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A review on moment-resisting beam-column connections for timber frames

Revisión de conexiones viga-columna resistentes a momentos para estructuras de madera

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ABSTRACT

The discontinuity of wood fibres at the intersections of beam-column connections poses challenges to the transmission of moments between these elements. Some engineering solutions employ different mechanical principles to solve this problem. For example, the polygonal arrangement of connectors is a widely adopted solution; the connectors are inserted perpendicularly to the bending plane and act in shear. Another example is the lever mechanism, which balances the moment through a torque created by the tension of a metal connector and compression against the wood. An innovative proposal involves reinforced concrete nodes, where the continuity of the bending moment is ensured by glued-in rods anchored in the timber beam and column with structural epoxy resin. Another solution is the pre-stressed connection, where the balancing moment is achieved by adjusting the compressive stresses introduced by the prestress. This work aims to present, classify, and compare these solutions in terms of structural behaviour.

Keywords: ductility; polygonal connection; lever connection; pre-stressed connection; moment strength; stiffness.

RESUMEN

La discontinuidad de las fibras de madera en las intersecciones de las uniones viga-columna plantea retos a la transmisión de momentos entre estos elementos. Algunas soluciones de ingeniería emplean diferentes principios mecánicos para resolver este problema. Por ejemplo, la disposición poligonal de los conectores es una solución ampliamente adoptada; los conectores se insertan perpendicularmente al plano de flexión y actúan a cortante. Otro ejemplo es el mecanismo de palanca, que equilibra el momento mediante un par creado por la tensión de un conector metálico y la compresión contra la madera. Una propuesta innovadora son los nudos de hormigón armado, en los que la continuidad del momento flector se garantiza mediante varillas encoladas ancladas en la viga y el pilar de madera con resina epoxi estructural. Otra solución es la conexión pretensada, en la que el momento de equilibrio se consigue ajustando los esfuerzos de compresión introducidos por el pretensado. El objetivo de este trabajo es presentar, clasificar y comparar estas soluciones en términos de comportamiento estructural.

Palabras clave: *ductilidad; conexión poligonal; conexión de palanca; conexión pretensada; resistencia a momentos; rigidez.*

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1. INTRODUCTION

Contrary to steel and concrete, where it is possible to create rigid frame joints through welding (in the case of steel) or the monolithic nature of the joint (in the case of concrete), the discontinuity of wood fibres at the intersection of beams and columns makes it challenging to achieve continuous moment connections.

The most widely used solution in practice, supported by a simplified but proven safe analysis model, is the bolted connection for shear, arranged in a circumferential or polygonal pattern at the intersection of these elements (Figure 1). In its simplest form, it is a wood-to-wood double shear connection, consisting of a double column with a central beam or a double beam with a central column, with the first variant being the most common. The forces transmitted by the connectors against the surrounding wood generate the continuity moment [1].

Those connectors are typically dowels or bolted, but the use of wooden dowels or expanded metal tubes also offers prospects for utilization. This connection has low rotational stiffness and is always of partial strength (i.e., with a bending strength lower than that of the connected pieces) because the diameter of the polygon has to be accommodated within the limited area of the joint, in compliance with regulatory spacing between connectors and from these to the sides and corners of the wood pieces.

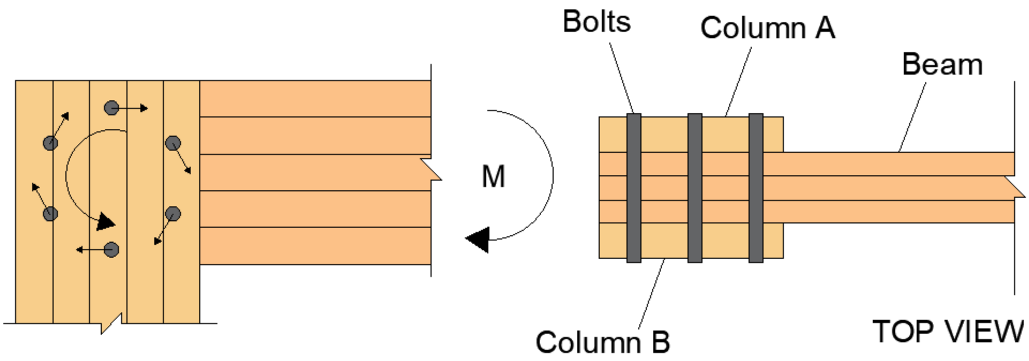

Figure 1. Polygonal joint

Another limitation is that it does not allow for a flat connection, except by using internal or external steel plates between the timber elements (Figure 2) [2], [3], [4], [5], [6], [7], [8]. However, these arrangements require duplicating the connection, substantially increasing the material and labour costs. Despite being a relatively old connection system, research continues to be published dedicated to studying and improving its structural performance, particularly regarding seismic behaviour and enhanced strength and stiffness capabilities, see [9]; [10]; [11]; [12]; [13]; [14]; [15].

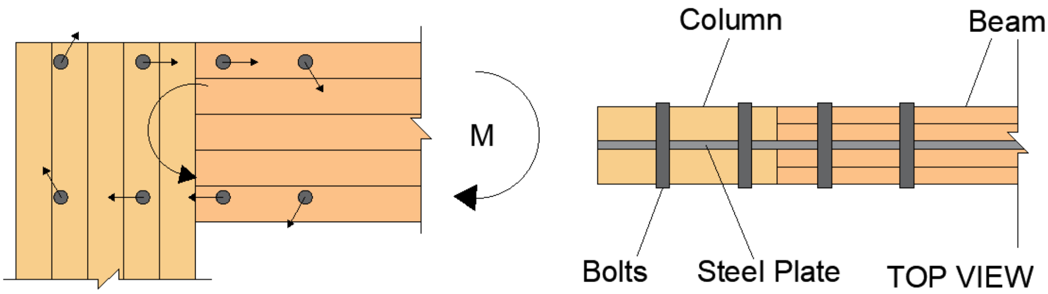

Figure 2. Plane polygonal joint (Lam et al., 2008)

