Structures 88 (2026) 111622

Contents lists available at ScienceDirect

Structures

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

Experimental characterization of the seismic response of traditional hold-downs and angle brackets in CLT structures: A critical overview and an annotated catalogue

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ARTICLE INFO

Keywords:
CLT structures
Seismic behaviour
Traditional hold-downs
Traditional angle brackets
Overview
Annotated catalogue

ABSTRACT

This study presents a comprehensive review of the seismic performance of traditional hold-downs and angle brackets in Cross-Laminated Timber (CLT) structures. These mechanical anchors play a crucial role in dissipating energy and ensuring ductile behaviour in CLT structures in seismic-prone areas. Despite numerous experimental studies conducted worldwide, key differences and similarities in mechanical performance across different traditional products remain insufficiently summarized. An annotated catalogue is developed by systematically collecting and analysing data from 39 experimental campaigns conducted between 2012 and 2023. Moreover, original data and figures from experimental campaigns on traditional hold-downs and angle brackets, are incorporated, providing additional contribution and originality to the study. The review examines key mechanical properties such as stiffness, peak load, ductility and ultimate displacement considering the influence of test configurations, load protocols, fastener types, and wood species. Findings reveal that the mechanical behaviour of hold-downs and angle brackets is strongly influenced by fastener patterns, geometric configurations, and anchoring details. A comprehensive annotated catalogue is present to support future research and design improvements in seismic applications for CLT structures.

1. Introduction and background

Cross-Laminated Timber (CLT) structures have increasingly become a valuable solution in the construction of low- to mid-rise buildings, being a sustainable and efficient alternative to conventional materials such as steel and reinforced concrete, also in seismic prone areas [1]. Lightness and stiffness of CLT panels, combined with the ability to dissipate energy in connections, ensure CLT to exhibit commendable seismic performances [2].

Two structural systems are typically adopted in multi-storey CLT buildings: balloon-type and platform-type [3]. In balloon-type systems, the wall panels run along multiple storeys, minimizing discontinuities along the wall height and preventing perpendicular-to-grain compression in the floor panels. A more efficient and predictable lateral load path is obtained as highlighted in the analytical and experimental investigations conducted by Chen et al. [4] on balloon-type CLT shear walls. Conversely, in platform-type systems, floors are inserted between wall panels at each storey, and mechanical anchors (e.g. hold-downs and angle brackets) connect the CLT wall panels to the foundations or the floor below.

In both balloon- and platform-type systems, the hold-downs are placed at the outer bottom corners of the wall panels in order to prevent the shear wall overturning, while the angle brackets are typically distributed along the wall panels primarily to limit the horizontal sliding of the panels.

In low- to mid-rise buildings, traditional hold-downs and angle brackets are typically proprietary products, developed and designed by a specific manufacturer who is responsible for production process, performance assessment, and verification of consistency in structural properties. The assessment of the product's performance is conducted according to technical specifications (e.g. European Assessment Documents in Europe) developed for those construction products that are not or not fully covered by a harmonised product Standards. The essential performance characteristics of a proprietary product are included in a

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https://doi.org/10.1016/j.istruc.2026.111622
Received 4 September 2025; Received in revised form 1 March 2026; Accepted 9 March 2026

technical document, such as the European Technical Assessments (ETAs) in Europe or the ICC-ES Evaluation Reports (ESRs) in the United States.

CLT panels are connected one-to-another by means of different types of joints, such as: i) vertical joints in multi-panel shear walls, ii) vertical joints between orthogonal walls, iii) wall-to-floor joints, and iv) floor panel-to-panel joints. Self-tapping screws (STSs) are typically used in vertical and floor panel-to-panel joints, whereas a combination of STSs and angle brackets is generally employed for wall-to-floor and orthogonal walls joints [5]. Seismic Force Resisting Systems (SFRS) in CLT platform-type buildings typically consist of either single or multi-panel shear walls. Single-panel shear walls are made with a single monolithic panel, while multi-panel shear walls comprise two or more panels, connected one-to-another along vertical joints by means of dowel-type fasteners (i.e. STSs or nails).

