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FIRE RESISTANCE TESTS ON BEAM-TO-COLUMN SHEAR CONNECTIONS

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Abstract. *This paper presents the results of an extensive experimental campaign on the fire behaviour of beam-to-column timber connections loaded perpendicular-to-the-grain. The experimental campaign addressed the fire behaviour of beam-to-column timber shear connections in a systematic way, testing a wide range of common connection typologies, significantly enlarging their experimental background.*

1 INTRODUCTION

1.1 Background

The fire performance of timber structures is largely influenced by the behaviour of the connections. Current structural fire design methods for timber connections according to EN 1995-1-2:2004 [1] are based on empirical rules [2] derived from a limited number of fire resistance tests on timber members loaded in tension parallel to the grain [3–5]. Given timber's inherent anisotropy, the mechanical properties and failure modes along the direction parallel to the grain, which is usually the longitudinal direction of structural members, are noticeably different from the equivalent properties in the directions perpendicular to the grain. Therefore, connections loaded in the directions parallel and perpendicular to the grain exhibit different behaviours, the latter being prone to brittle splitting failures. Regarding fire resistance, there was a question whether these different failure modes observed at normal temperature would significantly influence the fire behaviour. In addition, and as a consequence of timber's anisotropy, detailing requirements, set by the design at normal temperature, regarding fasteners' spacing, end and edge distances are quite different for connections loaded parallel and perpendicular to the grain. Being metal fasteners responsible for conducting heat into the core of the cross-section and increasing charring, fastener spacing and edge and end distances are bound to play a role on the fire resistance. Given the brittle behaviour observed at normal temperature, the use of perpendicular to the grain reinforcements is a common technique to overcome unwanted brittle failure modes [6]. However, the most common steel reinforcements, self-drilling screws inserted perpendicular to the grain, can contribute to an increased charring and thus to a lower fire resistance [7].

End-grain to side-grain connections, such as beam-to-beam and beam-to-column shear connections (Figure 1), are prevalent situations where members are loaded perpendicular to the grain by connections.

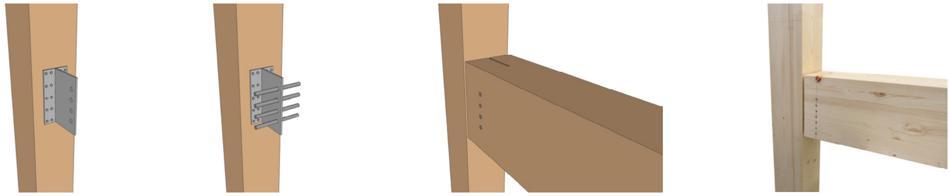

Figure 1. Typical beam-to-column connection loaded in shear (beam end-loaded in tension perpendicular to the grain)

The array of connection typologies used in these situations is quite extensive and comprises exposed and concealed metallic beam hangers (with a nailed header plate and a bearing plate or a dowelled steel-to-timber connection, respectively), dovetail carpenter connections, metallic dovetail connection devices, and diagonally-screwed connections, among others. The problem is that the fire performance of these connections has not been studied; manufacturers and designers addressed this problem by, e.g., adopting larger cross-sections and prescribing smaller tolerances for the gaps between the members.

1.2 Objectives and scope

This paper presents the results of an extensive experimental campaign on the fire behaviour of beam-to-column timber connections loaded perpendicular-to-the-grain. The experimental campaign addressed the fire behaviour of beam-to-column timber shear connections in a systematic way, testing a wide range of common connection typologies, significantly enlarging their experimental background.

2 EXPERIMENTAL CAMPAIGN

2.1 Test programme

The experimental campaign comprised tests at normal temperature and loaded fire resistance tests on beam-to-column connections in shear. Tests at normal temperature were also performed on the *beam-side* of the connection only, which was assumed to be the most exposed to fire and therefore critical for fire resistance. Over 30 full-scale tests at normal temperature were performed, covering 10 different connection typologies, and over 20 loaded fire resistance tests were conducted, including 12 connections typologies. An overview of the experimental campaign is presented in Table 1 and the geometry of the different connection typologies is presented in Figure 2 and Table 2.

