Fire Technology

DOI: 10.1007/s10694-013-0369-y

Performance of Timber Connections Exposed to Fire: A Review

*C. Maraveas**, *K. Miamis and Ch. E. Matthaiou, C. Maraveas Partnership—Consulting Engineers, 11, Metsovou Street, 106 82 Athens, Greece*

Received: 11 February 2013/**Accepted:** 5 November 2013

Abstract. Fire safety has always been a major concern in the design of timber construction. Even though wood is a highly combustible material, timber members can perform adequately under elevated temperatures. The thermal response of timber connections, however, is in most cases poor and determination of their fire resistance is usually the crucial factor in evaluating the overall load-bearing capacity of wood structures exposed to fire. The analysis of timber joints under fire conditions can be challenging due to their complexity and variety. After presenting the variation of the properties of timber with temperature, this paper reviews the fire performance of various connection types, such as bolted or nailed wood-to-wood and steel-to-timber joints. Results from relevant experimental programs and numerical studies are discussed in detail and future research needs are highlighted. The effect of several factors on the fire resistance of timber connections, such as the fastener diameter, timber thickness and joint geometry, is investigated and useful conclusions are drawn. Based on these, preliminary guidelines for the efficient design of timber connections under fire exposure are presented.

Keywords: Elevated temperatures, Fire, Wood, Timber, Connections, Wood–steel connections

1. Introduction

Wood is widely used in contemporary construction for architectural/aesthetic purposes and because it provides cost-efficient design solutions. Its combustible nature, however, makes timber structures vulnerable to fire. An appropriate fire design approach for timber members alone is not sufficient if the joints are not capable of sustaining the applied load in a fire situation. This observation suggests that the design of the connections, which are typically the weakest zones in timber construction, will most probably determine the overall fire resistance of the structure.

Despite recent efforts to investigate experimentally and simulate the behavior of timber connections under elevated temperature effects, their thermal response is not fully understood yet. This can be attributed to the complex nature of these connections (i.e. the different joint types used, their geometric characteristics, etc.), the degradation of the properties of wood with temperature increase, the applied load, the nature of fire protection (if any) and other phenomena such as charring

* Correspondence should be addressed to: C. Maraveas, E-mail: c.maraveas@maraveas.gr

Published online: 19 November 2013

of wood, heat transfer among steel and wood surfaces, etc. This article presents a literature review on the behavior of wood–wood–wood (WWW), wood–steel–wood (WSW) and steel–wood–steel (SWS) connections under fire exposure.

2. Fire Definition

For the majority of the tested connections presented in this paper, the temperature evolution in the furnace was according to the ISO 834 [1] standard time–temperature curve. In certain cases, however, this curve could not be followed because of the heating method used (e.g. electrical coil instead of gas burner) or the furnace and a correlation (based on the fire severity) between measured and presented fire resistances was made. Moreover, in one test program, fire exposure followed the CAN/ULC-S101 [2] standard fire curve. Most researchers tested their specimens in custom-made, small-scale furnaces with different geometries and heating capabilities. Convection and radiation effects vary according to the geometric characteristics and size of the furnace, because they depend on the location of the specimen with respect to the burner, the proximity to the walls, etc. As a consequence, heat transfer to specimens tested by different authors was not identical. This observation provides a reasonable explanation for the scatter and discrepancies among the fire resistances reported in the literature (Sect. 7).

As mentioned earlier, researchers exposed their specimens to standard fires, which are generally suited for experimental testing. In such fires, temperature rises rapidly during the initial stage of exposure (e.g. $T \approx 680°C$ at $t = 10$ min and $T \approx 840°C$ at $t = 30$ min according to the ISO 834 [1] curve) and then increases steadily at a slow rate. High temperature values (greater than 1,000°C) are reached after several hours of exposure. This is not the case for natural fires, in which temperature evolution depends on many parameters, such as the combustible material (fuel), the openings in a compartment, etc. Before flashover, temperature rises at a slower rate. Afterwards, it reaches a peak, which is followed by a steady decline (decay phase of the fire). Despite these differences in time–temperature evolution, peak temperature values in natural fires are almost always lower than those in standard fires, especially after prolonged exposures. This fact results in thermal profiles that have a more onerous effect on fire resistance. Therefore, experimental results obtained from standard fire exposure are more conservative and can be safely used in the design of timber connections subjected to real fires.

3. Properties of Timber at Elevated Temperatures

3.1. Thermophysical Properties

The fire performance of timber connections depends on the thermophysical properties of their components at elevated temperatures. Researchers investigated experimentally and numerically the variation of thermal conductivity [3–6] specific heat [3, 4, 7] and density [4, 8, 9] of timber and charcoal with temperature. Figures 1 and 2 present a plot of the collected data together with relevant mathematical expressions

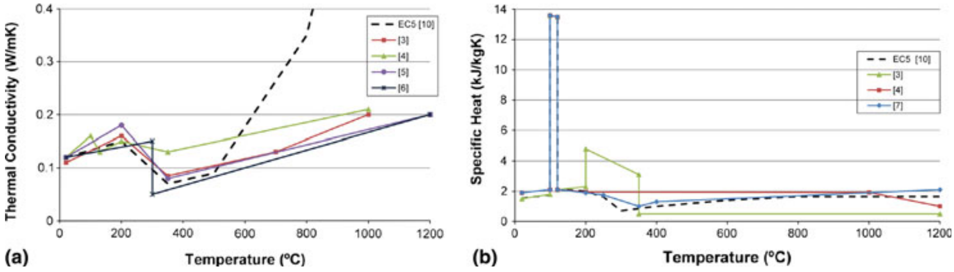

Figure 1. Plot of (a) thermal conductivity and (b) specific heat of wood/charcoal versus temperature.

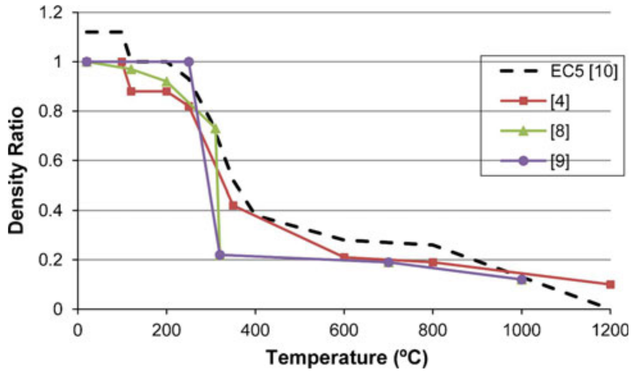

Figure 2. Wood/charcoal density reduction with temperature.

given in EN1995-1-2 [10]. The steep increase of thermal conductivity beyond 600°C, which was proposed by the authors of the Eurocode [10], is not reflected in the data encountered in the literature. It should be noted that thermal conductivity depends on grain orientation. The data presented here correspond to across-grain measurements. It has been reported [11] that the parallel-to-grain values are 1.5 to 2.8 times higher. In relevant numerical simulations, authors [11] correlated the transverse and longitudinal thermal conductivities by a factor of two. The sharp peak in the specific heat around 100°C, which occurs due to moisture evaporation [11], is not reported by all authors. Observed discrepancies suggest that these properties are not well defined yet. On the other hand, the thermophysical properties of steel at elevated temperatures have already been determined [12] accurately and are not discussed here.

3.2. Mechanical Properties

The mechanical properties of wood and steel at elevated temperatures are also important in determining the capacity of timber connections subjected to fire. For steel, their variation with temperature has been studied extensively and is reported

Table 1
Mechanical Properties of LVL and Sawn Timber at Room Temperature [25]

	LVL	Sawn timber
Bending strength (MPa), f_b	42	10
Tension parallel to grain (MPa), f_t	27	4
Compression parallel to grain (MPa), f_c	34	15
Shear in beams (MPa), f_s	4.50	3.80
Compression perpendicular to grain (MPa), f_p	12	8.90
Elastic modulus (MPa), E	13,200	6,000
Shear modulus (MPa), G	660	400

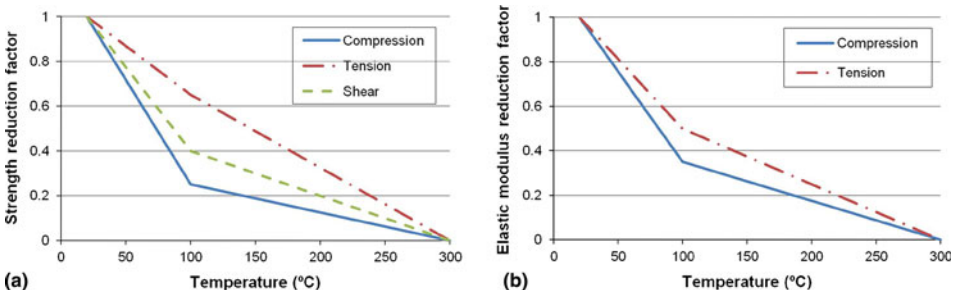

Figure 3. Variation of parallel-to-grain (a) strength and (b) Young's elastic modulus of wood with temperature according to EN1995-1-2 [10].

in EN1993-1-2 [12]. This is not the case for wood, as its mechanical properties vary considerably among different types of timber and depend on the direction of the loading with respect to grain orientation. To illustrate this, a relevant comparison (for ambient conditions) between laminated veneer lumber (LVL) and sawn timber is given in Table 1. EN1995-1-2 [10] proposes formulas for their reduction with temperature (Fig. 3).