Over the past two decades, several comprehensive experimental campaigns have been conducted to investigate the seismic response of platform-type CLT structures at building level. In the SOFIE project [6,7] a shake-table testing campaign was conducted on a three- and a seven-storey CLT buildings, providing the first deep insights into the seismic behaviour of CLT structures. Popovski et al. [8] investigated the mechanical behaviour of a two-story CLT building observing that the main failure mechanism was related to the yielding of the nails in hold-downs and angle brackets. The mechanical performance of a two-story CLT platform-type building under both monotonic and cyclic loads was investigated by Matos et al. [9]. The findings demonstrated the role of mechanical anchors in increasing the stiffness, resistance, and ductility of the shear walls, especially when subjected to cyclic loading. The results obtained from a shake-table test on a two-story CLT building conducted by Van de Lindt et al. [10] showed that CLT structures can dissipate a large amount of energy in the mechanical anchors while CLT panels behave almost elastically. Zhang et al. [11] analysed the mechanical behaviour under horizontal loads of one platform-type and one balloon-type three-storey CLT structure through quasi-static cyclic tests. The results revealed a comparable lateral resistance as well as energy dissipation of the two tested structures, suggesting that both structural types may be considered a valuable solution for mid-rise buildings in seismic prone areas. Hristovski et al. [12,13] and Sustersic et al. [14] emphasized the role of connections for the prediction of the seismic performance of CLT structures.

A review of the experimental studies conducted on CLT structures at building level makes it clear that the seismic performance of CLT structural systems is predominantly governed by connections, responsible for the energy dissipation and ductile behaviour of the structural system, while CLT panels behave almost elastically. In particular, hold-downs and angle brackets, play a crucial role in anchoring the shear walls to the foundation or the floor below. Due to their pivotal role in the seismic response of CLT structures, more than 39 experimental campaigns on traditional hold-downs and angle brackets were conducted between 2012 and 2023 worldwide, as detailed in Section 2.1. In addition, original data and figures obtained from experimental campaigns conducted at the Testing Laboratory archive of the Institute of Bioeconomy, National Research Council of Italy (CNR-IBE), and at the Mechanical Testing Laboratory archive of the Department of Civil Engineering, University of Bologna, and not published in previous papers are also included in this work, providing additional value and originality to the study. Such experimental campaigns were designed and executed on hold-down and angle brackets produced by different manufacturers, through different set-up configurations and load protocols, selecting different types, number and patterns of fasteners as well as adopting different wood species. The data and information collected from such campaigns were essential for the characterization of the seismic performance of mechanical anchors used in CLT buildings, providing a significant advancement and contribution to the entire sector of timber structures.

Despite the large amount of studies available in literature, to the authors' knowledge only a relatively limited number of them have attempted to collect and compare the information and data reported, summarizing the key differences and similarities in the mechanical performance of different traditional hold-downs and angle brackets tested under different set-up configurations. A web-based database initiative was recently developed by FPInnovations [15] to support modelling and design of mass timber structures, and a comprehensive database aimed at the comparative performance assessment of connections was presented by Heidari et al. [16].

For these reasons, this study presents a critical deep overview of the seismic response of traditional hold-downs and angle brackets for CLT structures with the aim to:

i) report the key information from different experimental campaigns on different traditional hold-downs and angle brackets in an annotated catalogue;
ii) summarize, compare, and analyse the mechanical parameters (e.g. stiffness, peak load, ductility and ultimate displacement), as declared in the original experimental studies, for different mechanical anchors tested across various configurations;
iii) identify similarities among traditional hold-downs and angle brackets from different manufacturers;
iv) assess the mechanical limitations in the use of traditional hold-downs and angle brackets in CLT buildings in seismic regions.

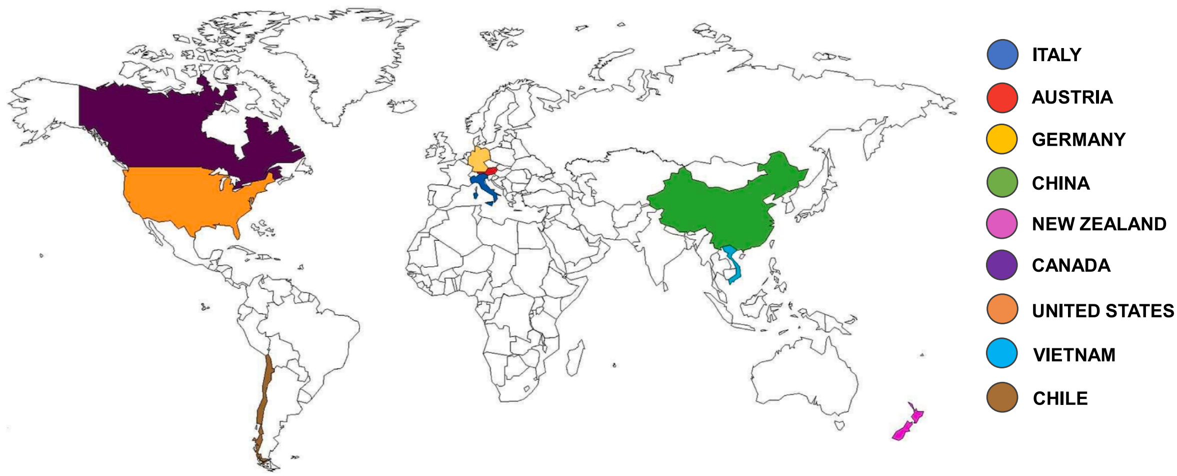
The outcome of this study will potentially act as a useful resource for guiding future experimental research and may lay the base for developing more effective hold-downs and angle brackets.