Another alternative explored by several authors is the use of the lever principle for load transmission. Unlike the polygonal joint, the balance of moments in such connections is achieved through connectors in the flexural plane, working in tension rather than in shear (Figure 3). This lever effect produces a balancing torque, with an arm dimension close to the height of the cross-section. The tensile force is supported by a metallic element, glued, screwed, or anchored to one metal plate. The compressive force is directly supported by the compression of the wood, parallel to the grain in one structural element and perpendicular to the grain in the other. The literature includes various approaches that explore this same physical principle in different ways [16], [17], [18], [19], [20], [21], [22], [23]. For instance, implementations using glued bars [24], screws [25] and even specific metallic components [17], [26] were investigated. Other variants involve extending the column or the beam, and calculation procedures were proposed for both cases [27].

The main issue with lever-based solutions is the introduction of high compressive forces perpendicular to the wood grain of the column (in the extended column variant, Figure 3, left) or the beam (in the extended beam variant, Figure 3, right). Some authors propose solutions to overcome or mitigate this problem, such as the diagonal wood split proposed by [28] or the use of a steel corner for the case studied [29].

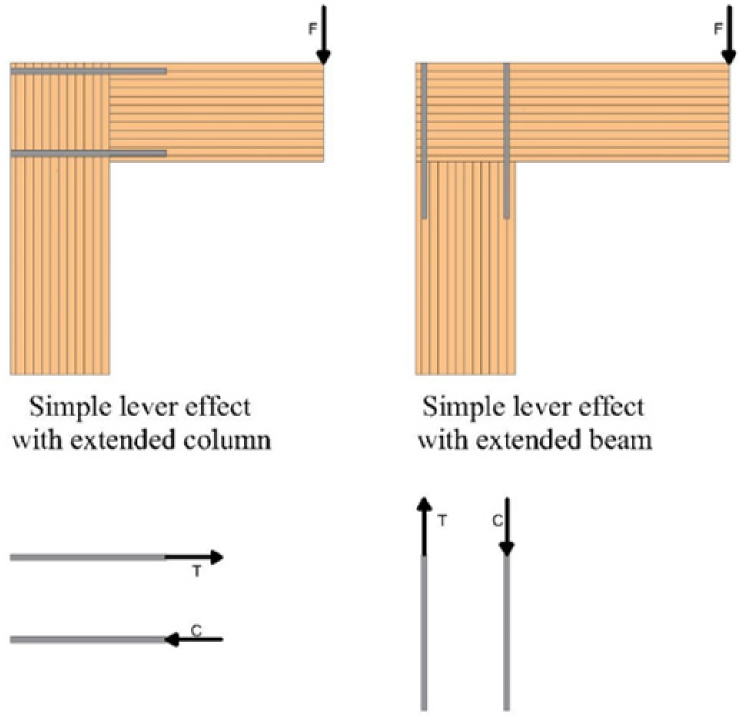

Figure 3. Lever-based connections

In [30], the reinforced concrete connection was first proposed, based on a mechanical model essentially similar to that governing the behaviour of connections in concrete structures (Figure 4). In particular, the role of the steel-concrete bond, which in these ensures the transfer of forces between the two collaborating materials, is fulfilled in wooden frames by bonding the rebars to the surrounding wood beam or column by using a structural adhesive (an epoxy resin, typically). The good results obtained and reported in that work regarding the performance of this type of connection led to the launch of a more ambitious research program, now nearing its end. Using the strongest polygonal connection that could be accommodated within the dimensions of the wooden pieces used in the test specimens as a basis for comparison, gains of about 300% in flexural strength and an order of magnitude (10×) in stiffness were achieved [31], by comparison to the polygonal solution.

A final line of research in moment-resisting connections for timber frames is that of prestressed connections (Figure 5). In these connections, the balancing moment is obtained by reducing/increasing (depending on the considered side) the compressive stresses introduced by the prestress, initially uniform. These connections exhibit high stiffness and strength and a good response to seismic events, but face challenges related to uncertainties in quantifying the loss of tension in the prestressed tendons [32], [33], [34], [35], [36], [37] and the high compressive forces perpendicular to the wood grain under the tendon anchorage.