2.2 Test specimens – typologies, geometries and materials

Connection typologies A.1 to C.2 are steel-to-timber dowelled connections, on the beam-side (Figure 2.1). In connection A.5, a common commercially available concealed beam-hanger was used, with a geometry that is very similar to the A.1 custom made connection. Typologies A.6 and C.2 are similar to connections A.1 and C.1, respectively, but the beams' cross-section was increased by 40 mm all around. Therefore, connections A.6 and C.2 are rated as R60 (thickness greater than 240 mm) and all the other connections (except D.1) are rated as R30 (thickness greater than 160 mm) [8]. Finally, typology D.1 is a commercially available aluminium dovetail connection, composed by two interlocking parts that are separately screwed to the column and to the end surface of the beam (Figure 2.2).

The various typologies were selected to test different failures modes, such as ductile dowel or embedment failures (B.1), brittle timber splitting or shear failures (A.1-3, A-6, B.2, and C.1-2), and connections reinforced against splitting (A.4). Construction tolerances, such as wider or smaller gaps between the beam and the column, were also considered (A.1-3).

The connections were produced with glued laminated timber made from spruce wood, strength class GL 24h (EN 1194 [9]). The custom made steel connections were manufactured with 5 mm thick steel plates grade S 355 (EN 10025-2 [10]), steel dowels grade 4.6 (EN 1993-1-8 [11]), and threaded nails grade 4.6 [11] with a diameter of 6 mm and a length of 80 mm. In the connections A.4, the full threaded

Table 1. Overview of the experimental campaign.

Connection typology	Type of test		Load	Number of tests
A.1	20 °C	beam-side	Until failure	3
	20 °C	full-connection	Until failure	1
	Fire	full-connection	$0.3 \cdot R_{A.1,mean,20°C}$	2
A.2	Fire	full-connection	$0.3 \cdot R_{A.1,mean,20°C}$	2
A.3	Fire	full-connection	$0.3 \cdot R_{A.1,mean,20°C}$	2
A.4	20 °C	beam-side	Until failure	1
	Fire	full-connection	$0.3 \cdot R_{A.4,mean,20°C}$	1
A.5	20 °C	full-connection	Until failure	1
	Fire	full-connection	$0.3 \cdot R_{A.1,mean,20°C}$	1
A.6	20 °C	beam-side	Until failure	1
	Fire	full-connection	$0.3 \cdot R_{A.1,mean,20°C}$	2
B.1	20 °C	beam-side	Until failure	5
	Fire	full-connection	$0.3 \cdot R_{B.1,mean,20°C}$	2
B.2	20 °C	beam-side	Until failure	5
	Fire	full-connection	$0.3 \cdot R_{B.2,mean,20°C}$	2
C.1	20 °C	beam-side	Until failure	3
	Fire	full-connection	$0.3 \cdot R_{C.1,mean,20°C}$	2
C.2	20 °C	beam-side	Until failure	3
	Fire	full-connection	$0.3 \cdot R_{C.2,mean,20°C}$	2
D.1	20 °C	full-connection	Until failure	1
	Fire	full-connection	$0.3 \cdot R_{D.1,mean,20°C}$	1

self-drilling screws used for reinforcement had a diameter of 9 mm, and were made from carbon steel with a characteristic tensile resistance of 25.4 kN. In the connections A.5, the commercial concealed beam hanger was cold formed from a 3 mm thick steel plate grade S 250 GD (EN 10346 [12]) and fixed to the column with threaded nails grade 4.6 [11] with a diameter of 4 mm and a length of 60 mm. The metal dovetail connections D.1 were made from aluminium grade EN AW-6082 (EN 755-2). One part was fixed to the end-grain of timber member, using 13 screws with a diameter of 8 mm and a length of 100 mm, and the other part was screwed to the side of the column member, using 8 similar screws.

3 TESTS AT NORMAL TEMPERATURE

3.1 Test set-up and procedure

A special test set-up was developed to load the beam-to-column connections in the same way during the tests at normal temperature and during the fire resistance tests. This set-up comprises a horizontal steel frame, inside which the connections are placed and loaded, and to which all load and displacement gauges and load actuators are attached. For the tests at normal temperature the steel frame was placed slightly above the floor (Figure 3a); and during the fire tests the frame was placed on top of a horizontal furnace, centring the connection with the furnace opening (Figure 3b). The load was applied in the beam through a loading plate connected by two rods to the hydraulic cylinders. As no equipment could be inside the furnace during the fire tests, the loading apparatus was placed outside the furnace opening, but as close as possible to the connection. In both the tests at normal temperature and the fire tests, the load was applied at the same distance from the connection, to assure the same distribution of internal forces.