2. The annotated catalogue

The annotated catalogue is intended to include multiple information of different traditional mechanical anchors. It is noteworthy to mention that the data (i.e. load-displacement curves) obtained from the original tests are not included in this catalogue since accessible and available from the studies in literature. A total of 15 and 24 experimental studies on hold-downs and angle brackets, respectively, are discussed and included in the annotated catalogue. As shown in Fig. 1, until 2018, the majority of experimental tests on hold-downs and angle brackets was conducted in Europe and Canada. Conversely, since 2018 a valuable contribution has been given from experimental studies conducted in China, Chile, United States, New Zealand and Vietnam.

2.1. Studies on hold-downs

Acler [17] carried out an extensive experimental campaign on four different types of hold-downs with the aim to analyses the influence of the fastener pattern throughout both monotonic and cyclic load protocols. The findings showed that the mechanical behaviour (i.e. stiffness, peak load and failure modes) is strongly influenced by the total number of fasteners. Full-nailing patterns promoted a brittle failure of the steel plate while yielding of the fasteners was reached when a partial-nailing pattern was adopted. A significant plastic deformation of the base steel plate was observed in the hold-down where no washer was used. Flatscher et al. [18,19] investigated the behaviour of a single type of hold-down in both wall-to-floor and wall-to-foundation joint configuration under either monotonic or cyclic load protocols. A partial-nailing pattern was adopted for all specimens, reaching a ductile failure mode with yielding of fasteners. Minimal discrepancies in terms of peak load, stiffness, and ductility were observed among the specimens with different load protocols (cyclic vs monotonic) and joint configurations (wall-to-floor vs wall-to-foundation). Gavric et al. [20,21] examined the cyclic behaviour of two types of hold-downs with partial-nailing patterns under uni-directional vertical-tensile and horizontal-shear load directions. The study confirmed that the adoption of partial-nailing pattern promoted a ductile failure for both load directions, despite two different failure modes were observed. Under a vertical-tensile load,



Year	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023
Hold-down	[17] [21]	-	-	[18] [20]	[31]	[19] [22] [29]	[30]	[23] [27] [32]	[26]	-	-	[25] [28]
Angle bracket	[17] [21]	[33]	[36]	[18] [20] [41]	[31] [47]	[19] [22]	[39] [40] [44]	[34] [38] [45]	-	[37] [35] [43] [46]	-	[25] [28] [42]

Fig. 1. Overview of the experimental campaigns conducted on traditional hold-downs and angle brackets, including: i) references to the studies; ii) geographic distribution based on the affiliation of the corresponding author; and iii) timeline of the studies.

a progressive yielding of nails was observed; a shear-torsional plastic deformation of the steel plate was conversely reached under horizontal-shear load protocol. Hummel [22] investigated a single type of hold-downs for both wall-to-floor and wall-to-foundation joint configurations under uni-directional vertical-tensile and horizontal-shear load directions. The findings were similar to those presented by Flatscher et al. [18,19] and Gavric et al. [20,21], confirming that a ductile failure mode in both load directions can be reached with a partial-nailing pattern, even though two distinct failure mechanisms are observed under tensile and shear load direction. The influence of nailing pattern was specifically studied by Polastri et al. [23] throughout a comprehensive experimental campaign conducted on four types of hold-downs. The results showed that the hold-down resistance is linearly proportional to the number of nails when a ductile failure (i.e. yielding of fasteners) is reached throughout a partial-nailing pattern. Masroor et al. [24,25] conducted an extensive experimental campaign on a single type of hold-down, examining full- and partial-nailing patterns under uni-directional vertical-tensile and horizontal-shear load directions. Both monotonic and cyclic load protocols were carried out. The hold-downs with full-nailing patterns showed a brittle failure mode in the steel plate, while those with partial-nailing pattern were characterized by yielding of nails. Under tensile load, despite a greater ultimate displacement, partial-nailing hold-downs exhibited lower stiffness than full-nailing hold-downs. Under horizontal-shear load, full-nailing hold-downs were characterized by a limited resistance primarily due to rotational effects and local buckling of the steel plates.