4. Charring Rate

Application of sufficient heat to wood leads to a process of thermochemical decomposition, called pyrolysis, which results in alteration of its chemical composition and physical appearance (formation of char). This phenomenon is accompanied by mass loss. In timber structural members, charring appears in all exposed surfaces in the form a layer which increases in depth with the progression of fire. According to EN1995-1-2 [10], the position of the charred layer coincides with that of the 300°C isotherm. This is verified by other authors [13]. The charred layer cannot carry structural loads (resulting in a reduction of the cross-section) but acts as thermal insulation for the remainder of the cross-section [13, 14]. The charring rate is defined as the ratio of the charred depth divided by the duration of fire and is considered to be constant with time. This is a common assumption for exposure to standard fire and is based on the principles of one-dimensional heat transfer, which hold true for members used in typical timber construction. The notion of a constant charring rate has been adopted by EN1995-1-2 [10]. Moreover, authors [13] refer to experimental results which verify this assumption for softwood. The importance of charring rate in the fire design of timber construction has already been emphasized [15]. Eurocode 5 [10] proposes a charring rate of 0.65 mm/min for softwoods and a slightly larger value if the effect of corner rounding is included. Experimental results (Table 2) from various authors [15–21] show that the charring rate of WWW, WSW and SWS connections can be estimated as 0.65, 0.70 and 0.50 mm/min, respectively. Research results confirm the conclusion that the charring rate of WSW connections is slightly higher than that of WWW joints because of the steel plates [22].

5. Design According to Eurocode 5

5.1. Timber Members

In contrast to steel and reinforced concrete construction, a different fire design approach must be employed for timber structures due to charring of wood and its effect on the resistance of load-bearing elements. EN1995-1-2 [10] proposes two methodologies for calculating the resistance of timber members exposed to fire.

Table 2
Measurements of Charring Rate for Timber Connections

Type of connection	Ref.	Type of fastener	Experimental charring rate (mm/min)
WWW	[16]	Bolts	0.67
		Screws	0.71
	[17]	Bolts	0.70
	[18]	Bolts	0.58
		Dowels	0.65
	[19]	Dowels	0.70
WSW	[1]	Bolts	0.75
	[16]	Bolts	0.60
		Dowels	0.70
	[17]	Bolts	0.82
	[18]	Bolts	0.53
	[20]	Bolts	0.80
	[20]	Dowels	0.81
			0.68
SWS	[15]	Bolts	0.41
	[16]	Bolts	0.57
		Nails	0.66
	[17]	Bolts	0.35

According to the "reduced cross-section method", an ineffective zone around the exposed perimeter is defined. Its depth equals that of the charred layer plus an additional 7 mm layer, which is formed linearly with time during the first 20 min of fire exposure. Based on this zone, a reduced cross-section is determined and the strength of the member is calculated using the mechanical properties of timber at ambient temperature conditions. On the other hand, the "reduced properties method", defines temperature dependent factors for reducing the mechanical properties of the residual cross-section. In this case, the latter is determined by taking into account only the charred layer.

5.2. *Timber Connections*

EN1995-1-2 [10] provides design rules for symmetrical three-member connections with various types of fasteners (nails, bolts, dowels, etc.) exposed to the ISO-standard fire. These apply to laterally loaded joints (i.e. the connectors are subjected to shear) and are generally limited to fire resistances less than 60 min. The design can be performed either by: (a) application of simplified rules or (b) by the "reduced load method". According to the first approach, the fire resistance of unprotected connections with dowels is considered to be 20 and 15 min for joints with other connector types. If a greater fire rating is desirable, the edge distance as well as the thickness and width of the side members should be increased. This is only applicable to connections with screws, nails and dowels, while the enhanced fire resistance cannot exceed 30 min. For higher fire ratings, application of fire protection is necessary and specific rules depending on the connection type are given. According to the "reduced load method", the load bearing capacity of the connection under fire exposure is obtained by reducing the room temperature capacity via a conversion factor η, given in Eq. (1), which incorporates the design fire resistance of the unprotected connection $t_{\mathrm{d,fi}}$ (in min) and a dimensionless parameter k depending on the connection type:

$$\eta = e^{-k \cdot t_{\mathrm{d,fi}}}. \tag{1}$$

Specifications for calculating the resistance of joints with axially loaded screws under elevated temperatures are also provided. These take into account the configuration of the connection, the edge distance and the embedment depth of the screws. It should be noted that the fire design of timber structures per EN1995-1-2 [10] has been discussed in detail by König [23, 24], who also provided information regarding the background of the specific code.

6. Embedding Strength

Another parameter that plays an important role in determining the capacity of timber connections is the embedding strength of the fasteners in wood. Austruy [25] defined it as "the average compressive stress at maximum load at the interface between a timber specimen and a stiff linear fastener, with the fastener's axis perpendicular to the surface of the specimen." At ambient temperature, the

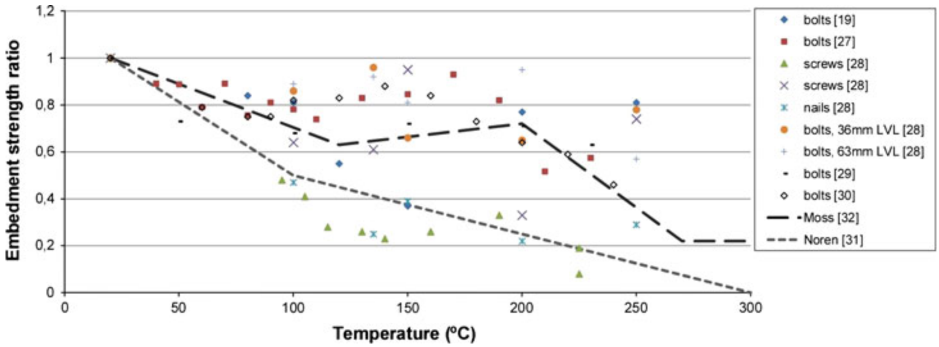

Figure 4. Variation of the embedding strength with temperature in timber connections.

embedding strength is a function of the timber density, the bolt diameter and the angle of the load with respect to the grain. EN1995-1-1 [26] provides numerical expressions for its calculation depending on the type of the connectors and the timber class.

The variation of the embedding strength with temperature has been experimentally investigated by researchers [17, 27–30]. Figure 4 shows a plot of the collected test data. Results refer to specimens loaded parallel to the grain. Besides experiments of bolted connections, Moss et al. [28] also tested specimens with screws and nails. The same authors [28] conducted tests of joints with loading perpendicular to the grain which are not presented here. The reduction pattern and the relative minimum around 100°C have already been reported [27]. An expression to correlate the embedding strength with temperature was proposed by Noren [31]. This was based on experiments of nailed connections and takes into account the parallel to grain compressive strength of wood as well as the moisture content. Moss et al. [32] also formulated tri-linear approximations to match the data from a relevant experimental program [28]. It can be observed that Noren's expression [31] matches well the data for joints with nails and screws, while the approximation proposed by Moss et al. [32] is comparable with results pertaining to bolted connections.

In the tests carried out by Chuo [17], heating of the specimens was not prolonged and resulted in a variable temperature profile. This was also the case for the screwed connections exposed to 2-h heating in the experimental program conducted by Moss et al. [28]. For the rest of their tests [28], the thermal profile of the joints was uniform due to long-duration fire exposure. Others [27, 29] also reported that 2.5 h heating led to a constant temperature throughout the cross-section of the specimens.

7. Fire Resistance of Timber Connections

7.1. Wood–Wood–Wood Connections

7.1.1. Experimental Studies. WWW connections usually consist of three timber members (two side members and a central one) joined together with various types of connectors (screws, bolts, nails, etc.). Steel plates are not used in this type of connection. Figure 5 presents a typical arrangement of a WWW connection together with the applied loading. Researchers [16–19, 25, 31] measured the fire resistance of such connections for different fastener types and layouts as well as varying member thicknesses. Members were typically made from LVL, while glued laminated timber GL28h (grade according to EN 1194 [33]) was used in some cases. The load ratio, which is defined as the applied load during the fire test divided by the ultimate strength of the connection at ambient temperature, ranged from 10 % to 30 % in most tests, with values as high as 65 % being reported. No fire protection was applied on the connections. Relevant information on the tests found in the literature is summarized in Table 3.

7.1.2. Discussion and Future Research Needs. Member thickness is one of the parameters that affect the fire performance of WWW connections. Figure 6, in which the fire resistance of unprotected specimens is plotted against wood thickness for various load ratios, shows that connections with thicker wood members can achieve higher fire ratings. This observation is reflected in current design specifications [10], which allow for higher fire resistances to be considered when member thickness is increased. Decrease in fire resistance is related to the charring of wood and the reduction of the embedding strength. Experimental results reported by [19] support this statement. More specifically, for a load ratio of 30 %, specimens with 84 mm side member thickness sustained the applied load 16 min longer than those with a thickness of 64 mm. For the specific connection configuration, a thickness increase of approximately 35 % led to improvement of the fire resistance by 40 %.

Emphasis should also be given on the effect of the applied load ratio. Experimental results show that its reduction increases the fire resistance. A relevant plot (Fig. 7) verifies this. For example, in results presented by Laplanche [19], the increase in fire resistance ranged from 20 min to 25 min for reducing the load ratio from 30 % to 10 %. Similarly, in connections with a side member thickness of 60 mm [18], the fire resistance dropped dramatically (approximately 28 min) when the load ratio was increased from 33 % to 65 %. It should also be noted that for load ratios ranging from 50 % to 65 %, the reported fire resistance is extremely low (7 min to 13 min). Despite considerable scatter of the collected data for fire resistances ranging between 20 min and 40 min (due to the effect of differ-

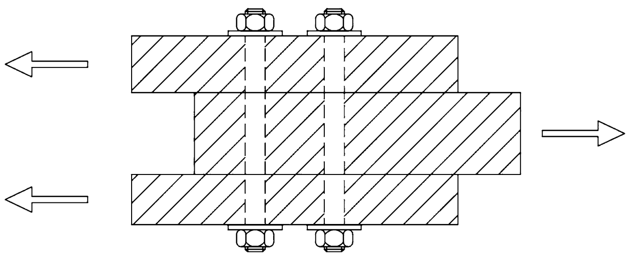

Figure 5. Typical arrangement of WWW connections.