At normal temperature, two types of tests were performed: tests on the beam-side of the connection (replacing the timber column by a steel profile, as presented in Figure 3a); and tests of the whole beam-to-column connection (with the same arrangement used in the fire tests, presented in Figure 3b). The

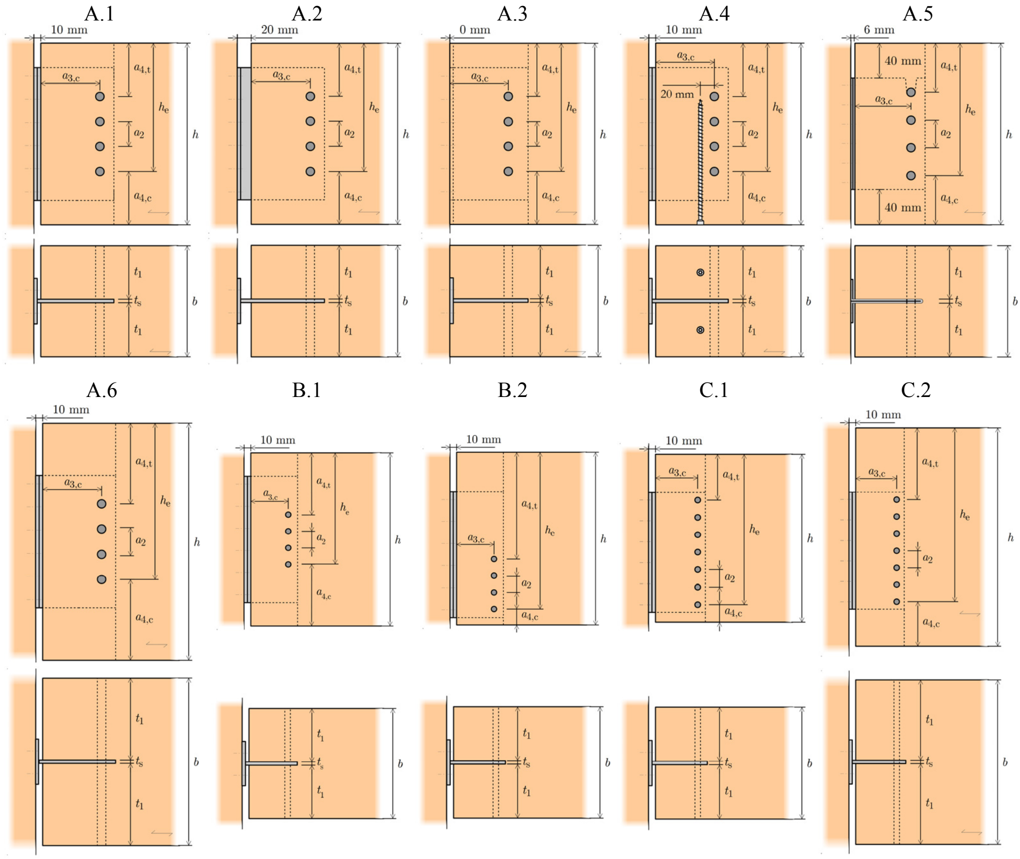

Figure 2.1. Geometries of the tested dowelled connections.

D.1

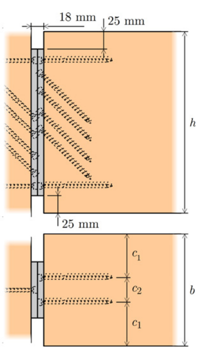

Figure 2.2. Aluminium dovetail connection

Table 2. Geometries of the tested dowelled connections.

Connection typology	b×h [mm^2]	d [mm]	n_d	h_e/h [ ]	a_2 [mm]	$a_{3,c}$ [mm]	$a_{4,t}$ [mm]	$a_{4,c}$ [mm]	s [mm]
A.1, A.4	160×260	12	4	0.71	36	84	76	76	10
A.2	160×260	12	4	0.71	36	84	76	76	20
A.3	160×260	12	4	0.71	36	84	76	76	0
A.5	160×260	12	4	0.73	40	80	70	70	6
A.6	240×340	12	4	0.66	36	84	116	116	10
B.1	160×260	8	4	0.64	24	56	94	94	10
B.2	160×260	8	4	0.91	24	56	164	24	10
C.1	160×260	8	7	0.91	24	56	92	24	10
C.2	240×340	8	7	0.81	24	56	132	64	10
D.1	160×260	-	-	-	-	-	-	-	18

b and h are the width and the height of the cross section; d is the dowel diameter and n_d is the number of dowels; h_e is the distance from the most distant fastener to the loaded edge; a_2 , $a_{3,c}$, $a_{4,t}$, and $a_{4,c}$ are the dowel spacing, unloaded end distance, loaded edge distance, and unloaded edge distance, respectively; and s is the gap between the beam and the column.