While the previous studies focused on hold-downs connected to CLT panels made with Spruce, other studies examined the influence of other wood species. Radiata Pine and Douglas Fir CLT panel were used in the tests conducted by Dong et al. [26]. The results showed that hold-down connected to Radiata Pine panels exhibited higher ductility but lower peak load than those connected to Douglas Fir. Benedetti et al. [27] tested hold-downs fastened to CLT panels made with Radiata Pine by means of either screws or nails. The tests were conducted under both monotonic and cyclic load protocols. The values of peak load obtained from the tests were similar to those conducted on the same types of hold-downs connected to CLT panels made with other wood species. Conversely, a lower value of stiffness was obtained on hold-downs connected to Radiata Pine CLT panels. It was also found that hold-downs fastened with screws reached higher values of peak load and stiffness but lower values of ductility than those connected with nails. The experimental campaign conducted by Khai Tran et al. [28] investigated the influence of the panel wood densities and number of fasteners (i.e. screws) by using both Larch and Pine CLT panels. The hold-downs connected to Larch CLT panels exhibited greater stiffness and peak load but less ductility compared to those connected to CLT panels made with Pine.

Ferracuti et al. [29], Pozza et al. [30], and Liu et al. [31,32] investigated the behaviour of hold-downs under a bi-directional vertical-tensile and horizontal-shear load direction. The results showed values of peak load from bi-directional load protocol lower than those obtained from previous studies for uni-directional loads.

2.2. Studies on angle brackets

Over the last years angle brackets have undergone a significant technological evolution that has primarily involved substantial modifications in the geometrical dimensions and number of fasteners. The first-generation of angle brackets was adopted in the first years after the appearances of CLT in the market and consisted of mechanical anchors typically used in light-frame timber structures; since 2019, a second-generation of angle brackets had been specifically designed for CLT structures with larger dimensions and higher number of fasteners than those adopted in the first-generation.

Several experimental tests were conducted on the first-generation of angle brackets between 2012 and 2019. Gavric et al. [20,21] investigated the cyclic behaviour of two types of angle brackets in two joint configurations (i.e. wall-to-floor and wall-to-foundation configurations) under uni-directional vertical-tensile and horizontal-shear load direction. The tests conducted under vertical-tensile load revealed a brittle failure mode for both joint configurations, predominantly due to either the anchor bolt pull-through or the nail withdrawal occurring in the steel base plate. Under horizontal-shear load, angle brackets exhibited a ductile failure mode primarily governed by the yielding of nails. Flatscher et al. [18,19] focused on a single type of angle bracket used in a wall-to-foundation and wall-to-floor joint configurations. The angle brackets subjected to a horizontal-shear load exhibited comparable values of peak load and stiffness in the two joint configurations, where the failure mode was governed by the yielding of fasteners. Conversely, the angle brackets tested under vertical-tensile load showed a brittle failure mode. A greater ductility was reached in wall-to-floor configuration than that obtained in the wall-to-foundation joint. Hummel [22] investigated the cyclic behaviour of a single type of angle bracket in wall-to-floor and wall-to-foundation joint configurations. The mechanical performance under vertical-tensile load was negatively affected by the significant plastic deformation of the steel plate, whereas yielding of nails was observed in the tests conducted under horizontal-shear load direction. Shen et al. [33–35], Schneider et al. [36] and Rezvani et al. [37] specifically studied the impact of the fastener patterns under uni-directional vertical-tensile and horizontal-shear load direction. The tests emphasized that the failure modes as well as the mechanical properties of the angle brackets are highly influenced by the fastener number, type and dimension under both horizontal-shear and vertical-tensile loads. Mahdavifar et al. [38] investigated the mechanical behaviour of two types of angle brackets connected to CLT panels composed of layers with different wood species and density. The results showed that the energy dissipation and the peak load are significantly influenced by the wood density of laminations. Under vertical-tensile load, the failure modes were characterized by the fastener withdrawal, leading to a value of peak load and ductility lower than those found for homogeneous CLT panels. Conversely, under horizontal-shear load, a failure mode characterized by the yielding of fasteners with the formation of a plastic hinge is observed. Liu et al. [31,39] and Pozza et al. [40] studied the mechanical behaviour of angle brackets subjected to bi-directional vertical-tensile and horizontal-shear load direction. The tests revealed that angle brackets exhibit a lower peak load when exposed to bi-directional loads, emphasizing the need to properly consider the appropriate load configurations acting on angle brackets in the design process of CLT shear walls. Acler [17] and Tomasi et al. [41] conducted a comprehensive experimental investigation to evaluate the mechanical behaviour of five different types of angle brackets, focusing on the influence of geometry and the number of nails, under horizontal-shear load. Consistently with the findings from previous studies, it was observed that angle brackets peak load and stiffness is linearly proportional to the number of nails. An experimental study conducted by Fanti et al. [42] investigated the performance of three types of angle brackets, where either fully-threaded screws or washers were adopted to prevent the plastic deformation of the steel base plate under vertical-tensile load.