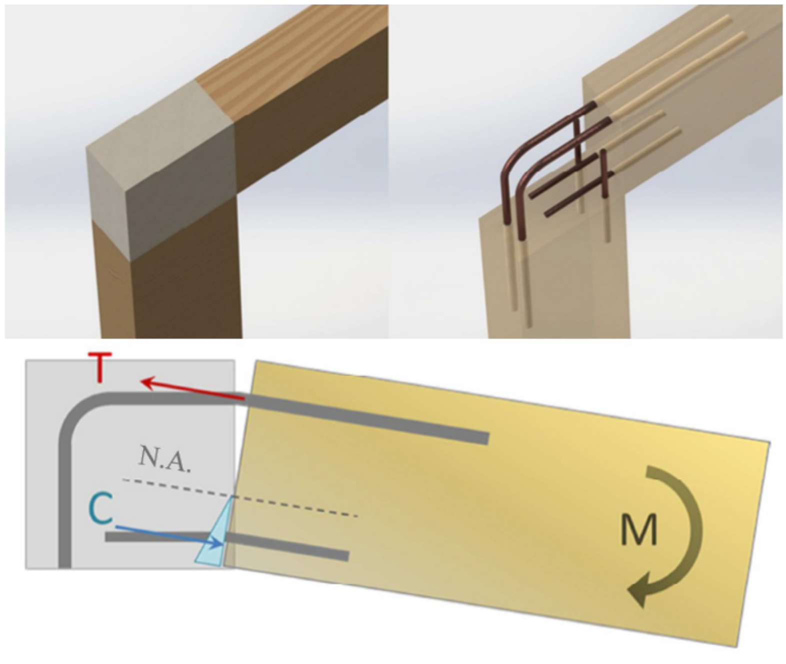

Figure 4. Reinforced concrete connection

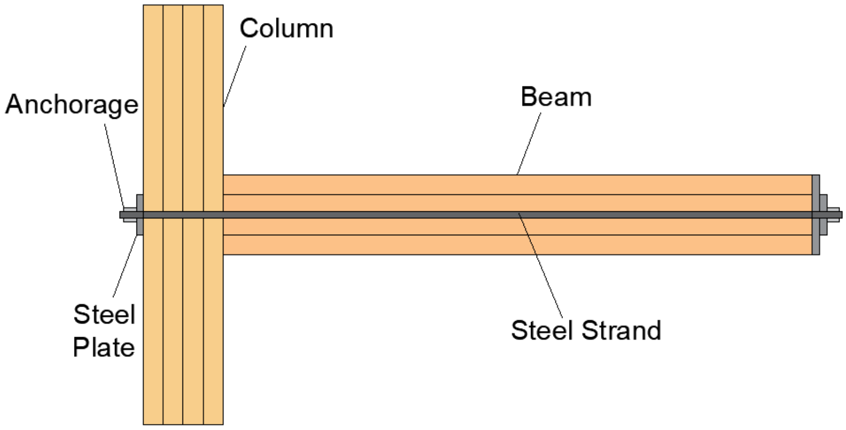

Figure 5. Prestressed connections

2. RELEVANT MOMENT-RESISTANT JOINT STRUCTURAL PROPERTIES

The moment-resistant connections have the main function of transferring bending stresses from the beam to the column. Among other advantages, when compared to a pinned connection, are the capacity to reduce the maximum bending moment in the span and the contribution to an increased overall stiffness of the structure.

For this type of connection, three main aspects must be observed, namely its bending strength, its stiffness and its ductility. These aspects are discussed and compared for the solutions presented before thorough Tables 2-6.

2.1. Bending strength

The resisting moment is the maximum moment that a connection can resist. The moment-resistant connections may be designed as either Full Strength (in which case its bending strength is not smaller than that of the cross-sections of the connected elements) or Partial Strength (when the connection is weaker than the cross-sections of the joined structure (beam and column), but strong enough to withstand the design loads).

2.2. Stiffness

The connections can have their behavior characterized by means of M-θ curves, ranging from "Rigid", when the relative angle between the beam and the column at the point of union does not change, and "Pinned", in which any change in the relative angle can occur without any bending moment. These extreme cases correspond to the axes of the M-θ diagram, and seldom correspond to the behavior of an actual connection. Usually, as it is somewhere between the Rigid and Pinned idealizations, the connection is called semi-rigid and its stiffness depends on the relative rotation between the jointed members. The rotational connection stiffness could be considered as the instantaneous stiffness (R_{kt}), that is obtained as the slope of the M-θ curve at the point considered (Equation (1)).

(1) $$R_{kt} = \frac{dM}{d\theta_r}$$

where:
R_{kt} is the instantaneous (tangent) joint stiffness
M is the moment in the connection
θ_r is the relative rotation between the notional end surfaces of the connection

2.3. Ductility

In most timber structures, the connections are the only regions where significant plastic deformation may develop. The energy dissipative capability of the structure is therefore highly dependent on the ductility of the connections. For this reason, these should be designed in order to ensure the maximum possible plastic deformation capability, namely when ductility is an issue, as in seismic-prone regions. The impact of the ductility in the design is well illustrated by the behavior factor (q) in Eurocode 8 [38], which ranges from 1 (elastic, non-dissipative) to 6 (plastic, highly dissipative) structures.

The ductility of a single joint can be determined from load-displacement curves. A calculation for the ductility ratio (D) is given in Equation (2) [39].

(2) $$D = \frac{\nu_u}{\upsilon_y}$$

where:
D is the ductility ratio
ν_u is the displacement at failure
υ_y is the displacement in the yield onset

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For some reference values comparisons, Fairweather [40] classifies some timber connections depending on the type of connection and its behaviour (Table 1).

Table 1. Ductility exhibited by different types of connections

Ductility (D)	Classification	Joint Type
1	Elastic	Rigid glued joints Strong connections Toothplate nail joints Large diameter bolts
2.5 +	Limited Ductility	Steel plates buckling under compression only Nailed steel side plates
4	Ductile	Nailed joints Small diameter bolts Nail-on plates Shearwalls with nailed sheathing

Source. [40]

3. CONNECTION BENDING STRENGTH, STIFFNESS AND DUCTILITY IN THE BIBLIOGRAPHY

As different cross-sections, timber grades and other features were used in the studies reported in the literature, a direct comparison of the results is only possible to a limited extent. Regarding bending strength, the comparable parameter is the effectiveness parameter (EP), defined as the ratio between the connection bending strength (CBS) and that of the connected timber elements (TBS). Regarding stiffness and given the variety of geometries considered by the different authors, it is also convenient to use a relative rather than an absolute value, as in the case of the bending strength. However, the comparison is far more complicated. While the joint may be looked at as a notional spring and a stiffness value (measured from tests) may be directly assigned to it, the procedure is not so simple for the connected elements. The rotational stiffness of a prismatic bar is in proportion with EI/L, as known from the basic Strength of Materials. That is, it depends not only on the cross section, but also on the element length. For instance, when the cross-section depth doubles, with everything else kept constant, the stiffness multiplies by a factor of 8; when the element length doubles, with everything else kept constant, the stiffness is halved. Therefore, the direct comparison of the measured stiffness of two tests is meaningless, unless they have elements with the same dimensions. To have a way out of this, it is considered the assumption that the ratio h/L is constant for all specimens. Even if this is not exact, it will allow setting comparable stiffness values, with an error smaller than one order of magnitude. Under this assumption, the measured stiffness of one test may be compared to that of a reference test if it is multiplied by (EIref/EI), where EIref is the stiffness of the reference test, and EI that of the test under consideration. This will be called the modified stiffness (MS). The reference test selected to be the reference for all the tests on the bibliography is the cross section of b=100mm h=200mm and timber grade of GL24h. Finally, the ductility ratios may be directly compared, as they are dimensionless parameters obtained from (2), where test-measured values are used. All these values are organized in the following tables, based on available results in the references. When values were not explicitly provided in the source, they were approximated from graphs presented by the authors, as accurately as possible. Blank values mean no information reported by the authors on that parameter.