Table 3
Fire Resistance Tests of WWW Connections Reported in the Literature

Ref.	Wood & fasteners	Side timber thickness (mm)	Central timber thickness (mm)	No. of fasteners	Diameter of fasteners (mm)	Applied load ratio	Fire resistance (min)
[25]	LVL, bolts	45	65	1	12	0.22	17.2
				6	12	0.21	20.4
[16]	LVL, bolts	45	63	6	12	0.15	20.5
[17]	LVL, bolts	45	63	1	12	0.14	17.5
				5	12	0.13	20.3
[18]	GL28, bolts	60	100	4	20	0.28	22.0
						0.56	14.0
		60	100	8	20	0.30	24.0
						0.59	15.0
		50	80	8	12	0.24	22.0
						0.57	13.0
[18]	GL28, bolts & dowels	60	100	6	20	0.33	35.0
						0.65	7.0
				12	20	0.21	38.0
						0.42	23.0
		50	80	6	12	0.28	32.0
						0.56	13.0
[19]	GL28, dowels	64	112	8	16	0.10	59.5
						0.20	45.5
						0.30	39.5
		84	160	8	16	0.10	79.0
						0.30	54.0
[16]	LVL, screws	45	63	17	6.3	0.15	30.3
[31]	Picea abies, nails	28	45	71	4	0.20	19.0
						0.20	19.2
						0.20	23.2
						0.30	17.9
						0.30	15.8
						0.40	11.2
						0.40	12.7
						0.50	8.2
						0.60	7.7
		40	45	84	4	0.40	24.7
						0.40	26.9
						0.40	25.2
		45	45	89	4	0.40	25.3
						0.40	26.4
						0.40	28.4
		20	45	52	2.8	0.10	19.9
						0.20	19.1
						0.20	18.8
						0.30	16.7
						0.30	14.9
						0.40	12.7
						0.40	12.5
						0.40	11.4
						0.50	10.4
						0.60	6.7

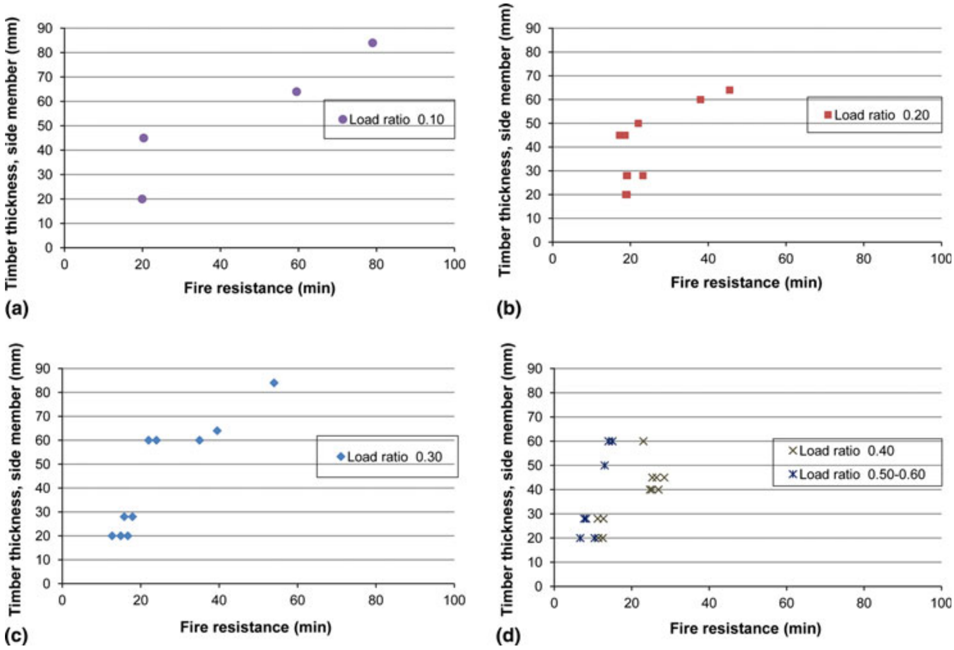

Figure 6. Variation of fire resistance with wood thickness for WWW connections.

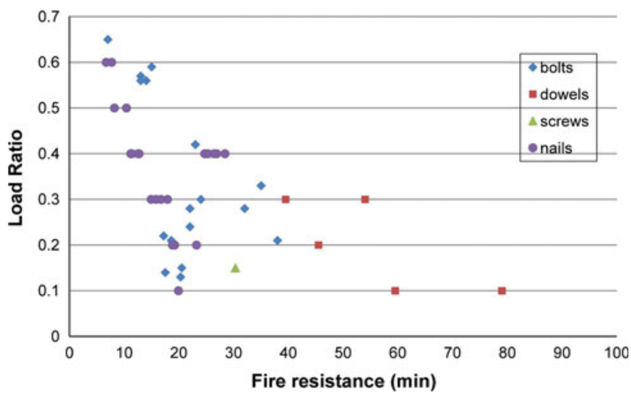

Figure 7. Load ratio versus fire resistance in unprotected WWW connections.

ent configurations, wood thickness, etc.), the load ratio reduces almost linearly with fire resistance. High fire rating values (greater than 60 min) correspond to doweled connections with a thick central member (greater than 110 mm).

Even though fasteners of different diameters were used in the tested connections, a direct comparison is only possible in the experiments presented by Dhima [18]. Connections with 12 mm bolts displayed approximately the same fire resistance with those in which 20 mm bolts were used. However, the load ratio was different (33 and 12 % respectively). Even though this implies that larger fasteners provide better fire resistance, it should be noted that the fastener number and arrangement were not identical. Further testing of connections with the same load ratio and connector layout should be conducted to verify this.

The effect of fastener type requires further experimental investigation. The collected experimental data refer to connections with bolts, dowels and screws. However, the applied load ratios were different. Results from one author [16] suggest that connections with screws have a greater fire resistance (approximately 10 min) compared with those with bolts, for the same load ratio. The same author [16] pointed out the increased heat transfer to timber resulting from the bolt head, which is the most probable reason for the reduced fire resistances of the bolted connections tested in his experimental series.

At ambient temperature, current codes [26, 34] account for net tension failure in WWW connections by setting minimum edge distance requirements according to the type of connector used. An edge distance of 1.5*d*, where *d* is the fastener diameter, is given in the Canadian Standard CAN/CSA-O86-09 [35]. However, under elevated temperature effects, the adequacy of the proposed edge distances must be reexamined due to the reduction of the cross-section resulting from charring. Experimental data considering this phenomenon are not extensive. The effect of edge distance in nailed WWW joints was studied by Noren [31], who reported that charring parallel to the side members led to a faster decrease in the load bearing capacity of edge nails and that these connectors failed in a different mode than those located at the middle. Others [19] reported that a 43 % increase of the material towards the edge led to a fire resistance increase of 60 and 30 %, for load ratios 10 and 30 % respectively. In order to avoid a premature failure of this type, authors [15] suggested that edge distances be increased by the product of the charring rate and the desirable fire resistance.

In future research work, the effect of fastener type on fire resistance should be further investigated by testing connections with identical load ratios and wood members. Configuration of the connectors should be similar. The temperature evolution in the region around the connectors is not well documented and understanding it is crucial. The need to determine the charring rate around the connectors has already been reported [16]. It should also be noted that tests of fire protected WWW connections have not been reported in the literature. The improvement of the fire resistance resulting from application of thermal insulation should also be investigated.

7.1.3. Numerical Analysis Studies. Few attempts to simulate the fire resistance of WWW connections have been conducted so far. A 3D finite element model was proposed to describe the behavior of dowelled WWW connections under fire exposure [36]. The model included uncoupled thermal and mechanical analyses. Initially, the temperature profile of the connection was determined by taking into account the appropriate heat transfer mechanisms (conduction, radiation, etc.) as well as the thermophysical properties of timber (its transformation to charcoal was included) and steel at elevated temperatures. A mechanical analysis followed, with plasticity being considered for both materials. Friction at the timber–steel interface was modeled according to the Mohr–Coulomb criterion. The numerical results compared well with the data from a relevant experimental program [19]. Others [25] also created a 3D finite element model to simulate the thermal response of bolted WWW connections subjected to elevated temperatures with limited success. However, for the bolts and wood region around them, analysis temperatures were in agreement with the measured ones close to the failure time [25]. Johansen's failure theory was then used to calculate the fire resistance, which matched the results of connections tested by the author accurately [25].

A component model for dowelled-type timber connections subjected to fire has also been proposed [37]. The authors initially modeled a single fastener connection at room temperature, in which the parameters of the timber component were selected from formulas given in EN1995-1-1 [26] and experiments, while the force–displacement curve for the dowel component was obtained by fiber analysis of its section. This model was expanded to include elevated temperature effects by applying reduction factors (as proposed by the EN1993-1-2 [12] and EN1995-1-2 [10]) to the material properties. The required temperature profile was obtained via 3D finite element simulation of the connection.

Despite some efforts, limited numerical analysis of WWW connections under fire has been conducted up to date. Even though 3D finite element simulations seem to describe their behavior in an accurate way, current analyses are limited to doweled connections. Further modeling of different configurations (namely varying wood member size, different type and number of fasteners, etc.) followed by comparison with a wider range of experimental results (current and future) is required. Moreover, the proposed component model [37] cannot directly assess the thermal field and requires the use of a 3D finite element model for its determination.

7.1.4. Failure Modes. The predominant failure type for the tested WWW connections is related to the reduction of the embedding strength around the region of the fasteners with temperature. Lau [16] reported that failure of the tested specimens occurred due to elongation of the bolt holes (Fig. 8a) combined with splitting of wood around that region and that bending of the bolts was not evident. A similar failure mode (splitting of wood near the screws) was observed for screwed connections (Fig. 8b) [16]. WWW connections tested in another experimental program [17], failed by crushing of the timber members around the region of the bolts. On the contrary, this failure mode was not reported in the experiments by Laplanche [19]. Because the load ratio of these connections was low (less than 50 %), failure occurred due to prolonged fire exposure times which led to charring and loss of strength of the wood members [22].