Several experimental campaigns were conducted also on second-generation of angle brackets, specifically develop to reach higher mechanical performances and prevent undesired failure modes typically observed in the first-generation of angle brackets. Masroor et al. [24,25] presented an experimental study on a single type of angle bracket for a wall-to-foundation joint configuration, adopting both full- and partial-nailing patterns under uni-directional vertical-tensile and horizontal-shear load directions. Both nailing patterns showed comparable mechanical responses under horizontal-shear load, with a failure mode that was primarily related to the nail yielding. Under vertical-tensile load, full-nailing angle brackets generally showed limited displacement capacity yet provided higher stiffness compared to partial-nailing pattern. Tests on partial-nailing angle brackets incorporated by a washer, however, showed noticeably increased of peak load, preventing premature brittle failure (e.g. anchor bolt pull-through) and allowing nails to contribute more effectively. Brown et al. [43] tested a single type of angle bracket in a wall-to-floor joint configuration. The research aimed to investigate the influence of the CLT outer layer orientation on the mechanical performance under horizontal-shear load. The findings revealed that the resistance was significantly affected by the orientation of the outer layer, as the nails primarily engaged this layer. Xiong et al. [44] conducted an experimental study on a single type of angle brackets used in a wall-to-floor joint configuration under uni-directional vertical-tensile and horizontal-shear load direction. Under vertical-tensile load, the angle brackets exhibited a ductile response, allowing a high energy dissipation evidenced by plasticization in the steel plate and yielding of the screws. Conversely, under horizontal-shear load, the failure was primarily controlled by screws yielding associated to a limited rotation of the steel plates. Both the studies by Brown et al. [43] and Xiong et al. [44] demonstrated that in some tests and under large displacements, local failures of wood can be observed.

In wall-to-floor joint configuration, D′Arenzo et al. [45,46] designed an innovative shear-tension angle bracket that significantly enhanced mechanical behaviour in both vertical-tensile and horizontal-shear load direction. This improvement was achieved through the combined use of nails and inclined fully-threaded screws embedded in the floor panel, as well as with thicker steel plates. The experimental tests demonstrated that this angle bracket reduced the possibility to engage the nail withdrawal from the floor panel ensuring significant values in terms of peak load and ductility. Casagrande et al. [47] conducted experimental tests on three types of angle brackets focusing on the evaluations of peak and yield load as well as displacements, stiffness, and ductility. A specific type of angle bracket including washer, used typically for wall-to-foundation joint configuration, demonstrated greater mechanical behaviour under vertical-tensile loads than those without washer, being potentially a valuable alternative to traditional hold-downs in terms of peak load. The findings by Masroor et al. [24,25] and Casagrande et al. [47] are confirmed by the study by Khai Tran et al. [28], which emphasizes that adding washer greatly improves the vertical-tensile mechanical behaviour of partial-fastening angle brackets. The use of washer prevents premature brittle failure modes related to the anchor bolt pull-through in wall-to-foundation joint configuration.