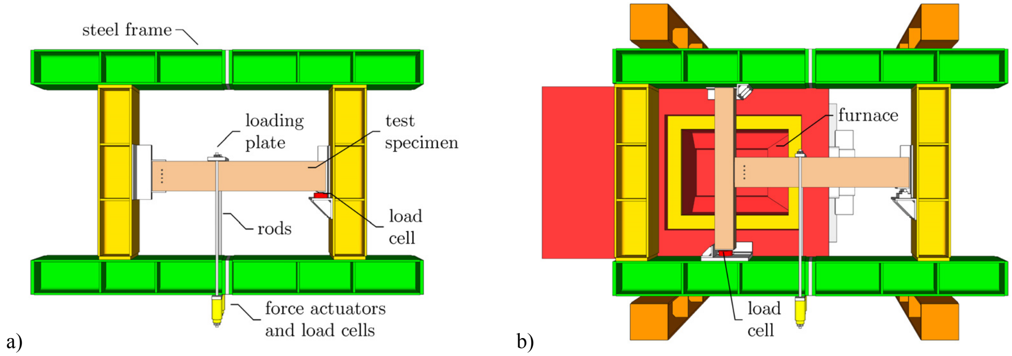

Figure 3. Top view of the test set-up: a) tests at normal temperature; b) fire resistance tests.

beam-side only tests were performed because most connections were estimated to fail on the beam-side (Table 3 and Figure 4a) and because it was assumed that this part of the connection would be most exposed to fire and therefore critical to fire resistance. The load applied by the loading plate, through the rods and hydraulic cylinders, was monitored using rod-end compression load cells. In the beam-side tests, another load cell was placed in the beam support opposite to the tested connection and the shear load in the connection was calculated by simple equilibrium ($F_{connection} = F_{rod} - F_{support}$). In the tests with the full beam-to-column connection, a load cell was positioned beneath the column and, therefore, the shear force going through the connection was directly measured. In addition to the load cells, displacement transducers were placed in the loading plate and in the connection area to assess the relative displacement between the timber member and the steel plate. The tests at normal temperature were performed at the laboratories of ETH Zurich, Switzerland. They were conducted in accordance with EN 26891:1991 [13], which prescribes a loading procedure based on the estimated load-carrying capacity F_{est} of the connection: the load is increased up to $0.4 \cdot F_{est}$, then reduced to $0.1 \cdot F_{est}$, and thereafter increased until failure.

3.2 Results

The estimated and experimental load-carrying capacities of the tested connections are presented in Table 3 and Figure 4. Most connections were expected to exhibit splitting failures (A.1-3,5-6, B.1, and C.1-2); only connections B.2 and, possibly, A.4, were expected to display ductile dowel failures. Regarding the estimated splitting capacities of the beam-side of the connections, it has to be mentioned that EN 1995-1-1 does not have specific rules for end-loaded members loaded perpendicularly to the grain, but only to mid-span loaded members. However, both the former German code for timber structures DIN 1052:2004 [6] and A. Leijten [14] (who developed the calculation model for mid-span loaded members in EN 1995-1-1) state that for connections at the end of a cantilever the splitting strength is half of the strength of mid-span loaded members.

The results show that the beam-side load-carrying capacity is substantially underestimated by current design methods, even taking into account that $R_{k,estimated}$ are characteristic 5% percentile values and $R_{mean,experimental}$ are mean values. Also worth noticing is that the column-side load-carrying capacity seems to be significantly underestimated, as no failure in the column-side was observed in the full connection tests performed on typology A.5.

Table 3. Estimated and experimental load-carrying capacities of the connections tested at normal temperature.

Connection typology	Type of test	Number of tests	$R_{k,estimated}$ [kN]		$R_{mean,experimental,20°C}$ [kN]		
			Beam-side	Column-side			
A.1	Beam-side	3	27.0	-	51.5	(7%)	-
	Full connection	1	27.0	30.7	51.2	-	(beam-side failure)
A.2	-	-	27.0	28.3	-	-	-
A.3	-	-	27.0	33.6	-	-	-
A.4	Beam-side	3	>27.0	30.7	64.0	(3%)	-
A.5	Full connection	1	28.6	14.3	39.3	-	(beam-side failure)
A.6	Beam-side	3	42.0	30.7	58.5	(11%)	-
B.1	Beam-side	5	23.1	39.9	31.8	(10%)	-
B.2	Beam-side	5	37.6	39.9	46.7	(7%)	-
C.1	Beam-side	3	54.5	39.9	68.6	(8%)	-
C.2	Beam-side	3	62.7	39.9	89.3	(4%)	-
D.1	Full connection	1	52.0 (acc. to the manufacturer)		60.3	-	(column-side failure)

Coefficient of variation between parentheses.