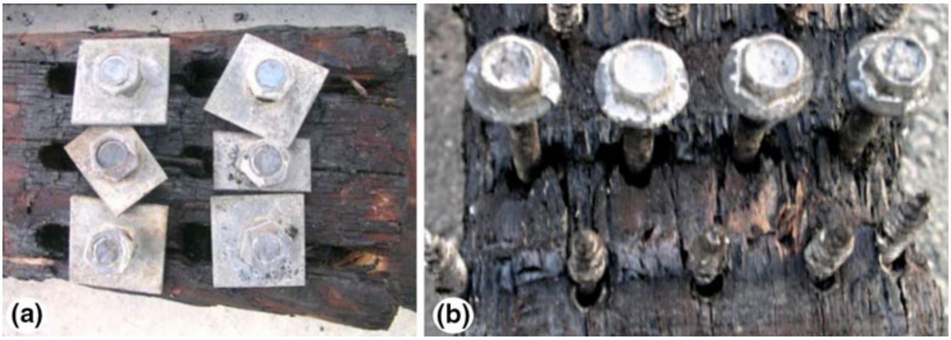

Figure 8. Failure modes observed in (a) bolted [16] and (b) screwed [16] WWW connections.

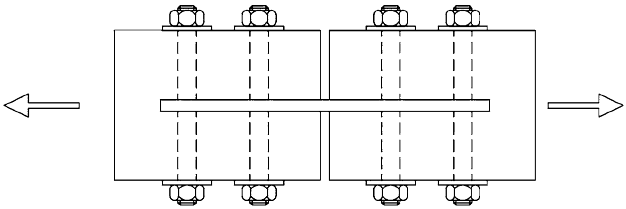

Figure 9. Arrangement of a typical WSW connection.

7.2. Wood–Steel–Wood Connections

7.2.1. Experimental Studies. The thermal behavior of WSW connections has also been experimentally investigated [16–21, 25, 38]. Such connections are typically made of four timber pieces (two on each side) connected together with a thin steel plate. Various types of fasteners can be used. Figure 9 presents a schematic of such a connection with the joined members loaded axially. A different arrangement was used in tests by Frangi et al. [38], who tested multiple shear steel-to-timber connections (three steel plates slotted in the timber members). Tables 4 and 5 summarize results from fire resistance tests of WSW connections reported in the literature. Information regarding the timber quality, (in most cases LVL or glue laminated timber-Glulam), the fastener type and quantity, the member dimensions and the load ratio (ranging from 0.10 to 0.58) is provided. In certain cases fire protection (intumescent paint on steel or gypsum boards/plywood) was applied.

7.2.2. Discussion and Future Research Needs. The results (plotted for selected load ratio cases in Fig. 10) show that timber thickness increase improves the fire resistance of WSW connections. In one source [21], for a load ratio of 0.1, thicker specimens (100 mm thickness) failed 38 min later than thinner specimens (75 mm thickness). The influence of timber thickness is not so strong for higher load ratios. In connections loaded at 30 % of their ultimate strength [21], it is observed that for a thickness increase of 23 mm (from 77 mm to 100 mm), the fire resistance

Table 4
Fire Test Results of Unprotected WSW Joints Reported in the Literature

Ref.	Wood & fasteners	Side timber thickness (mm)	Steel plate thickness (mm)	No. of fasteners	Diameter of fasteners (mm)	Applied load ratio	Fire resistance (min)
[25]	LVL, bolts	45	6	1	12	0.24	17.2
				5	12	0.18	15.0
[16]	LVL, bolts	45	6	5	12	0.14	16.5
[17]	LVL, bolts	45	6	1	12	0.14	16.5
				4	12	0.12	15.1
[20]	Lumber, bolts	38	9.5	2	12.7	0.10	14.7
						0.30	8.2
[18]	GL28, bolts	60	6	16	20	0.19	22.0
						0.39	15.0
				32	20	0.19	23.0
						0.38	16.0
		50	6	8	12	0.29	17.0
						0.58	10.0
				16	12	0.25	18.0
						0.49	11.0
				32	12	0.24	18.0
						0.48	13.0
[20]	Glulam, bolts	60	9.5	4	12.7	0.11	28.0
						0.18	22.5
						0.29	17.5
				2	19.1	0.11	27.0
						0.32	15.0
				4	19.1	0.10	26.0
						0.30	14.0
		80	9.5	4	19.1	0.10	39.8
						0.29	19.0
[21]	GL28, dowels	76	8	8	16	0.10	55.5
						0.20	41.0
						0.30	36.0
		75	10	8	20	0.10	52.0
						0.30	37.0
		77	6	8	12	0.10	54.0
						0.30	39.0
		100	10	8	20	0.10	90.0
						0.30	45.0
[16]	LVL, dowels	45	6	32	5	0.12	25.8
[19]	GL28, dowels &bolts	60	10	6 dowels 2 bolts	20	0.10	50.0
						0.20	40.5
						0.30	35.5

Table 4 continued

Ref.	Wood & fasteners	Side timber thickness (mm)	Steel plate thickness (mm)	No. of fasteners	Diameter of fasteners (mm)	Applied load ratio	Fire resistance (min)
[38]	GL24h, dowels	37 side 54 central	3 × 6	18	6.3	0.075	41.0
						0.15	33.0
						0.30	33.0
				18	6.3	0.30	73.0
				9	6.3	0.30	32.5
		44 side 100 central	2 × 6	8	12	0.30	34.5
[38]	GL36h, dowels	37 side 54 central	3 × 6	27	6.3	0.30	31.0

improved only by 5 min. On the contrary, the thickness of the steel plate does not seem to affect the fire resistance of joints loaded at the same load ratio. This is confirmed by experiments from Ayme [21], in which specimens with different steel plate thickness (6 mm and 10 mm) failed approximately after the same fire exposure time (the thickness of the side wood member was 77 mm and 75 mm respectively), as well as results from other authors [38]. As expected, the temperature in the protected (by the timber members) steel plate will remain low regardless of its thickness and, consequently, the fire resistance will be determined from the failure of the wood section.

Figure 11 presents a plot of the applied load ratio versus the fire resistance for the unprotected WSW connections found in the literature. Despite considerable scatter, the data confirm the anticipated increase in fire rating with load ratio reduction. In one source [20], for specimens with side members of 60 mm thickness, the fire resistance increased approximately 10 min to 13 min when the load ratio dropped from 30 % to 10 %. For the same reduction in the load ratio, other authors [21] reported improvement of the fire rating by 15 min to 20 min for side members with thicknesses ranging from 75 mm to 77 mm. The scatter in the data is justified by the various parameters affecting fire resistance, such as the arrangement of the tested connections, the member thickness, the type of the connector, the variability of heat transfer among different test programs, etc. Despite this observation, the derived plot shows that the load ratio can provide insight regarding the fire resistance of WSW connections. One specimen (multi-shear connection) [38] with a load ratio of 0.30 had an exceptionally high fire resistance (73 min) and is not consistent with the rest of the data.

It might be expected that a greater fastener diameter would reduce the fire resistance under a constant load ratio, due to increased heat transfer to wood near the vicinity of the connector. However, the collected results suggest that variation of the fastener diameter does not influence the fire resistance in a major way, as stated by others [20]. Observation of Fig. 12 confirms this. In experiments involving connections with bolt diameters of 12.7 mm and 19.1 mm [20], the difference in fire resistance was negligible (two to three min) when the same load ratio was

Table 5
Fire Test Results of Protected WSW Joints Reported in Literature

Ref.	Wood & fasteners	Type of protection	Side timber thickness (mm)	Central steel plate thickness (mm)	No. of fasteners	Diameter of fasteners (mm)	Applied load ratio	Fire resistance (min)
[16]	LVL, bolts	Intumescent paint	45	6	5	12	0.14	19.7
[20]	Glulam, bolts	Gypsum board	60	9.5	4	19.1	0.3	51
[20]	Glulam, bolts	Douglas fir plywood	80	9.5	4	19.1	0.29	34
[16]	LVL, dowels	Intumescent paint	45	6	32	5	0.12	29.0
[38]	GL24h, dowels	Three-layered timber boards	37 side 54 central	3 × 6	27	6.3	0.30	64.5
[38]	GL24h, dowels	Gypsum board	37 side 54 central	3 × 6	27	6.3	0.30	60.5

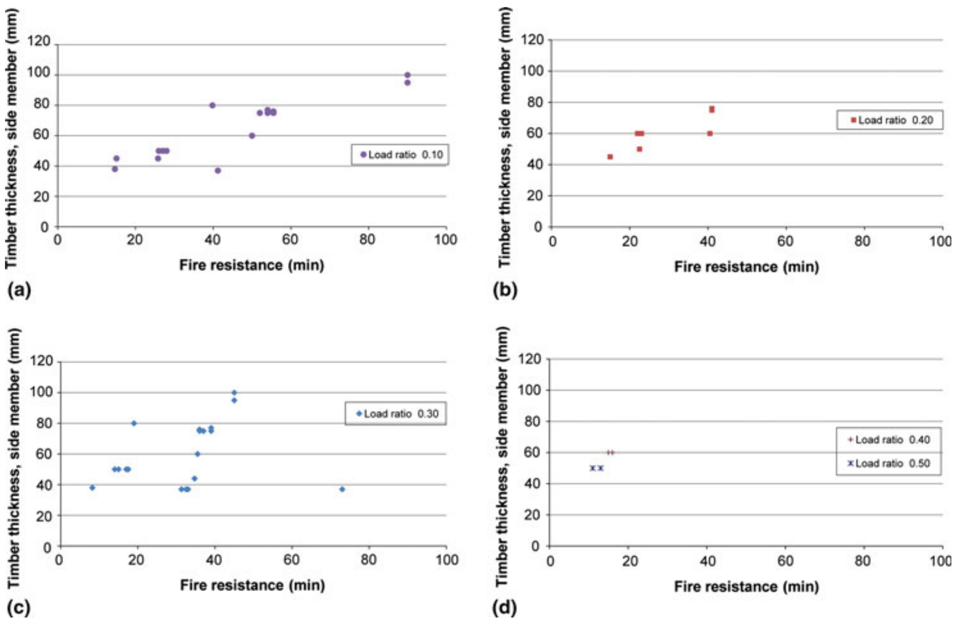

Figure 10. Effect of timber thickness on the fire resistance of tested WSW connections.

applied. This is also valid for other tests [22], in which 12 mm and 20 mm bolts were used (identical specimen configuration).