2.3. Structure of the annotated catalogue

The vast amount of information available from the experimental campaigns reported in the two previous sections has been systematically collected and organized into an annotated catalogue divided into three main parts: i) Product and test configuration; ii) Material and fasteners; and iii) Mechanical parameters. A detailed description of each section is reported in this section, using one of the tests conducted by Masroor et al. [24] on a hold-down as an example.

As reported in Table 1, the first parts, i.e. *Product and test configuration*, includes: i) Reference to the original study available in literature - *Ref.*; ii) The mechanical Anchor ID as detailed in Sections 3.1 and 4.1 for hold-downs and angle brackets, respectively; iii) Name of the traditional mechanical anchor; iv) Product certification document - *PCD*; v) Manufacturer; vi) Direction of the applied load, whether uni-directional vertical-tensile (*F1*), uni-directional horizontal-shear (*F2/3*), or bi-directional (*F1□F2/3*), see Fig. 2.a and Fig. 2.b; vii) Load type (i.e. monotonic or cyclic); viii) Standard used for the load protocol; and ix) Number of replicates for each test.

The second part deals with the *Materials and the fasteners* adopted in the tests for both wall-to-foundation and wall-to-floor joint configuration, see Fig. 2.a and see Fig. 2.b. As shown in Table 1, this part includes: i) Number of layers of the CLT wall panel; ii) Thickness of the CLT wall panel - t_{TOT}; iii) Layup of the CLT wall panel; iv) Wood species of CLT

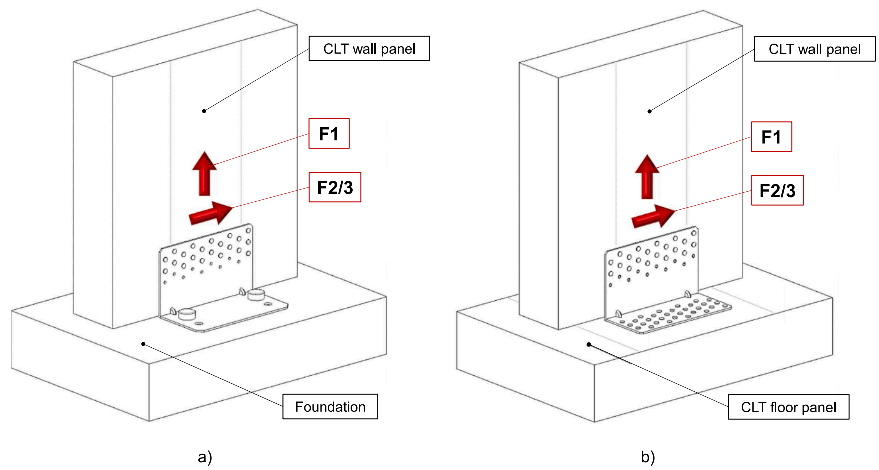


Fig. 2. Angle brackets in: a) wall-to-foundation and b) wall-to-floor joint configurations, illustrating vertical-tensile (F1) and horizontal-shear (F2/3) load directions.

Table 1
Example of the three parts characterizing the annotated catalogue.

PRODUCT AND TEST CONFIGURATION								
Ref.	**Anchor ID**	**Product**	**PCD**	**Manufacturer**	**Load direction**	**Load type**	**Load protocol**	**Number of replicates**
[24]	HD620-L	WHT620	ETA 11/0086	Rotho Blaas s.r.l.	F1	cyclic	ASTM E2126 [48]	3
MATERIALS AND FASTENERS								
CLT wall panel					**Fastener connected to CLT wall panel**			
Layers	**t_{TOT} [mm]**	**Layup**	**Wood species**	**ρ_m [kg/m^3]**	**Type of fastener**		**Number of fastener**	**Fastener pattern**
3	105	35-35-35	SPF Lumber	-	Ring shank nails Ø4×60mm		22	Partial
Floor panel or foundation					**Anchoring to floor panel or foundation**			
CLT panel	**Steel beam**	**Concrete support**		**Type of fastener**	**Number of fastener**		**Washer**	**Fastener pattern**
-	✔	-		Bolt Ø22	1		Yes	-
MECHANICAL PARAMETERS								
Standard procedure	**K_{el} [kN/mm]**	**v_y [mm]**	**F_y [kN]**	**F_{max} [kN]**	**v_u [mm]**	**F_u [kN]**	**μ [-]**	**Failure mode**
EN12512 [49]	9.243	8.55	78.90	90.56	21.42	72.60	2.51	Yielding of fasteners

The annotated catalogue is available in the supplementary material of this document as an open-access spreadsheet (.xlsx).

wall panel; v) Mean density of CLT panel - ρ_m; vi) Type of fastener connected to the CLT wall panel; vii) Number of fasteners; viii) Fastener pattern (e.g. full or partial); ix) Type of floor panel or foundation, whether CLT panel, Steel beam or Concrete support; Type of connection to the floor panel or to the foundation, including: x) Type of fastener; xi) Number of fasteners; xii) Use of washer; and xiii) Fastener pattern.