Table 2 provides a comprehensive comparison of polygonal connections (Figure 1), where each specimen within this group is of the polygonal type, differing in terms of fasteners and reinforcements utilized by the respective authors. The "joint type" column outlines specific characteristics for each connection. For instance, "NR" means a connection with no reinforcements, while "R-" indicates the use of some form of reinforcement. In the case of Bakel et al. (2017) [9], "R-DWV" denotes the reinforcement with densified veneer wood (DWV) plates. Yang et al. (2019) [10] delved into the study of five types of reinforcements, including FP (Fiber plates), EBFRP (Externally bonded Fiber Reinforced Polymer), SP (Steel plates), NSM (Near-Surface Mounted CFRP rods), and ER (Epoxy repair). Basterrechea et al. (2021) [12] considered various ("V") timber cross-sections. Johanides et al. (2022) [15] explored two types of fasteners, namely bolts and dowels (A), and fully threaded screws (B). They also investigated two locations of reinforcements perpendicular to the grain: R1- in the beam only and R2-in both the beam and columns.

It is evident, except for [9], that these connections are characterized by notably low stiffness. Their resistance to bending moments is also comparatively low, ranging from an EP of 15% up to a maximum of 54%. Nevertheless, most of the reported ductility values surpass 4, indicating a ductile connection. Even the least ductile cases register values above 3, verging on the higher range of the class "limited ductility".

Table 3 presents results for the "plane polygonal type" group, which involves a flat connection where internal or external steel plates within or on the faces of the timber elements, respectively (Figure 2), are used. In comparison to traditional polygonal connections, with three timber elements, plane polygonal connections generally show higher stiffness and bending strength. This improvement is due to the contribution of the steel plates. However, this enhancement in structural performance comes at the expense of reduced ductility. Many authors reported brittle failure behaviour, particularly caused by tension perpendicular to the grain and longitudinal shear. In order to address this, most researchers opted to include reinforcement provisions. The most prevalent approach involved adding perpendicular-to-grain reinforcement using self-tapping screws (STS). This adjustment aimed to address the lower ductility exhibited by plane polygonal connections while maintaining their improved stiffness and bending strength characteristics. Similar to Table 2, in the "joint type" column, "NR" signifies a connection without reinforcements, while "R-" indicates the utilization of some form of reinforcement. "STS" denotes the most commonly utilized reinforcement method employing self-tapping screws, "PRR" represents plain round rods, "CLT" signifies cross laminated timber, and "DWV" indicates densified veneer wood plates. The reference section for the calculation of the modified stiffness (MS) was the same as in Table 2.

Table 2. Comparisons of results for Polygonal type reported on the experimental bibliography

Authors	Joint type	TG	b (mm)	h (mm)	TBS (kNm)	CBS (kNm)	EP	S (kNm/rad)	MS (kNm/rad)	D (-)
2017 - Bakel *et al.* [9]	R-DWV	GL24h	45	300	16.2	22.6	140%	2723	1715	-
2017 - Bakel *et al.* [9]	R- DWV	GL24h	80	600	115.2	107.9	94%	26975	1195	-
2019 - Yang *et al.* [10]	NR	GL40h	135	245	54	8	15%	153	102	5.56
2019 - Yang *et al.* [10]	ER	GL40h	135	245	54	9.7	18%	230	154	3.27
2019 - Yang *et al.* [10]	R-EBFRP	GL40h	135	245	54	10.5	19%	252	168	6.72
2019 - Yang *et al.* [10]	R-NSM	GL40h	135	245	54	11.8	22%	247	165	5.82
2019 - Yang *et al.* [10]	R-SP	GL40h	135	245	54	24.6	46%	751	502	3.48
2019 - Yang *et al.* [10]	R-FP	GL40h	135	245	54	12.8	24%	288	193	6.16
2021 - Basterrechea *et al.* [12]	V100	C24	100	200	16	3.8	24%	66	66	-
2021 - Basterrechea *et al.* [12]	V50	C24	50	200	8	2.7	34%	48	96	-
2021 - Basterrechea *et al.* [12]	V65	C24	65	130	4.4	1.7	39%	24	134	-
2021 - Basterrechea *et al.* [12]	V60	C24	60	200	9.6	4.4	46%	108	180	-
2022 - Johanides *et al.* [14]	A	GL24h	100	300	36	17	47%	-	-	7.05
2022 - Johanides *et al.* [14]	B	GL24h	100	300	36	18.9	53%	-	-	3.71
2022b - Johanides *et al.* [15]	A – R-1	GL24h	100	300	36	19.5	54%	-	-	7.86
2022b - Johanides *et al.* [15]	A – R-2	GL24h	100	300	36	19.4	54%	-	-	8.64
2022b - Johanides *et al.* [15]	B – R-1	GL24h	100	300	36	19.1	53%	-	-	3.85
2022b - Johanides *et al.* [15]	B – R-2	GL24h	100	300	36	18.7	52%	-	-	4.24

TG (timber grade); b (base); h (height); TBS (timber beam strength); CBS (connection bending strength); EP (effectiveness parameter); S (stiffness); MS (modified stiffness); D (ductility).