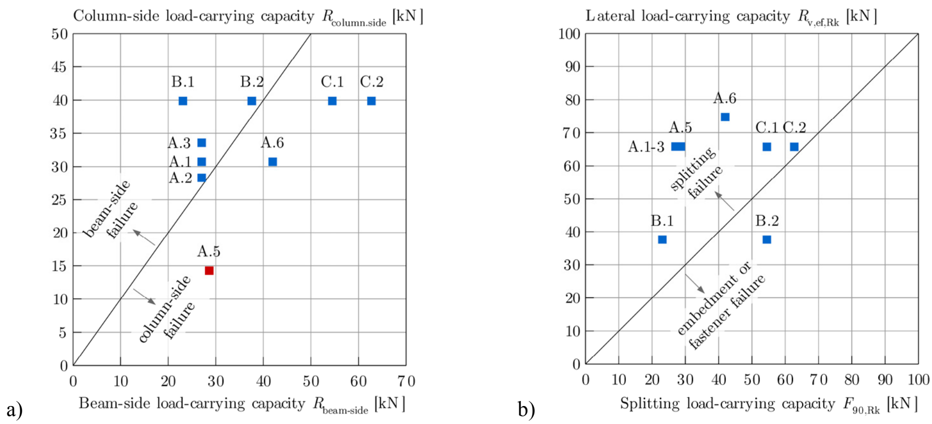

Figure 4. Estimated load-carrying capacities of the tested dowelled connections: a) beam-side and column-side capacities; b) beam-side failure modes.

4 FIRE RESISTANCE TESTS

The fire resistance tests were conducted in the small horizontal furnace of the Laboratory for Fire Testing at the Swiss Federal Laboratories for Materials Science and Technology (Empa), in Dübendorf, Switzerland.

4.1 Test set-up and procedure

As previously described (Figure 3b), the test set-up used in the fire tests is very similar to one used in the tests at normal temperature. The same horizontal steel frame was positioned over the furnace, in such a way that the connection area was centred above the furnace's opening. In its plane, the connection was supported on a load-cell, placed at the bottom of the column-member (to measure directly the shear load in the connection, transferred as a compression force in the lower half of the column), and on a roller, located at the opposite end of the beam-member. The load was applied through a loading plate positioned on the top-side of the beam (which during the test it's on its side), connected by two steel rods to the

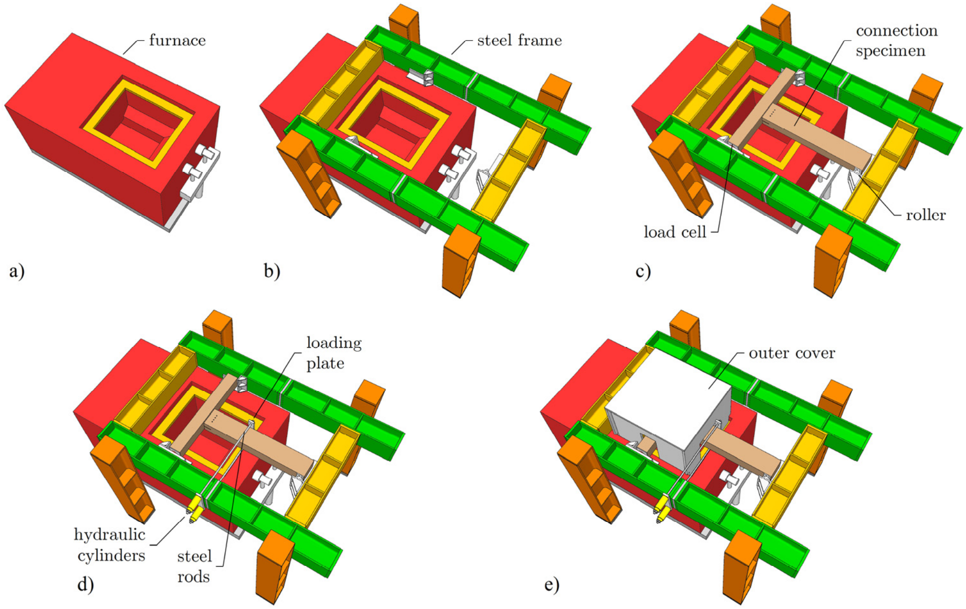

Figure 5. Schematic overview of the fire resistance tests set-up: a) horizontal furnace; b) steel frame over the furnace; c) connection specimen inside the frame; d) loading apparatus; e) outer cover.

