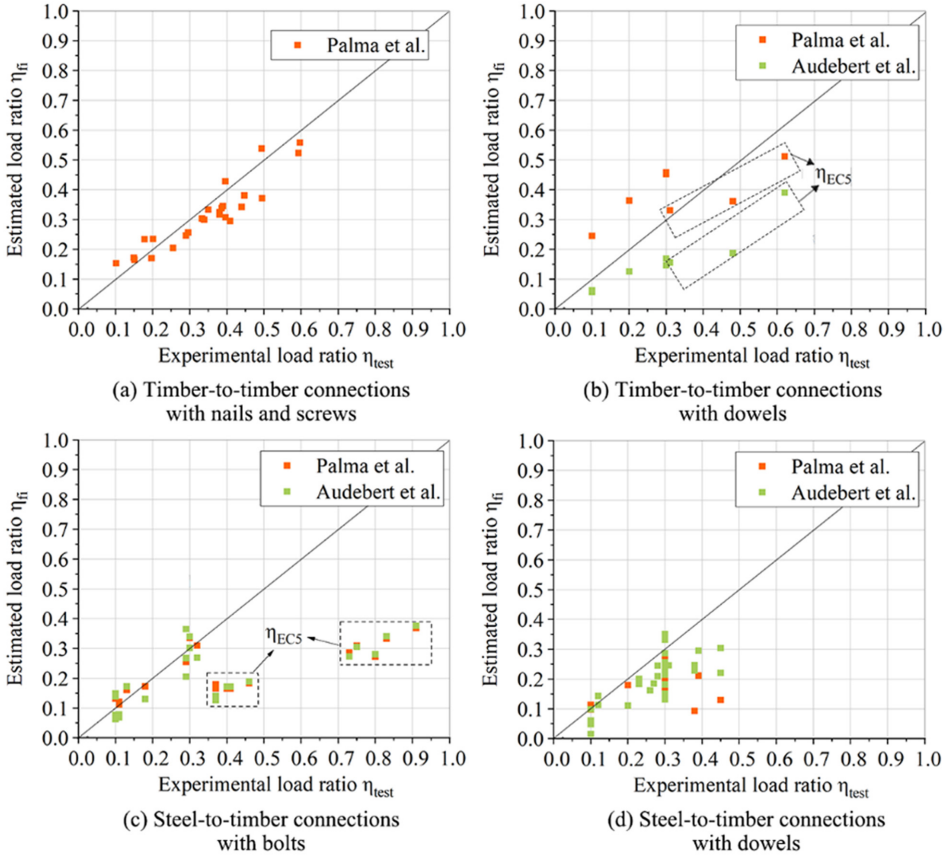


Figure 9. Two improved calculated models and experimental results.

Palma et al. (Palma. 2016) respectively. Those two calculated models were developed based on the experimental tests during the last three decades, which are mainly presented in this paper. So the calculated results from the two improved models are in good accordance with the presented experimental data (Fig. 9). In addition, the calculated model by Audebert et al. is also valid when the fire resistance value greater than 30 min, while the maximum period of validity for the fire design rules of connections in EN 1995-1-2 is 30 min or 40 min. The calculated models by Palma et al. also improve the valid limit of fire resistance for dowels connections to 60 min. However, some restrictions for the fastener diameter, side timber member as well as the maximum fire resistance that can be achieved for bolts connections are also needed to be concerned (Table 11). The two new improved methods proposed by Audebert et al. and Palma et al. can also be used by engineers to design the dowel-type timber connections in fire as a complement to EC5.

4. CONCLUSIONS

The main conclusions can be drawn as follows:

1. Previous experimental tests of dowel-type connections in fire during the last three decades are collected. The type of the fasteners (e.g. nails, screws, bolts or dowels), the connection configurations (e.g. WWW connections, WSW connections, SWS connections), and the load type (e.g. connections loaded in parallel to the grain, loaded with an angle to the grain, loaded in bending or in shear) are varied among those tests. They can be used as good reference for research and design of the dowel-type connection in fire.
2. Fire design rules of timber connections in EN 1995-1-2 and complemented models in *Fire safety in timber buildings* are presented in this paper. Although EN 1995-1-2 provide an easy way for engineers to design the timber connections in fire, there are still some limitations in this code. The thickness of the timber side member is one of the key factors influencing the fire resistance of the connections, but it is not considered in the formulas of the reduced load method in EN 1995-1-2. In addition, EN 1995-1-2 often overestimates fire resistance of the bolts connections and provides too conservative results for steel-to-timber dowels connection loaded with an angle to the grain, or in bending, or in shear.
3. Results of two improved calculated models by Audebert et al. and Palma et al. are in good accordance with the presented experimental data. These two models can be used as a complement to EC5. Some restrictions in these models are needed to be concerned. More experimental supports are still required such as new connection configurations and longer fire exposure.

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