Table 3. Comparisons of results for plane polygonal type reported on the experimental bibliography

Authors	Joint type	TG	b (mm)	h (mm)	TBS (kNm)	CBS (kNm)	EP	S (kNm/rad)	MS (kNm/rad)	D (-)
2008 - Lam et al. [4]	NR	GL24h	130	304	48.1	33.6	70%	822	172	brittle
2008 - Lam et al. [4]	R-STS	GL24h	130	304	48.1	64.2	133%	819	172	<5.97
2013 – Leijten and Brendon [5]	R - DWV	C24	60	300	21.6	40	185%	4744	2343	-
2015 - Wang et al. [8]	NR	GL24h	130	305	48.4	38.4	79%	504	105	1.5
2015 - Wang et al. [8]	R -STS	GL24h	130	305	48.4	69.3	143%	407	84	3.3
2015 - Wang et al. [8]	R -CLT	GL24h	130	305	48.4	58.3	120%	441	91	3.5
2015 - He and Liu [3]	NR	GL24h	200	300	72	17.9	25%	257	36	-
2015 - He and Liu [3]	R - PRR	GL24h	200	300	72	23.2	32%	256	36	1.25
2015 - He and Liu [3]	R - STS	GL24h	200	300	72	33.4	46%	276	39	2.57
2017 - Solarino et al. [7]	NR	GL24h	140	300	50.4	37.7	75%	696	141	brittle
2019 - Shu et al. [6]	NR-S1	GL24h	180	280	56.4	21.4	38%	598	116	-
2019 - Shu et al. [6]	NR-S2	GL24h	180	320	73.7	26	35%	827	107	-
2021 - Dong et al. [2]	NR	C22	315	400	184.8	130.7	71%	8897	388	1.7
2021 - Dong et al. [2]	R-STS	C22	315	400	184.8	149.4	81%	9023	394	4.5

TG (timber grade); b (base); h (height); TBS (timber beam strength); CBS (connection bending strength); EP (effectiveness parameter); S (stiffness); MS (modified stiffness); D (ductility).

Table 4 presents the results for solutions based on the lever effect, where EB represents Extended Beam, and EC represents Extended Column. This group exhibits significant variability in terms of the type of fasteners used; for instance, [24] employed glued-in rods, [25] utilized screws, and [17], [26] developed specific metallic components. Upon analysing Table 4, lever effect connections prove to be competitive in rotational stiffness and moment resistance compared to the previously discussed solutions. However, these solutions are generally characterized by low ductility. It is essential to note that the assessment of ductility remains somewhat uncertain, as it has been less extensively discussed in the studied references. The diverse range of fasteners and reinforcements used within this group contributes to the variability in performance observed across different studies.

Table 4. Comparisons of results for lever effect type reported on the experimental bibliography

Authors	Joint type	TG	b (mm)	h (mm)	TBS (kNm)	CBS (kNm)	EP	S (kNm/rad)	MS (kNm/rad)	D (-)
2004 - van Houtte et al. [24]	EB	LVL24	63	600	90.7	118.7	131%	-	-	-
2004 - van Houtte et al. [24]	EB	LVL24	89	800	227.8	206.8	91%	-	-	-
2004 - van Houtte et al. [24]	EB	LVL24	105	1000	420	424.5	101%	-	-	-
2006 - Nakatani et al. [25]	EC	GL24h	240	400	153.6	75	49%	4364	217	1.98
2006 - Nakatani et al. [25]	EC	GL24h	240	600	240	122	51%	8904	131	2.93
2006 - Nakatani et al. [25]	EC	GL24h	240	800	345.6	153	44%	16301	102	2.64
2006 - Komatsu et al. [17]	EC - B300	GL30h	120	360	77.8	28.8	37%	5760	666	-
2006 - Komatsu et al. [17]	EC - B360	GL30h	120	420	105.8	37	35%	7400	539	-
2006 - Komatsu et al. [17]	EC - B450	GL30h	120	540	175	48.3	28%	12075	413	-
2008 - Komatsu et al. [17]	EC - L300	GL30h	120	360	77.8	33.6	43%	4800	555	-
2008 - Komatsu et al. [17]	EC - L360	GL30h	120	420	105.8	44.3	42%	6329	461	-
2008 - Komatsu et al. [17]	EC - L450	GL30h	120	540	175	75.3	43%	15060	516	-
2008 - Madhoushi and Ansel [18]	EB	C24	51	100	2	2.3	115%	-	-	-
2008 – Vasek [19]	EC	GL24h	-	-	-	38	-	-	-	-
2010 – Fragiacomo and Batchelar [20]	EB	-	-	-	-	-	-	1821	-	-
2012 – Scheibmair [21]	EB	LVL45	105	1050	868.2	588	68%	5957	39	-
2017 - O´neil et al. [22]	EB (OFC3)	C16	249	600	156.2	23.4	15%	-	-	-
2017 - O´neil et al. [22]	EB (PFC3)	C16	249	600	156.2	52.5	34%	-	-	-
2017 - O´neil et al. [22]	EB (PFC2)	C16	249	600	156.2	51.2	33%	-	-	-
2014 - Walker and Xiao [28]	Diagonal	LVL44	90	360	86.9	19.7	23%	-	-	-
2022 - Navaratnan et al. [29]	Steel-C	GL24h	385	1000	1540	634	41%	123000	244	2

TG (timber grade); b (base); h (height); TBS (timber beam strength); CBS (connection bending strength); EP (effectiveness parameter); S (stiffness); MS (modified stiffness); D (ductility).

Table 5 provides insights into the Concrete Corner (C-C) joint type, wherein the intersection region between the beam and column is replaced with high-strength concrete. According to experiments conducted by Gonçalves (2014) [41], D'Alva (2016) [42], and Ribeiro et al. (2024) [31], this connection demonstrated nearly full strength and exhibited high stiffness, as evidenced by significantly elevated values of modified stiffness when compared to all the evaluated solutions. Regarding ductility, [31] reported a mechanically ductile failure characterized by rod yielding. However, the ductility values were relatively low, ranging from elastic (D = 1.84) for the Φ12 specimens to limited ductility for the Φ16 specimens (D = 2.49) according to Table 1. It is worth noting that the authors of this article have conducted experiments with the same dimensions but with reinforcements using screws and concrete with fibres, which yielded superior ductility results. These findings will be published in a future article.