hydraulic cylinders. Since no equipment could be exposed to fire, only the displacements of the loading plates were measured, not the relative displacements between the members in the connection. The connection specimen was then partially enclosed by an insulated outer cover that allows the timber members to deform. The test set-up is presented in Figure 5.

After the whole test set-up was ready, the connections were loaded up to the target load level (Table 1), which was approximately 30% of the load-carrying capacity at normal temperature, and that load was maintained throughout the fire test. Once the displacements stabilized, the burners were started and the connection area exposed to the standard ISO 834 [15] time-temperature curve. After failure, reached when the displacements in the beam increased ever rapidly and it no longer could sustain the applied load, the specimens were promptly removed from the furnace and cooled with water (Figure 6).

Since the full connections tested at normal temperature failed on the beam-side (Table 3) and the column-side of the connections (header plate nailed to the column-member) was the same for every connection, except A.5 and D.1 (which were commercially available parts), to assure a beam side failure in the fire tests, the column-member was partially protected with insulation. This was mainly to focus the tests on the beam-side of the connections and to avoid that burning from the sides and the back of the column could promote a premature failure.

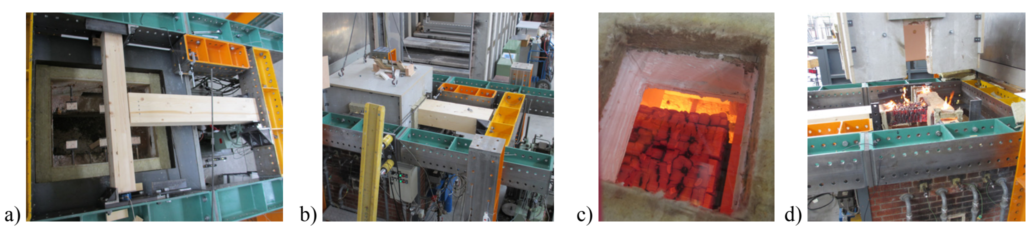

Figure 6. Fire test: a) connection specimen in the steel frame over the furnace; b) outer cover enclosing a connection during a test; c) view of a connection inside the furnace; d) removal of the outer cover after a test.

4.2 Results and discussion

The main results of the fire resistance tests are presented in Table 4. Two replicas of most connection typologies were tested and the corresponding results were consistent. All the dowelled connections (A, B, and C) exhibited fire resistances higher than 30 minutes and connections A.6 and C.2 reached 60 minutes, which is accordance with Lignum's documentation on the fire resistance of timber connections [8].

Regarding the influence of the gap between the beam and the column, it can be seen that an increase from 10 mm (A.1) to 20 mm (A.2) lead to an average reduction from 44 to 33-34 minutes of fire resistance. On the other hand, a reduction of the gap from 10 mm (A.1) to 0 mm (A.3) led to an increase of the fire resistance of only 3-4 minutes. The larger 20 mm gap between the beam and the column induced failures in the column-side of the connection, unlike the 10 mm and 0 mm gaps for which failure occurred in the beam-side (embedment of the dowels followed by splitting of the beam). As charring from the sides was mostly prevented in the column-members, the heat damage to the column side came mostly from above and below the header plate, directly affecting the outermost nails in withdrawal and compression zone of the header plate, allowing it to rotate.

The connection reinforced with self-drilling screws (A.4) exhibited a fire resistance 5 minutes lower than the corresponding unreinforced connection (A.1). The reinforcement with self-drilling screws is an effective and widespread way to deal with the brittle splitting failures of the unreinforced connections loaded perpendicularly to the grain. However, as observed in a previous experimental campaign performed by the authors on the fire resistance of tension connections reinforced with self-drilling screws [7], the reinforcement screws also conduct heat into the cross-section (Figure 8), which in some cases compromises the fire resistance.

Table 4. Results of the fire resistance tests.