The effect of edge distance on the fire rating of WSW joints has also been investigated experimentally. A relevant study was conducted by Peng et al. [15], who reported that a 40 % increase led to improvement of the fire resistance by approximately 20 %. These authors [20] stated that reduction of the edge distance resulting from charring leads to premature failure of the connection. Increase of the edge distance also enhanced the fire resistance of multi-shear steel-to-timber connections [38]. Further research is required, however, to clarify the correlation between edge distance and fire resistance for this connection type.

Another parameter that influences the fire resistance of WSW connections is the fastener type. In the studied experiments, only bolts and dowels were used. For specimens [16] with the same geometric characteristics and connector layout, the connections with dowels sustained the applied load 9 min longer than those with bolts. This can be attributed to the increased heating rate of bolts, due to the larger area exposed to temperature effects (existence of bolt head, nut, washer, etc.) [39]. Based on numerical analysis, Audebert et al. [39] stated that, in steel-to-timber connections, heating of the bolts is two times greater than that of dowels. A similar finding has been reported by another author [19].

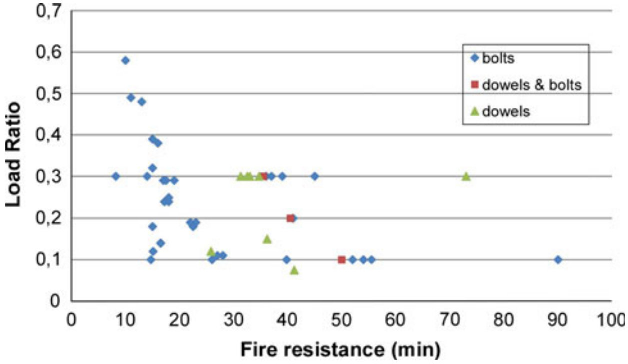

Figure 11. Fire resistance of tested WSW connections versus the applied load ratio.

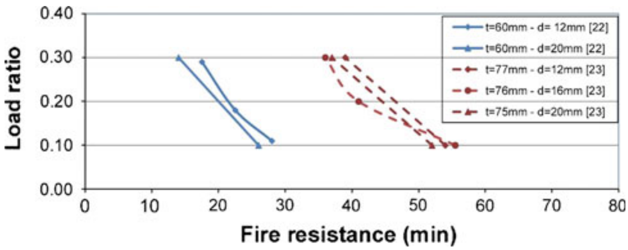

Figure 12. Effect of fastener diameter on the fire resistance of tested WSW connections.

The presence of fire protection increases significantly the fire resistance of WSW connections. From the collected data, it can be observed that the use of gypsum board increased the fire resistance by about 37 min [20] (250 % increase). The presence of plywood was also beneficial, as the fire rating improved by 15 min [20] (130 % increase). The use of intumescent paint on the steel plate was not as effective, because it led to an increase of only 3 min [16]. This is to be expected, as the major portion of the steel plate is protected from both sides by the timber members.

Despite considerable efforts, the experimental database of WSW connections under fire conditions is still incomplete. Tests for heavily-loaded joints are few and their thermal response is neither well documented nor well-understood. An extensive experimental study focusing on the effect of the parameters discussed should be carried out. Furthermore, results from Frangi et al. [38] show that other variants of this connection type could lead to improvement of the fire resistance.

7.2.3. Numerical Analysis Studies. The simulation of WSW connections under elevated temperature effects has been the object of current research. The finite element model generated to simulate WWW connections under fire exposure by

Racher et al. [36] was also applied to dowelled WSW joints. Results were in good agreement with relevant experimental data [19].The thermo-mechanical behavior of similar joints loaded in tension parallel to the grain was simulated via 3D finite element analysis by others [39]. A distinction was made between the thermal and the mechanical models, which were validated against experimental results [39]. Comparisons involved temperature evolution, charring rate and evolution of the load slip curve along the timber–steel interface. Afterwards, the separate models were combined into a single numerical model for thermo-mechanical analysis. Audebert et al. [40] used the finite element method (FEM) to evaluate the distribution of loads among the fasteners under fire exposure. The evolution of these loads with time was also reported. The same authors [40] proposed a simplified analytical formula (Eq. (2)) to predict the fire resistance of WSW connections, based on the load ratio η, the diameter of the fastener d and the thickness of the timber member t_1:

$$t_f = 48.47 - 0.46 \cdot d + 0.22 \cdot t_1 - 8.68 \cdot \ln(\eta) + 0.015 \cdot d \cdot t_1 - 0.22 \cdot d \cdot \ln(\eta) + 0.0075 \cdot t_1 \cdot \ln(\eta). \tag{2}$$

Thermal analysis (via the FEM) of WSW connections with bolts and dowels exposed to fire has also been conducted by others [11, 41], who presented temperature distributions in the timber section and the connectors and used the reduced embedding strength approach to calculate the joint capacity under elevated temperature effects. Based on results from this analytical model and experimental data, these authors [11] also derived a mathematical expression (Eq. (3)) to calculate the fire resistance of such connections considering three parameters (load ratio, fastener diameter and timber thickness):

$$t_f = -0.0042 \cdot \ln(\eta) \cdot \left(t_1^2 + 2 \cdot d^2\right). \tag{3}$$

A similar correlation, given in Eq. (4), was proposed in other work done by these authors [20]:

$$t_f = (t_1/\beta) \cdot \left(1 - \eta^M (d/t_1)^N\right), \tag{4}$$

where $M = 0.15736$, $N = 0.06004$ and $\beta = 0.8$ mm/min.

The temperature distribution of one-bolted and four-bolted connections was also simulated via a commercial FE program [25]. However, the author stated that comparison with experimental data from a relevant program [25] led to inconclusive results. Despite this, the calculated failure times (based on Johansen's approach) matched closely the experimental values for multi-bolted connections (approximately 5 % difference) [25].

The thermal response of multi-shear connections tested in Switzerland [38] was simulated via the FEM [13, 42], with the authors reporting the temperature distribution in the timber section and the steel plates after different times of fire exposure. In another work published by Erchinger et al. [13], an analytical model for calculating the load-bearing capacity of such connections under elevated temperature effects was presented. The same authors [13] also proposed mathematical expressions for determining the effective cross-section based on the number of dowels, their diameter, the fire exposure time and the dimensions of the residual section.

The FEM is the predominant method for analyzing WSW connections under elevated temperature effects. The simulation of the thermal field and the thermo-mechanical response seem to be credible, because they match the experimental results with sufficient accuracy. The analytical expressions found in the literature yield different results. Table 6 shows the error among the values calculated from Eqs. (2) to (4) and the experimental results. It can be observed that the correlation proposed by Peng et al. [20] matches the test data found in the found in the literature well for bolted connections (the average error is 11 %). This is not the case for doweled joints (a mean error of 38 % was calculated). On the contrary, the equations by Peng et al. [11] and Audebert et al. [40] can only be used for doweled connections, because for bolted joints errors greater than 100 % were calculated in certain cases.

7.2.4. Failure Modes. Various failure modes for this connection type have been reported in the literature. Experimental results from [15] show that elongation of the bolt holes (Fig. 13a) occurred in all specimens. In certain cases, this mode was combined with a shear-through failure (Fig. 13b), splitting of wood (Fig. 13c) or tear-out of the edge material (Fig. 13d). Other bolted (Fig. 13e) and dowelled (Fig. 13f) connections failed via splitting of wood around the fasteners [16]. Bolt hole elongation was observed for the WSW connections tested by others [17]. However, Laplanche [19] reported block shear of the wood section along the connector lines. Failures of heated multi-shear connections were similar to those with a single steel plate [38] and typically involved elongation of the holes and splitting of wood [38].

7.3. Steel–Wood–Steel Connections

7.3.1. Experimental Studies. Another connection type commonly encountered in timber structures is that in which the joined timber members (one on each side) are sandwiched between two steel plates. Figure 14 presents a typical arrangement of a bolted SWS connection loaded in tension. Experimental work regarding the fire performance of such connections has been carried out recently [15–17, 25, 38]. Tables 7 and 8 present a summary of the geometric characteristics, the load ratio and the measured fire resistance of the tested connections found in the literature. In most cases LVL was used in conjunction with bolts, while two authors [16, 38] selected to test nailed connections. Buchanan and King [43] tested connections with steel gusset plates and reported that they behaved poorly when no fire protection was applied. In certain cases, fire protection materials, namely intumescent paint or gypsum boards, were applied to improve the fire resistance of the tested specimens.

Table 6
Fire Resistance of WSW Connections-Comparison Among Experimental and Analytical Expressions