The third part is related to the *Mechanical parameters* obtained from each test and reported in the original study, as reported in Table 1, includes: i) The Standard procedure adopted in the original study to determine the mechanical properties from the experimental load-displacement curves; The value of: ii) Stiffness - K_{el}; iii) Yield displacement - v_y; iv) Yield load - F_y; v) Peak load - F_{max}; vi) Ultimate displacement - v_u; vii) Ultimate load - F_u; viii) Ductility - μ; and ix) the failure mode. It is noteworthy to mention that the mean values of mechanical properties are reported when two or more replicates were conducted in the original study.

3. Hold-downs

3.1. Geometrical and mechanical configurations

Hold-downs are mechanical anchors, typically located at the edges of CLT shear walls (Fig. 3), to prevent the overturning of the wall panels. Traditional hold-downs are three-dimensional nailing plates manufactured from cold-formed steel plates and typically consist of three components: a vertical plate, a base plate, and two lateral triangular plates. Such components are either welded on-to-another or formed through a cold-forming process that involves cutting and bending of the steel. Small diameter dowel-type fasteners are typically used (e.g. ring shank nails or STSs) to connect the hold-down to the CLT panels whereas a steel bolt is adopted to anchor the horizontal base plate to the foundation. A steel washer is commonly adopted in order to enhance the base plate's load distribution and prevent local deformation.

Traditional hold-downs are typically characterized by similar geometric configurations across different manufacturers. A summary of nine different types of traditional hold-downs available in the market is reported in Table 2 in terms of geometrical dimensions and number of holes n_v for five different manufactures, namely: Simpson Strong-Tie Int. Ltd. [50], Rotho Blaas s.r.l. [51], Adolf Würth GmbH & Co. [52], GH-Baubeschläge GmbH [53], and Soltech s.r.l. [54]. The same nomenclature (HD XXX-Z) adopted for the annotated catalogue is used in Table 2, where *XXX* indicates the height of the vertical plate of the hold-down in millimetres and *Z* refers to the base width (either type-"S" small in case of a $B < 65$ mm or type-"L" large in case of a $B \geq 80$ mm). Additionally, the column "AC" specifies whether, for certain hold-downs data from experimental tests are reported in the annotated catalogue (either "Y" yes or "N" no). Hold-downs marked with "N" are not

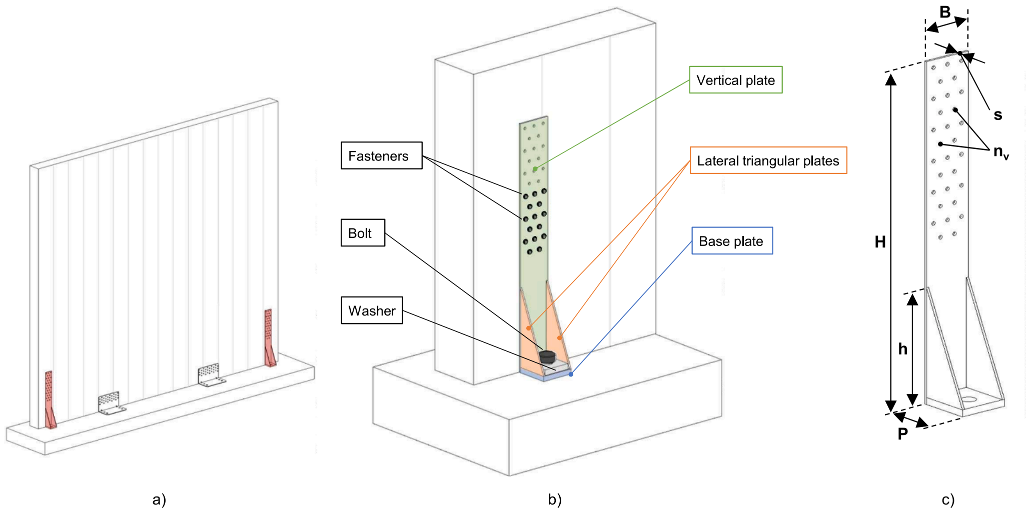


Fig. 3. Hold-downs: a) installation in a CLT shear wall, b) definition of structural components, and c) geometrical parameters

Table 2
Geometric parameters of traditional hold-downs from selected manufactures.