Table 5. Comparisons of results for concrete corners type reported on the experimental bibliography

Authors	Joint type	TG	b (mm)	h (mm)	TBS (kNm)	CBS (kNm)	EP	S (kNm/rad)	MS (kNm/rad)	D (-)
2014 – Gonçalves [41]	C-C	C35	80	160	11.9	9.8	82%	1000	2066	-
2016 - D`Alva [42]	C-C	C35	80	160	11.9	10.4	87%	1000	2066	-
2024 - Ribeiro *et al.* [31]	C-C (Φ12)	C32	100	225	33.7	18.7	55%	2341	1507	1.84
2024 - Ribeiro *et al.* [31]	C-C (Φ16)	C32	100	225	33.7	20.5	61%	6027	3880	2.49

TG (timber grade); b (base); h (height); TBS (timber beam strength); CBS (connection bending strength); EP (effectiveness parameter); S (stiffness); MS (modified stiffness); D (ductility).

In Table 6, various results from the literature for the prestressed connections group are presented (Figure 5). Smith et al. (2014) carried out tests with a prestressing force (PT) varying between 50kN and 250kN. As the post-tensioning force increased, the effectiveness parameter (EP) improved, but there was a trade-off with reduced ductility. Even with decreased ductility, the connections remained classified as "ductile" based on Table 1. Li et al. (2018) explored combinations of variables, including post-tension force (PT: 45kN or 60kN), steel plate (SP), self-tapping screws (STS), steel cap (SC), steel angles (SA), and the use or absence of disc springs (DS) in seven groups. Notably, groups with energy dissipation devices exhibited higher EP compared to the initial two groups without such devices. Wanninger & Frangi (2014) stood out in this category, demonstrating the highest modified stiffness and significant ductility. Their approach involved using hardwood to reinforce the column region subjected to compressive perpendicular stresses and applying a prestress force of 550kN.

Table 6. Comparisons of results for prestressed connections type reported on the experimental bibliography

Authors	Joint type	TG	b (mm)	h (mm)	TBS (kNm)	CBS (kNm)	EP	S (kNm/rad)	MS (kNm/rad)	D (-)
2014 - Smith et al. [32]	PT-50kN	GL32h	240	483	298.6	20	7%	4000	92	10
2014 - Smith et al. [32]	PT-100kN	GL32h	240	483	298.6	29	10%	4000	92	8
2014 - Smith et al. [32]	PT-150kN	GL32h	240	483	298.6	38	13%	6000	137	6
2014 - Smith et al. [32]	PT-200kN	GL32h	240	483	298.6	49	16%	4000	92	5
2014 - Smith et al. [32]	PT-250kN	GL32h	240	483	298.6	60	20%	6154	141	4
2014 - Wanninger and Frangi [37]	R-Hardwood PT-550kN	GL24h	400	600	576	198	34%	32000	283	16
2018 - Li et al. [35]	S1-PT-45kN+SP	GL28h	200	300	84	19.92	24%	846	109	-
2018 - Li et al. [35]	S2-PT-45kN+STS	GL28h	200	300	84	15.74	19%	761	98	-
2018 - Li et al. [35]	S3-PT-45kN+S-P+SC	GL28h	200	300	84	47.97	57%	752	97	-
2018 - Li et al. [35]	S4-PT-60kN+S-P+SC	GL28h	200	300	84	55.38	66%	780	101	-
2018 - Li et al. [35]	S5-PT-45kN+S-P+SC+DS	GL28h	200	300	84	46.78	56%	847	110	-
2018 - Li et al. [35]	S6-PT-60kN+S-P+SC+DS	GL28h	200	300	84	50.19	60%	861	111	-
2018 - Li et al. [35]	S7-PT-45kN+S-P+SA	GL28h	200	300	84	49.51	59%	914	118	-

TG (timber grade); b (base); h (height); TBS (timber beam strength); CBS (connection bending strength); EP (effectiveness parameter); S (stiffness); MS (modified stiffness); D (ductility).

4. CONCLUSIONS

As the main conclusion, the review of moment-resisting connections for timber frames reveals a diverse range of solutions, each with its unique characteristics and challenges. The three main types of connections discussed – polygonal, lever-based, and prestressed – offer different approaches to address the discontinuity of wood fibres at beam-column intersections.

The polygonal connections, commonly relying on steel bolts, are widely used in practice. Despite their proven safety, they exhibit limitations in terms of rotational stiffness and bending strength, always leading to partial strength connections. Researchers continue to explore variations and enhancements to attenuate these limitations, considering factors such as fastener types, reinforcements, steel plates and timber grades.

Lever-based connections, employing the lever principle for load transmission, introduce an innovative approach. While these connections demonstrate competitive rotational stiffness and moment resistance, they face challenges associated with high compressive forces perpendicular to the wood grain. Some studies have explored solutions to mitigate these challenges, including the implementation of diagonal wood splits and the use of steel corners.

A new approach proposed by the authors, with the incorporation of reinforced concrete, stands out with the highest modified stiffness among all and nearly full strength.

Prestressed connections, relying on the introduction of prestress to balance moments, exhibit high ductility. However, challenges related to uncertainties in quantifying tendon tension loss and high compressive forces perpendicular to the wood grain need further investigation. Different approaches, such as varying prestressing forces and incorporating energy dissipation devices, are explored to optimize the performance of prestressed connections.

The structural properties of moment-resisting joints, including bending strength, stiffness, and ductility, are crucial for their performance in timber frames. While there is variability in the results reported in the literature due to different connection types, materials, and testing conditions, the presented comparisons provide valuable insights into the strengths and limitations of each connection type. In summary, this review contributes to the understanding of moment-resisting connections for timber frames, offering a comprehensive overview of existing solutions and their structural behaviour. Further research and innovation in this field are essential to address challenges, improve performance, and promote the use of timber in construction.