Ref.	Wood & fasteners	Side timber thickness (mm)	Diameter of fasteners (mm)	Applied load ratio	Experimental fire resistance (min)	Ref. [4]	Error (%)	Ref. [25]	Error (%)	Ref. [14]	Error (%)
[25]	LVL, bolts	45	12	0.10	17.2	22.4	30	20.1	17	35.7	107
		45	12	0.18	15.0	16.7	11	16.6	11	29.2	95
[16]	LVL, bolts	45	12	0.14	16.5	19.1	16	18.1	10	32.0	94
[17]	LVL, bolts	45	12	0.14	16.5	19.1	16	18.1	10	32.0	94
		45	12	0.12	15.1	20.6	36	19.0	26	33.7	123
[20]	Lumber, bolts	38	12.7	0.10	14.7	17.1	16	16.5	13	32.5	121
		38	12.7	0.30	8.2	8.9	9	10.7	31	20.2	146
[18]	GL28, bolts	60	20	0.19	22.0	30.7	40	20.9	5	33.3	51
		60	20	0.39	15.0	17.4	16	14.5	4	24.2	61
		60	20	0.19	23.0	30.7	33	20.9	9	33.3	45
		60	20	0.38	16.0	17.9	12	14.7	8	24.5	53
		50	12	0.29	17.0	14.5	15	15.3	10	26.1	53
		50	12	0.58	10.0	6.4	36	9.8	2	18.5	85
		50	12	0.25	18.0	16.2	10	16.4	9	27.7	54
		50	12	0.49	11.0	8.4	24	11.2	2	20.4	85
		50	12	0.24	18.0	16.7	7	16.7	7	28.2	56
		50	12	0.48	13.0	8.6	34	11.4	12	20.6	58
[20]	Glulam, bolts	60	12.7	0.11	28.0	36.4	30	26.7	5	40.8	46
		60	12.7	0.18	22.5	28.3	26	22.8	1	35.4	57
		60	12.7	0.29	17.5	20.4	17	18.8	7	30.1	72
		60	19.1	0.11	27.0	40.1	49	25.5	5	40.3	49
		60	19.1	0.32	15.0	20.7	38	16.5	10	27.0	80
		60	19.1	0.10	26.0	41.9	61	26.3	1	41.4	59
		60	19.1	0.30	14.0	21.9	56	17.1	22	27.8	99
		80	19.1	0.10	39.8	68.9	73	36.1	9	51.9	30
		80	19.1	0.29	19.0	37.1	95	24.5	29	38.8	104
[21]	GL28, dowels	76	16	0.10	55.5	60.8	10	34.8	37	49.3	11
		76	16	0.20	41.0	42.5	4	27.8	32	41.2	1
		76	16	0.30	36.0	31.8	12	23.4	35	36.5	1
		75	20	0.10	52.0	62.1	19	33.5	36	49.4	5
		75	20	0.30	37.0	32.5	12	22.1	40	35.7	4
		77	12	0.10	54.0	60.1	11	36.3	33	49.0	9
		77	12	0.30	39.0	31.4	19	25.0	36	37.2	5
		100	20	0.10	90.0	104.4	16	46.0	49	62.9	30
		100	20	0.30	45.0	54.6	21	31.1	31	49.3	10
[16]	LVL, dowels	45	5	0.12	25.8	18.5	28	20.9	19	36.0	39
[19]	GL28, dowels	60	20	0.10	50.0	42.6	15	26.1	48	41.4	17
	& bolts	60	20	0.20	40.5	29.7	27	20.5	49	32.6	19
		60	20	0.30	35.5	22.2	37	16.9	52	27.5	22

7.3.2. Discussion and Future Research Needs. Similarly to the other two connection types, the load ratio affects the fire rating in a major way. Specimens with wood members of 80 mm [15] showed a better fire resistance (65 % increase) when the load ratio was reduced from 0.3 to 0.1. Observation of Fig. 15 shows

Figure 13. Failure modes of heated WSW connections tested by Peng et al. [15] (a–d) and Lau [16] (e, f). (a)–(d) Reprinted with permission of Multi-Science Publishing Co Ltd. Copyright© 2012 *Journal of Structural Fire Engineering*.

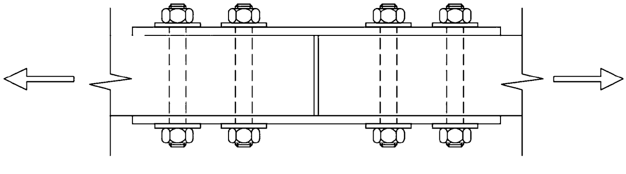

Figure 14. Typical arrangement of SWS connections.

that, for load ratios between 0.1 and 0.2, most specimens failed after 10 min to 15 min of fire exposure. For comparison purposes, only unprotected specimens were plotted. It should be noted that there are fewer data for this type of

Table 7
Fire Resistance Tests of Unprotected SWS Connections Collected from the Literature

Ref.	Wood & fasteners	Central timber thickness (mm)	Steel plate thickness (mm)	No. of fasteners	Diameter of fasteners (mm)	Applied load ratio	Fire resistance (min)
[25]	LVL, bolts	63	6	1	12	0.21	11.4
				4	12	0.17	10.4
[15]	Glulam, bolts	80	9.5	4	12.7	0.10	14.0
						0.30	8.5
		130	9.5	4	19.1	0.10	23.5
						0.19	15.5
[16]	LVL, bolts	63	6	4	12	0.14	8.8
[17]	LVL, bolts	63	6	1	12	0.14	7.4
				4	12	0.12	10.6
[16]	LVL, nails	63	3	33	3.15	0.12	9.0
[38]	GL24h, nails	112	4	54	4	0.15	13.3
						0.30	11.8

Table 8
Fire Resistance Tests of Protected SWS Connections Collected from the Literature

Ref.	Wood & fasteners	Type of protection	Central timber thickness (mm)	Steel plate thickness (mm)	No. of fasteners	Diameter of fasteners (mm)	Applied load ratio	Fire resistance (min)
[16]	LVL, bolts	Intumescent paint	63	6	4	12	0.14	18.7
[15]	Glulam, bolts	Gypsum board	80	9.5	4	12.7	0.30	41.5
[15]	Glulam, bolts	Intumescent coating	130	9.5	4	19.1	0.19	25.5
[16]	LVL, nails	Intumescent paint	63	3	33	3.15	0.12	17.2
[38]	GL24h, nails	Intumescent paint	112	3	54	4	0.15	36.8
							0.30	23.8

connection than for the WWW and WSW joints. However, the reduction of the fire resistance with the load ratio is still evident.

The thickness of timber is also important for SWS connections. As anticipated, better fire resistance is achieved in joints with thicker members. For instance, in one experimental series [15], the specimens with 130 mm wood thickness sustained the imposed temperature effects 13 min longer than those 80 mm thick, when the load ratio was 10 %. Results from the literature, which are plotted in Fig. 16, confirm this observation and show an almost linear increase of fire resistance with timber member thickness. Steel plate thickness is not expected to play an important role on the fire performance of such connections, as steel plates are very thin

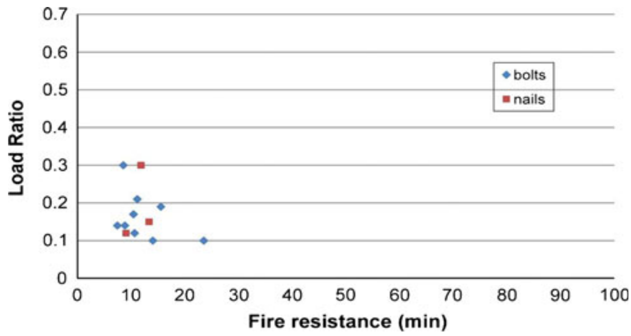

Figure 15. Variation of fire resistance with load ratio for SWS connections tested by various authors.

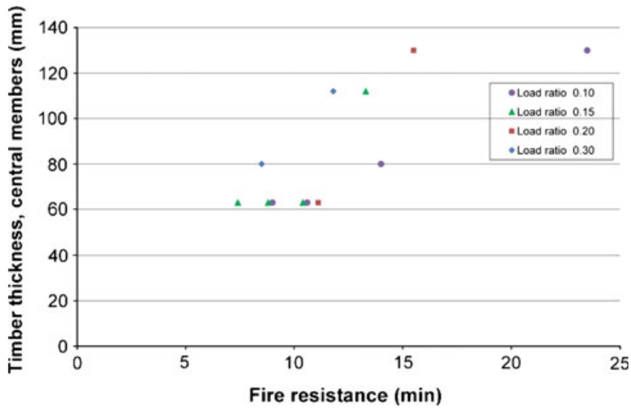

Figure 16. Fire resistance of unprotected SWS connections plotted against timber thickness.

compared with the wood members. Variations in their thickness will affect the thermal field inside the timber section in a minor way.

Even though greater edge distance is expected to improve the thermal response of SWS connections, limited experimental work on this crucial area has been conducted so far. One author [16] reported that the edge distance reduced until splitting of timber along the edge occurred. The effect of the steel plates in transferring heat flux to wood around the edge is not well discussed in the literature and further research on this issue is required.

Contrary to WWW and WSW joints, the gathered data show that the connector type (bolts or nails) does not affect the thermal response of SWS connections in a major way. Specimens with nails and bolts [16] displayed similar fire resistances under approximately the same load ratio. This is anticipated, as transmission of heat to the timber members is governed by the steel plates and not the connector parts (bolt head, washer, etc.).

The beneficial effect of fire protection is evident in the experimental results. In one case, the use of gypsum board increased the fire resistance by about 33 min [14], which corresponds to an increase of approximately 390 %. The presence of intumescent paint was not so effective, as it improved the fire resistance by only about 8 min [15] or, in other experiments [16], by 10 min (90 to 110 % increase).

A limited number of experiments regarding this type of connection is encountered in the literature. The complete absence of data for load ratios exceeding 30 % is noteworthy. Tests of joints at higher load ratios should be included in future work. The fire resistance of unprotected specimens was less than 25 min, showing that this type of connection generally behaves poorly in fire conditions. The effect of fastener diameter on the fire performance of such joints should also be investigated. Testing involving other bolt arrangements, sizes and connector types should be carried out, as the existing data majorly refer to one and four-bolted connections. The effect of other types of protection materials and schemes should also be investigated in future work.

7.3.3. Numerical Analysis Studies. Numerical analysis studies of SWS connections under fire conditions are extremely limited. Austruy [25] tested one bolted SWS joints and created a 3D finite element model to predict the temperature evolution in the various parts of the connection. Afterwards, the author used Johansen's equations to calculate the failure time of the connection. Even though numerical results pertaining to the thermal profiles of the bolts did not show good correlation to the temperature measurements, the calculated fire resistance matched the experimental accurately (a 13.5 % difference was reported [25]).

A 3D finite element heat transfer model for four-bolted SWS connections has also been reported in the literature [41]. Comparison with experimental results from the relevant experimental program [15] showed that the model underestimated temperatures for the initial stages of fire exposure [41], but calculated values matched measurements close to failure time [41]. A structural model, which took into account the obtained temperature profiles, was then created to calculate the load bearing capacity at elevated temperatures according to the possible failure modes. The authors reported that Noren's theory [31] for reduction of the embedding strength with temperature gave the most accurate results [41].

Limited research related to the simulation of SWS joints at elevated temperatures has been conducted so far. The FEM has been used only for thermal analysis of such connections and comparison with test data raises doubts about the validity of the results. Contrary to the other two timber connection types studied, the time–temperature evolution of SWS joints cannot be described with sufficient accuracy. Coupled thermo-mechanical analyses of SWS are absent from the literature and should be included in future research work. Estimated fire resistances should be compared with a wider range of experimental results.