Connection		Load E_{fire}	Fire resistance t_{fi}
Typology	Specimen	[kN]	[min.]
A.1	A.1.F-1 [a]	15.2	40
	A.1.F-2	15.4	44
A.2	A.2.F-1	15.4	33
	A.2.F-2	15.4	34
A.3	A.3.F-1	15.5	48
	A.3.F-2	15.5	47
A.4	A.4.F-1	19.4	39
A.5	A.5.F-1	15.4	39
A.6	A.6.F-1	15.5	76
	A.6.F-2	15.5	83
B.1	B.1.F-1	9.4	49
	B.1.F-2	9.5	43
B.2	B.2.F-1	14.0	43
	B.2.F-2	13.9	45
C.1	C.1.F-1	20.5	44
	C.1.F-2	20.5	42
C.2	C.2.F-1	20.6	78
	C.2.F-2	20.6	73
D.1	D.1.F-1	17.8	36

A.1 (10 mm gap) A.2 (20 mm gap) A.3 (0 mm gap)

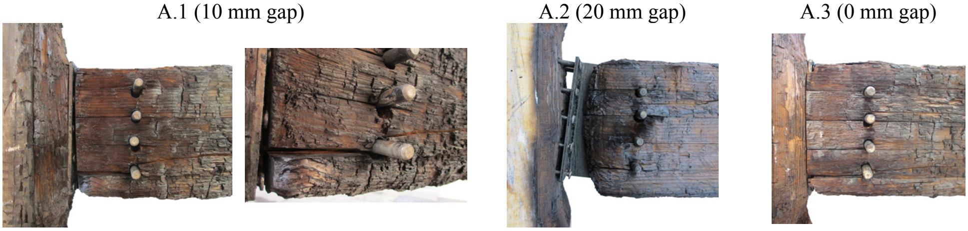

Figure 7. Influence of the gap between the beam and the column: connections A.1, A.2, and A.3 after the fire tests.

The common commercially available concealed beam-hanger (A.5) had gap of only 6 mm between the beam and the column (Figure 2.1 and Table 2), but a much lower estimated load-carrying capacity of the column-side of the connection (Table 3 and Figure 4a), although it failed on the beam-side at normal temperature. This commercial connection exhibit a fire resistance 5 minutes lower than the custom A.1 connection and failed in the column-side, with a failure mode similar to that of the connections A.2. However, the load in connection A.5 during the fire test was about 40% of the load-carrying capacity of the beam-side at normal temperature, instead of the 30% in connections A.1-3.

A.4 (reinforced) A.5 (commercial beam-hanger)

Figure 8. Connections A.4 and A.5 after the fire tests.

The influence of the failure mode can be analysed in the connections B.1 and B.2. These connections had smaller sized dowels (diameter of 8 mm) and showed brittle splitting (B.1) and ductile embedment/dowel failures (B.2) at normal temperature. In the fire tests, however, both typologies exhibited similar extensive embedment failures, followed by splitting. In fire, the smaller minimum dowel spacing perpendicular to the grain prescribed by EN 1995-1-1 (only $3{\cdot}d$, compared to $5{\cdot}d$ parallel to the grain) leads to a premature failure when all the wood between the dowels is charred (Figure 9). Also the smaller minimum unloaded edge distances (connection B.2) perpendicular to the grain (only $3{\cdot}d$, compared to $7{\cdot}d$ parallel to the grain) result in the complete charring of the wood surrounding the outermost dowels. Regardless of the failure mode at normal temperature, both connection typologies exhibited approximately the same fire resistance.

B.1 B.2

Figure 9. Connections B.1 and B.2 after the fire tests.

Connections C.1 exhibited shear/splitting failures at normal temperature and the highest load-carrying capacities of the R30 connections (Table 3). In the fire tests, they reached approximately the same fire resistance as connections A.1. In fire, the dowels remained mostly straight and the wood surrounding the dowels closer to the unloaded edge charred completely. After failure, splitting cracks could be observed in the beam side.

Figure 10. Connections C.1 after the fire tests.

Connections A.6 and C.2 reached more than the estimated minimum 60 minutes of fire resistance [8]. Regarding connections A.6 (with 12 mm dowels), the long fire exposure charred the wood below and above the header steel plate nailed to the column, affecting the tension (nail withdrawal) and compression zones and allowing the steel plates to rotate (Figure 11, left). The dowels' end distance in connections C.1 (8 mm dowels) was smaller than in connections A.6 and, consequently, so was the moment in the header plate. Therefore, the rotation of the header plate was negligible. On the other hand, the charred depth in the zone between the dowels and the end of the beam was significantly higher than elsewhere in the beam, due to additional heat coming from the burning column-member and transferred by the dowels into the cross-section. After the wood between the dowels charred, the load was mostly transferred through the last dowel and it ultimately bent (Figure 11, right).