DECLARATION OF COMPETING INTEREST

The authors of this article declare that they have no financial, professional or personal conflicts of interest that could have inappropriately influenced this work.

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REFERENCES

[1] A. Faria and J. Negrão, *Projecto de Estruturas de Madeira*. 2009.
[2] W. Dong, M. Li, M. He, and Z. Li, 'Experimental Testing and Analytical Modeling of Glulam Moment Connections with Self-Drilling Dowels', *Journal of Structural Engineering*, vol. 147, no. 5, May 2021, doi: 10.1061/(ASCE)ST.1943-541X.0002977.
[3] M. J. He and H. F. Liu, 'Comparison of glulam post-to-beam connections reinforced by two different dowel-type fasteners', *Constr Build Mater*, vol. 99, pp. 99–108, Nov. 2015, doi: 10.1016/j.conbuildmat.2015.09.005.
[4] F. Lam, M. Schulte-Wrede, C. C. Yao, and J. J. Gu, 'Moment resistance of bolted timber connections with perpendicular to grain reinforcements', *10th World Conference on Timber Engineering 2008*, vol. 2, pp. 978–985, 2008.
[5] A. J. M. Leijten and D. Brandon, 'Advances in moment transfering dvw reinforced timber connections – Analysis and experimental verification, Part 1', *Constr Build Mater*, vol. 43, pp. 614–622, 2013, doi: 10.1016/j.conbuildmat.2013.03.020.
[6] Z. Shu, Z. Li, X. Yu, J. Zhang, and M. He, 'Rotational performance of glulam bolted joints: Experimental investigation and analytical approach', *Constr Build Mater*, vol. 213, pp. 675–695, 2019, doi: 10.1016/j.conbuildmat.2019.03.002.
[7] F. Solarino, L. Giresini, W. S. Chang, and H. Huang, 'Experimental tests on a dowel-type timber connection and validation of numerical models', *Buildings*, vol. 7, no. 4, Dec. 2017, doi: 10.3390/buildings7040116.
[8] M. Wang, X. Song, X. Gu, Y. Zhang, and L. Luo, 'Rotational Behavior of Bolted Beam-to-Column Connections with Locally Cross-Laminated Glulam', *Journal of Structural Engineering*, vol. 141, no. 4, 2015, doi: 10.1061/(asce)st.1943-541x.0001035.
[9] R. van Bakel, G. Rinaldin, A. J. M. Leijten, and M. Fragiacomo, 'Experimental–numerical investigation on the seismic behaviour of moment-resisting timber frames with densified veneer wood-reinforced timber joints and expanded tube fasteners', *Earthq Eng Struct Dyn*, vol. 46, no. 8, pp. 1307–1324, Jul. 2017, doi: 10.1002/eqe.2857.
[10] J. Q. Yang, S. T. Smith, and Z. Wang, 'Seismic behaviour of fibre-reinforced-polymer- and steel-strengthened timber connections', *Advances in Structural Engineering*, vol. 22, no. 2, pp. 502–518, Jan. 2019, doi: 10.1177/1369433218794001.
[11] J. Q. Yang, S. T. Smith, Z. Wang, P. Feng, and N. Sirach, 'Modelling of hysteresis behaviour of moment-resisting timber joints strengthened with FRP composites', *Int J Mech Sci*, vol. 179, Aug. 2020, doi: 10.1016/j.ijmecsci.2020.105593.
[12] M. Basterrechea-Arévalo, J. M. Cabrero, B. Iraola, and R. Goñi, 'Modelling of moment transmitting beam-to-column timber connections accounting for frictional transmission', *Eng Struct*, vol. 247, Nov. 2021, doi: 10.1016/j.engstruct.2021.113122.
[13] M. Johanides, L. Kubíncová, D. Mikolášek, A. Lokaj, O. Sucharda, and P. Mynarčík, 'Analysis of rotational stiffness of the timber frame connection', *Sustainability (Switzerland)*, vol. 13, no. 1, pp. 1–15, Jan. 2021, doi: 10.3390/su13010156.
[14] M. Johanides, A. Lokaj, P. Dobes, and D. Mikolasek, 'Numerical and Experimental Analysis of the Load-Carrying Capacity of a Timber Semi-Rigid Dowel-Type Connection', *Materials*, vol. 15, no. 20, Oct. 2022, doi: 10.3390/ma15207222.
[15] M. Johanides, A. Lokaj, D. Mikolasek, P. Mynarcik, P. Dobes, and O. Sucharda, 'Timber Semirigid Frame Connection with Improved Deformation Capacity and Ductility', *Buildings*, vol. 12, no. 5, May 2022, doi: 10.3390/buildings12050583.
[16] M. Batchelar, 'Timber Frame Moment Joints with Glued-In Steel Rods-A Designer's Perspective', in *9th World Conference on Timber Engineering*, 2006.
[17] K. Komatsu, K. Hosokawa, S. Hattori, H. Matsuoka, K. Yanaga, and T. Mori, 'Development of Ductile and High-Strength Semi-Rigid Portal Frame Composed of Mixed-Species Glulams and H-shaped Steel Gusset Joints', in *Proceedings of the World Conference on Timber Engineering 2006*, Portland, 2006.
[18] M. Madhoushi and M. P. Ansell, 'Behaviour of timber connections using glued-in GFRP rods under fatigue loading. Part II: Moment-resisting connections', *Compos B Eng*, vol. 39, no. 2, pp. 249–257, Mar. 2008, doi: 10.1016/J.COMPOSITESB.2006.11.002.
[19] M. Vašek, 'Semi rigid Timber Frame and Space Structure Connections by Glued-in Rods Experimental program', in *Proceedings of the 10th world conference on timber engineering*, Miyazaki, Japan, 2008.
[20] M. Fragiacomo, M. Batchelar, C. Wellington, and A. Buchanan, 'Moment joints in timber frames using glued-in steel rods: Experimental investigation of long-term performance', *11th World Conference on Timber Engineering 2010, WCTE 2010*, vol. 1, pp. 700–709, 2010.