7.3.4. Failure Modes. Several authors reported the failure modes of SWS connections subjected to fire. In one experimental series [15], the following failure types were reported: bolt hole elongation (Fig. 17a), shearing through the timber member (Fig. 17b) and wood splitting (Fig. 17c). Bending of the bolts was justified [15]

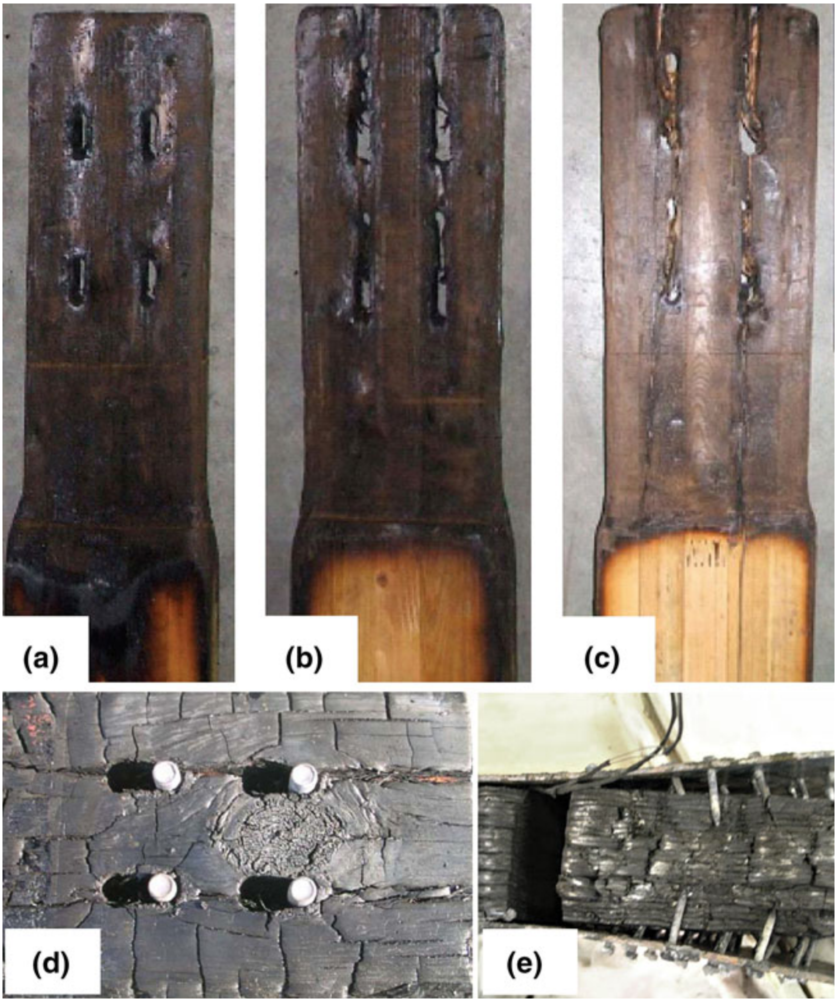

Figure 17. Failure of SWS specimens as reported by Peng et al. [15] (a–c) and Lau [16] (d, e). (a)–(c) Reprinted with permission of Multi-Science Publishing Co Ltd. Copyright© 2012 *Journal of Structural Fire Engineering*.

by the existence of steel plates, which increased the rate of heat transfer and reduced the yield strength of the connectors. In other experiments [16], wood splitting around the bolts (Fig. 17d) or excessive deformation of the nails and loss of embedding strength was reported (Fig. 17e). Chuo [17] stated that, in his experimental program SWS and WWW connections failed in a similar way (elongation of the bolt holes). In the nailed joints tested by others [38], the reported failure mode was either block shear (specimens with fire protection) or excessive deformation of the connectors (unprotected connections).

7.4. Comments for All Connection Types

7.4.1. Failure Mechanism Categorization. The failure pattern of timber connections subjected to fire is majorly affected by the evolution of the thermal profile in their cross-section. Moreover, temperature rise in fasteners and the sensitive region around them is critical in determining the failure pattern, because it affects the embedding to strength to the connected members. It should also be noted that SWS connections are more vulnerable to fire, because the steel plates are directly exposed to fire and accelerate heat transfer to the timber members and the connectors. Failure modes can be categorized as follows:

- *Failure of the connectors* Bending/deformation failure of the connectors is associated with loss of the yield strength because of temperature increase. This type of failure typically occurs for thin connectors (nails, screws, etc.), due to their low bending resistance and high embedding strength, when they experience a steep temperature rise (e.g. detachment of the insulation protecting them or due to existence of external steel plates) and timber members have not undergone considerable charring. For larger connectors, such as bolts, failure typically occurs due to the formation of a charring zone around the fasteners, which remain straight after fire exposure. It should be highlighted that the connector failure modes according to Johansen's theory are valid for timber joints at elevated temperatures. In contrast to ambient conditions, however, failure is associated with embedding strength loss/fastener bending resistance reduction resulting from temperature rise in the connectors and the wood region around them.
- *Timber member failure* These types of failure (wood splitting, crushing, etc.) occur due to thermal degradation of wood. Crushing of wood, which results in hole elongation, is observed because of charring around the connectors. Excessive force transfer from the fastener to this thermally weakened region leads to crushing of the material. This failure mode is directly associated with the diminishment of the embedding strength. It is most commonly encountered in joints with large-diameter bolts because of the increased heat transfer from the connector to wood. In connections (loaded parallel to the grain) with thin timber members, their thermal degradation might cause splitting of wood along the fasteners. Block shear failure of the heated timber member, via reduction of both shear and tensile resistance, is also possible. It is the predominant failure mode (often referred to as edge material tear-out) in connections with small edge distances, because charring (which initiates from the outer portion of the cross-section) further reduces them. Net tension failure of the timber cross-section is also possible in connections with multiple, large-diameter fastener holes, especially when excessive charring develops along the fracture section.
- *Failure of the steel plates* Rupture of the connecting steel plates has not been reported in the literature. Despite their high thermal conductivity, a large portion of their yield strength is maintained at elevated temperatures and possible failure is preceded by that of the timber member. This failure mode can only occur for external thin plates used in conjunction with thick timber elements and connectors with high load bearing capacities.

Table 9
Calculated (per EN1995-1-2 [10]) Versus Experimental Fire Resistances of Timber Connections

Ref.	Wood & fasteners	Type of protection	Side timber thickness (mm)	Applied load ratio	Fire resistance (min)		
					Experimental	Simplified rules	Reduced load method
WWW connections							
[16]	LVL, bolts	–	45	0.15	20.5	15.0	–
[18]	GL28, bolts	–	60	0.30	24.0	29.3	30.0
[19]	GL28, dowels	–	84	0.30	54.0	30.0	40.0
[16]	LVL, screws	–	45	0.15	30.3	15.0	20.0
WSW connections							
[18]	GL28, bolts	–	60	0.19	23.0	29.3	30.0
[21]	GL28, dowels	–	76	0.20	41.0	30.0	30.0
[16]	LVL, dowels	–	45	0.12	25.8	20.0	–
[20]	Glulam, bolts	Gypsum board	60	0.3	51	45.2	43.8
[20]	Glulam, bolts	Douglas fir plywood	80	0.29	34	34.5	33.0

7.4.2. Comparison of EN1995-1-2 Design Methods with Experimental Results. EN1995-1-2 [10] makes a clear distinction between connections with external steel plates and joints with timber side members. For the latter, two design approaches have been proposed (Sect. 5.2).The simplified rules pertaining to unprotected connections are applicable to the majority of the data collected from the literature, as they cover the design of joints with different types of connectors (bolts, dowels, screws, nails, etc.) and most of the tested specimens satisfy the side member thickness/fastener diameter requirements set by the specific code. However, these rules do not refer to the design of connections including different types of fasteners (e.g. dowels and bolts). The reduced load method, which provides an analytical formula for calculating the design fire resistance of unprotected timber connections, has more stringent requirements for member thickness in doweled and bolted connections. Even so, it is applicable to most tests carried out by researchers to date. The design of multiple steel-to-timber connections is not covered by this code. Table 9 compares calculated fire resistances (according to both methods given in EN1995-1-2 [10]) for selected joints tested by researchers with the respective experimental results. The values proposed by EN1995-1-2 [10] are lower (typically from 5 min to 10 min) than those reported by the authors. This is to be expected, because a design code has to yield results that are on the safe side. Despite this, the design methods given in the specific code can be used to conservatively estimate the fire resistance of unprotected timber connections.

Calculation of the fire resistance in protected connections is limited to gypsum board insulation or wood-based panels. Reference to other protection materials, such as intumescent paint, is not made. For protected joints, the only difference between the two calculation methods lies with the determination of the unprotected fire resistance. Experimental results are in good agreement with calculated values (Table 9).

On the contrary, a design method is not proposed for timber connections with steel side members (SWS joints). The provisions of EN1995-1-2 [10] take into account only the external steel plates and refer to EN1993-1-2 [12] for determining their resistance at elevated temperatures. A correlation among the collected experimental data and the regulations of EN1995-1-2 [10] is not relevant.

7.4.3. Analysis Method Selection. According to the failure patterns reported in the literature, determination of the thermal profile in the vicinity of the connectors is critical. Most thermal FE analyses have yielded results that match measurements of the thermal field around that region. Analytical methods lead to satisfactory estimation of the mechanical response of timber connections subjected to fire, but cannot accurately predict temperature evolution in the fasteners and the region around them. In certain cases, authors conducted FE simulations to determine the thermal profile for their analytical (mechanical) models. Moreover, such models are typically restricted to a specific fastener type (e.g. dowels). On the contrary, the applicability of the FEM is not limited to a single connection type and can account for different connector arrangements. Based on these observations, a coupled thermo-mechanical analysis via the FEM is considered to be the most appropriate method for evaluating the fire performance of timber connections. Incorporation of a contact element at the timber–connector interface to account for relative slip and embedding strength variation with temperature is also deemed necessary. It should be noted that, in certain cases, computational effort can be reduced by inputting the thermal profile (which can be determined directly from the basic principles of heat transfer) of certain components. For example, in SWS connections, temperature evolution in the thin external steel plates can be readily calculated by assuming a flat plate exposed to fire from three sides.