Figure 11. Connections A.6 and C.2 after the fire tests.

Finally, the commercial aluminium dovetail connection D.1 also reached more than 30 minutes of fire resistance, failing in the connector itself after 36 minutes. This connection had a gap of 18 mm between the beam and the column (thickness of the connector), which is larger than the 10 mm of most of the other connections.

Figure 12. Connection D.1 after the fire test.

5 CONCLUSIONS

This paper presents the results of an extensive experimental campaign on the fire behaviour of beam-to-column timber connections loaded perpendicular-to-the-grain. A special test set-up was developed and it was successfully used in both the tests at normal temperature and the fire tests. The results of the fire resistance tests show that the tested typologies of steel-to-timber dowelled connections reached more than 30 and even 60 minutes of fire resistance. However, aspects such as a wider gap between the beam and the column, reduced dowel spacing, and the presence of reinforcement with self-drilling screws all have a negative influence on the fire resistance. Even though the beam-side of these connections is apparently more exposed to fire, failures in column-side also occurred and, therefore, the fire resistance of these connections has to take into account these two parts.

REFERENCES

[1] EN 1995-1-2:2004 Eurocode 5: Design of timber structures – Part 1-1: General – Structural fire design, European Committee for Standardization (CEN), Brussels, 2004.

[2] J. König, M. Fontana, The performance of timber connections in fire – Test results and rules of Eurocode 5, in: Proceedings of International RILEM Symposium "Joints in Timber Structures," RILEM Publications, Stuttgart, Germany, 2001: pp. 639–648.

[3] J. Norén, Load-bearing capacity of nailed joints exposed to fire, Fire and Materials. 20 (1996) 133–143.

[4] D. Dhima, Vérification expérimentale de la résistance au feu des assemblages d'éléments en bois, Centre Technique Industriel de la Construction Métallique (CTICM), Saint-Rémy-lès-Chevreuse, France, 1999.

[5] J. Kruppa, T. Lamadon, P. Racher, Fire resistance tests of timber connections, CTICM - Departement Incendie et Essais, Saint-Remy-Les-Chevreuse, France, 2000.

[6] DIN 1052:2004-08. Entwurf, Berechnung und Bemessung von Holzbauwerken. Allgemeine Bemessungsregeln und Bemessungsregeln für den Hochbau, Deutsches Institut für Normung (DIN), 2004.

[7] P. Palma, A. Frangi, E. Hugi, P. Cachim, H. Cruz, Fire resistance tests on steel-to-timber dowelled connections reinforced with self-drilling screws, in: Proceedings of the 2nd Ibero-Latin-American Congress on Fire Safety – 2nd CILASCI, Coimbra, Portugal, 2013.

[8] A. Frangi, I. Brühwiler, J. Studhalter, R. Wiederkehr, Lignum-Dokumentation Brandschutz. 3.1 Feuerwiderstandsbemessung – Bauteile und Verbindungen, Lignum Holzwirtschaft Schweiz, Zurich, 2011.

[9] EN 1194:1999. Timber structures – Glued laminated timber – Strength classes and determination of characteristic values, European Committee for Standardization (CEN), Brussels, 1999.

[10] EN 10025-2:2004. Hot rolled products of structural steels - Technical delivery conditions for non-alloy structural steels, European Committee for Standardization (CEN), Brussels, 2004.

[11] EN 1993-1-8:2005. Eurocode 3: Design of steel structures – Part 1-8: Design of joints, European Committee for Standardization (CEN), Brussels, 2005.

[12] EN 10346:2009. Continuously hot-dip coated steel flat products. Technical delivery conditions, European Committee for Standardization (CEN), Brussels, 2009.

[13] EN 26891:1991. Joints made with mechanical fasteners – General principles for the determination of strength and deformation characteristics, European Committee for Standardization (CEN), Brussels, 1991.

[14] A. Leijten, Splitting strength of beams loaded by connections - Model comparison, in: Proceedings of the CIB-W18 Meeting 35, CIB, Kyoto, Japan, 2002.

[15] ISO 834-1:1999. Fire-resistance tests. Elements of building construction. Part 1: General requirements, International Organization for Standardization (ISO), 1999.