[21] F. Scheibmair, 'Expediente Moment Connections for Timber Portal Frame Buildings', University of Auckland, New Zealand, 2012.
[22] C. O'Neill, D. McPolin, S. E. Taylor, A. M. Harte, C. O'Ceallaigh, and K. S. Sikora, 'Timber moment connections using glued-in basalt FRP rods', *Constr Build Mater*, vol. 145, pp. 226–235, Aug. 2017, doi: 10.1016/j.conbuildmat.2017.03.241.
[23] L. F. Sirumbal-Zapata, C. Málaga-Chuquitaype, and A. Y. Elghazouli, 'Experimental assessment and damage modelling of hybrid timber beam-to-steel column connections under cyclic loads', *Eng Struct*, vol. 200, Dec. 2019, doi: 10.1016/j.engstruct.2019.109682.
[24] A. van Houtte, A. Buchanan, and P. Moss, 'Moment-Resisting Connections In Laminated Veneer Lumber (LVL) Frames', *8th World Conference on Timber Engineering*, pp. 247–252, 2004.
[25] M. Nakatani, T. Mori, and K. Komatsu, 'Development of moment-resisting joint systems using lagscrewbolts', *9th World Conference on Timber Engineering 2006, WCTE 2006*, vol. 1, pp. 845–851, 2006.
[26] P. Zardani and P. Quenneville, 'Design method for coupled-splice timber moment connections', in *WCTE 2014 - Word conference on timber engineering 2014*, Quebec city - Canada, 2014.
[27] M. Fragiacomo and M. Batchelar, 'Timber frame moment joints with glued-in steel rods. I: Design', *Journal of Structural Engineering (United States)*, vol. 138, no. 6, pp. 789–801, 2012, doi: 10.1061/(ASCE)ST.1943-541X.0000419.
[28] J. Walker and R. Xiao, 'Experimental Testing of a Portal Frame Connection Using Glued-In Steel Rods', in *Materials and Joints in Timber Structures, RILEM bookseries 9*, 2014, pp. 555–566. doi: 10.1007/978-94-007-7811-5_51.
[29] S. Navaratnam, J. Thamboo, T. Ponnampalam, S. Venkatesan, and K. B. Chong, 'Mechanical performance of glued-in rod glulam beam to column moment connection: An experimental study', *Journal of Building Engineering*, vol. 50, Jun. 2022, doi: 10.1016/j.jobe.2022.104131.
[30] J. H. Negrão, L. D. Brito, A. G. Dias, C. C. Júnior, and F. R. Lahr, 'Numerical and experimental study of small-scale moment-resistant reinforced concrete joints for timber frames', *Constr Build Mater*, vol. 118, pp. 89–103, 2016, doi: 10.1016/j.conbuildmat.2016.05.036.
[31] A. Ribeiro, J. Negrão, A. Dias, A. Marques, and L. Cardoso, 'Reinforced concrete corner for timber frame moment resisting joints – Design and experimental assessment', *Eng Struct*, vol. 301, p. 117298, Feb. 2024, doi: 10.1016/j.engstruct.2023.117298.
[32] T. Smith *et al.*, 'Post-tensioned glulam beam-column joints with advanced damping systems: Testing and numerical analysis', *Journal of Earthquake Engineering*, vol. 18, no. 1, pp. 147–167, Jan. 2014, doi: 10.1080/13632469.2013.835291.
[33] M. P. Newcombe, S. Pampanin, A. Buchanan, and A. Palermo, 'Section analysis and cyclic behavior of post-tensioned jointed ductile connections for multi-story timber buildings', *Journal of Earthquake Engineering*, vol. 12, no. SUPPL. 1, pp. 83–110, 2008, doi: 10.1080/13632460801925632.
[34] Z. Li, X. Zheng, M. He, Y. Sun, and G. He, 'Experimental and analytical investigations into the time-dependent performance in post-tensioned timber beam-column joints under sustained loads and varied environment', *Constr Build Mater*, vol. 251, Aug. 2020, doi: 10.1016/j.conbuildmat.2020.118943.
[35] Z. Li, M. He, and K. Wang, 'Hysteretic Performance of Self-Centering Glulam Beam-to-Column Connections', *Journal of Structural Engineering*, vol. 144, no. 5, May 2018, doi: 10.1061/(asce)st.1943-541x.0002012.
[36] F. Wanninger and A. Frangi, 'Experimental and analytical analysis of a post-tensioned timber frame under horizontal loads', *Eng Struct*, vol. 113, pp. 16–25, Apr. 2016, doi: 10.1016/j.engstruct.2016.01.029.
[37] F. Wanninger and A. Frangi, 'Experimental and analytical analysis of a post-tensioned timber connection under gravity loads', *Eng Struct*, vol. 70, pp. 117–129, Jul. 2014, doi: 10.1016/j.engstruct.2014.03.042.
[38] 'European Committee for Standardization' CEN, 'EN 1998 Eurocode 8: Design of structures for earthquake resistance', 2005.
[39] H. J. Blass *et al.*, *Timber Engineering - STEP 1 - Basis of design, materials properties, structural components and joints*, First Edition. Centrum Hout, The Netherlands, 1995.
[40] R. H. Fairweather, 'Beam column connections for multi-storey timber buildings', Master's theses, University of Canterbury, 1992.
[41] T. Gonçalves, 'Estudo Experimental de Nós de Pórticos de Madeira em Betão Armado', Master's theses, University of Coimbra - FCTUC, Coimbra, Portugal, 2014.
[42] M. D'Alva, 'Estudo Experimental de Nós de Pórticos de Madeira em Betão Armado Sujeitos à Flexão Simples', Master's theses, University of Coimbra - FCTUC, Coimbra, Portugal, 2016.