8. Preliminary Guidelines for Future Codes

Based on the literature reviewed, future codes should include the following guidelines with regard to the fire design of timber connections:

- Tabulated data providing the fire resistance of typical timber connections should be generated. The data must refer to a wide range of joints and include different geometries, fastener arrangements and types. The fire rating of these connections with various fire protection types should also be listed.
- Analytical expressions for calculating the fire resistance of connections involving parameters such as the edge distance, the fastener diameter and spacing, the thickness of the timber members, insulation (if any), the load ratio, etc. should be developed based on current and future research. To achieve better accuracy, separate equations have to be provided for each connection type.
- A simplified approach to determine their fire rating should also be incorporated in future specifications. This should include the following steps: (a) Evaluation

of the thermal profile of the connection, especially around the timber–connector interface, depending on its geometry and the fire exposure. (b) Selection of the appropriate values for the embedding strength and the mechanical properties, based on the temperature profile. (c) Calculation of the capacity according to the most plausible failure mechanism/theory.
- General rules for selecting the various parameters (e.g. material properties at elevated temperatures) in numerical simulations must be established.

9. Conclusions

A review of the performance of timber connections exposed to fire was presented in this paper. The thermo-mechanical properties of timber at elevated temperatures and its charring rate were studied. The variation of the embedding strength with temperature was also reported. Afterwards, experimental and numerical studies on the fire resistance of WWW, WSW and SWS connections were presented and the effect of several parameters was discussed.

Results show that timber connections are extremely vulnerable to fire. The fire rating of unprotected joints loaded axially is typically less than 30 min. Conventional wisdom suggests reducing the applied load ratio or increasing the thickness of the wood members to enhance the fire resistance. However, this is impractical or not economical in most cases. As of today, the most effective solution is the application of fire protection. More research is needed, nevertheless, to determine the effect of other insulating materials or types on improving the thermal response of timber connections. Loss of embedding strength due to temperature rise is the predominant factor in determining their fire resistance and typically results in elongation of the fastener holes. This failure pattern is commonly encountered in conjunction with others. Furthermore, investigation of different connection types is deemed necessary, as they might be more effective in fire situations. The absence of research related to the thermal behavior of joints with axially loaded connectors is also remarkable. Relevant work should be included in future publications.

References

1. ISO 834-1 (1999) Fire-resistance tests—elements of building construction—part 1: general requirements. ISO, Switzerland
2. CAN/ULC-S101-07 (2007) Standard methods for fire endurance tests of building construction and materials. Laboratories of Canada, Ottawa
3. Mehaffey JR, Cuerrier P, Carisse G (1994) A model for predicting heat transfer through gypsum board/wood stud walls exposed to fire. Fire Mater 18:297–305. doi:10.1002/fam.810180505
4. Janssens ML (1994) Thermo-physical properties for wood pyrolysis models. In: Pacific timber engineering conference, Gold Coast, Australia
5. Knudson RM (1973) Performance of structural wood members exposed to fire. PhD dissertation, University of California, Berkeley, USA

6. Fredlund B (1993) Modelling of heat and mass transfer in timber structures during fire. Fire Saf J 20:39–69
7. Konig J, Walleij L (2000) Timber frame assemblies exposed to standard and parametric fires, part 2: a design model for standard fire exposure. Rapport I 0001001. Swedish Institute for Wood Technology Research, Stockholm
8. Lie TT (1992) Structural fire protection. ASCE manuals and reports of engineering practice, no. 78. American Society of Civil Engineers, New York
9. Takeda H, Mehaffey JR (1998) WALL2D: a model for predicting heat transfer through wood-stud walls exposed to fire. Fire Mater 22:133–140
10. EN 1995-1-2 (2009) Eurocode 5—design of timber structures part 1–2: general rules—structural fire design. CEN, Brussels
11. Peng L, Hadjisophocleous G, Mehaffey J, Mohammad M (2011) Predicting the fire resistance of wood–steel–wood connections. Fire Technol 47:1101–1119. doi:10.1007/s10694-009-0118-4
12. EN 1993-1-2 (2005) Eurocode 3—design of steel structures part 1–2: general rules—structural fire design. CEN, Brussels
13. Erchinger C, Frangi A, Fontana M (2010) Fire design of steel-to-timber dowelled connections. Eng Struct 32:580–589. doi:10.1016/j.engstruct.2009.11.004
14. Buchanan AH (2001) Structural design for fire safety. Wiley, Chichester
15. Peng L, Hadjisophocleous G, Mehaffey J, Mohammad M (2012) Fire performance of timber connections, part 1: fire resistance tests on bolted wood–steel–wood and steel–wood–steel connections. J Struct Fire Eng 3(2):107–131. doi:10.1260/2040-2317.3.2.107
16. Lau PH (2006) Fire resistance of connections in laminated veneer lumber (LVL). MEFE thesis, Department of Civil Engineering, University of Canterbury
17. Chuo TCB (2007) Fire performance of connections in laminated veneer lumber. MEFE thesis, Department of Civil Engineering, University of Canterbury
18. Dhima D (1999) Verification expérimentale de la résistance au feu des assemblages d' éléments en bois. CTICM Ref. INC-99/399-DD/NB, CTICM, Paris, France
19. Laplanche K (2006) Etude du comportement au feu des assemblages de structures bois: Approche expérimentale et modélisation. Thèse doctorale, Université Blaise Pascal – Clermont II
20. Peng L, Hadjisophocleous G, Mehaffey J, Mohammad M (2010) Fire resistance of wood–steel–wood connections. In: Sixth international conference on structures in fire, East Lansing, USA, June 2–4, 2010
21. Ayme N (2003) Assemblages bois-métal en double cisaillement. Rapport Du Comportement Au Feu D' Assemblages Bois, CTICM Ref.SR21-03/121-NA/PB, CTICM, Paris, France
22. Peng L, Hadjisophocleous G, Mehaffey J, Mohammad M (2010) Fire resistance performance of unprotected wood–wood–wood and wood–steel–wood connections: a literature review and new data correlations. Fire Saf J 45:392–399. doi:10.1016/j.firesaf.2010.08.003
23. König J (2005) Structural fire design according to Eurocode 5—design rules and their background. Fire Mater 29:147–163. doi:10.1002/fam.873
24. König J (2005) Structural fire design of timber structures according to Eurocode 5. Fire Saf Sci 8:303–313. doi:10.3801/IAFSS.FSS.8-303
25. Austruy C (2007) Fire resistance of timber connections. http://perso.crans.org/austruy/rapport%20latex/rapportlatex.pdf. Assessed on 7 June 2013
26. EN 1995-1-1 (2006) Eurocode 5: design of timber structures—part 1-1: general—common rules and rules for buildings. CEN, Brussels

27. Dias de Moraes P, Rodrigues JPC, Correia NDF (2012) Behavior of bolted timber joints subjected to high temperature. Eur J Wood Wood Prod 70:225–232. doi:10.1007/s00107-011-0539-x
28. Moss PJ, Nilsen TM, Fragiacomo M, Buchanan AH (2010) Fire resistance of connections in timber structures. In: Sixth international conference on structures in fire, East Lansing, USA, June 2–4, 2010
29. Correia ND, Rodrigues JPC, Moraes PD (2010) Timber bolted joints subjected to high temperatures. In: Sixth international conference on structures in fire, East Lansing, USA, June 2–4, 2010
30. Manriquez MJ (2008) Influence of the temperature on the mechanical resistance of the Pariacá. MSc thesis, Federal University of Santa Catarina, Florianópolis, Brazil
31. Noren J (1996) Load-bearing capacity of nailed joints exposed to fire. Fire Mater 20:133–143
32. Moss PJ, Buchanan AH, Fragiacomo M, Austruy C (2010) Experimental testing and analytical prediction of the behavior of timber bolted connections subjected to fire. Fire Technol 46:129–148. doi:10.1007/s10694-009-0096-6
33. EN 1194:1999 (1999) Timber structures—glued laminated timber—strength classes and determination of characteristic values. CEN, Brussels
34. Standards New Zealand NZS 3603:1993 (1993) Timber structures standard. SNZ, Wellington
35. CAN/CSA-O86-09 (2009) Engineering design in wood. Canadian Standards Association, Rexdale
36. Racher P, Laplanche K, Dhima D, Bouchaïr A (2010) Thermo-mechanical analysis of the fire performance of dowelled timber connection. Eng Struct 32:1148–1157. doi:10.1016/j.engstruct.2009.12.041
37. Cachim PB, Franssen JM (2009) Numerical modeling of timber connections under fire loading using a component model. Fire Saf J 44:840–853. doi:10.1016/j.firesaf.2009.03.013
38. Frangi A, Erchinger C, Fontana M (2010) Experimental fire analysis of steel-to-timber connections using dowels and nails. Fire Mater 34:1–19. doi:10.1002/fam.994
39. Audebert M, Dhima D, Taazount M, Bouchaïr A (2011) Numerical investigations on the thermo-mechanical behavior of steel-to-timber joints exposed to fire. Eng Struct 33:3257–3268. doi:10.1016/j.engstruct.2011.08.021
40. Audebert M, Dhima D, Taazount M, Bouchaïr A (2012) Behavior of dowelled and bolted steel-to-timber connections exposed to fire. Eng Struct 39:116–125. doi:10.1016/j.engstruct.2012.02.010
41. Peng L, Hadjisophocleous G, Mehaffey J, Mohammad M (2012) Fire performance of timber connections, part 2: thermal and structural modelling. J Struct Fire Eng 3(2):133–153. doi:10.1260/2040-2317.3.2.133
42. Erchinger C, Frangi A, Mischler A (2006) Thermal investigations on multiple shear steel-to-timber connections. In: WTCE 2006 9th world conference on timber engineering, Portland, OR, USA, August 6–10, 2006
43. Buchanan AH, King AH (1991) Fire performance of gusset connections in glue-laminated timber. Fire Mater 15:137–143