Hold-down ID	Manufacturer	H [mm]	B [mm]	P [mm]	h [mm]	s [mm]	n_v [-]	AC
HD309-S	[48]	309	64	62	170	2.8	18*	N
HD340-S	[49-52]	340	60	63	150	3	20	Y
HD403-S	[48]	403	64	62	170	2.8	18*–26	Y
HD440-S	[49-52]	440	60	63	150	3	30	Y
HD540-S	[49-52]	540	60	63	150	3	42–45	Y
HD569-S	[48]	569	64	62	170	2.8	32	Y
HD620-L	[49-52]	620	80	83	150	3	52–55	Y
HD790-L	[48]	790	90	60	170	3	41**	N
HD740-L	[49,51,52]	740	140	83	150	3	75	Y

*In the area defined by the two lateral triangular plates, additional holes are provided to limit bending in the vertical plate
**The vertical plate contains larger holes to allow for the insertion of additional fasteners

included in the annotated catalogue since experimental data are not available; however, such hold-downs are included in Table 2 as they contribute to the overview of the geometrical properties. As an example, a hold-down having a base plate width *B* equal to 60 mm and a vertical plate height *H* of 440 mm is represented with the nomenclature "HD440-S". It is noteworthy to mention that the same geometrical configuration may have been adopted from different manufacturers (e.g. HD440-S from manufacturers [50-53]).

As reported in Table 2, the thickness of the vertical plate typically ranges between 2.8 mm and 3.0 mm, the height of the lateral triangular plates varies from 150 mm to 170 mm while the diameter of the holes in the vertical plate ranges between 4.7 mm and 5.0 mm. The height of the vertical plates in hold-downs with a base plate width ranging from 60 mm to 64 mm (type-S) is between 309 mm and 569 mm and a number of holes between 18 and 45. Conversely, the height of the vertical plates and the number of holes is between 620 mm and 790 mm and between 52 and 75, respectively, for hold-downs with a wider base plate (i.e. ranging from 80 mm to 140 mm - type-L).

3.2. Mechanical behavior

The CLT wall uplift force (*F*) is transferred to the hold-down as a vertical shear force acting on the fasteners in a single-plane steel-to-timber connection. A deformation of the small-diameter dowel-type fasteners engaging at least one plastic hinge is typically observed. The shear force is transferred to the vertical steel plate as a tensile force (*T*) balanced by the tensile reaction force (*R*) in the anchor bolt. The geometrical eccentricity between the tensile force in the vertical steel plate and the reaction force commonly causes a rotation (θ) of the bottom part of the hold-down (i.e. base plate, triangular plates and bottom part of the vertical steel plate) and a consequent bending deformation of the vertical steel plate in proximity of the lowest row of fasteners as shown in Fig. 4.a.

The failure mode of a traditional hold-downs is strictly influenced by the fastener pattern. When a full-nailing pattern is employed (i.e. the number of fasteners is equal to the number of holes), a brittle failure of the vertical steel plate in proximity of the lowest row of fasteners caused by the combined tensile-bending stresses is typically observed. Conversely, a ductile failure governed by the yielding of fasteners is reached when partial-nailing patterns are used (i.e. the number of fasteners is significantly smaller than the number of holes). A failure mode characterized by a significant plastic deformation and punching of the steel plate may be also attained when no washer is installed, see Fig. 4.b.

The mechanical response of traditional hold-downs is characterized by an almost elastic behaviour for values of uplift smaller than 6–8 mm while the ultimate displacement is typically reached between 15 mm and 25 mm. The plastic phase is quite limited with values of ductility ranges from 1.5 to 2.5 and the hysteretic loops are characterized by a pronounced pinching effect during the load reversal. No significant differences are typically observed between the load-displacement curve obtained from a monotonic test and the backbone curve obtained from a cyclic test on the same hold-down. Fig. 5 shows a typical cyclic and monotonic load-displacement curve for a traditional hold-down, using as an example that obtained from test on a hold-down HD440-S characterized by a full-nailing pattern with 30 ring shank nails.

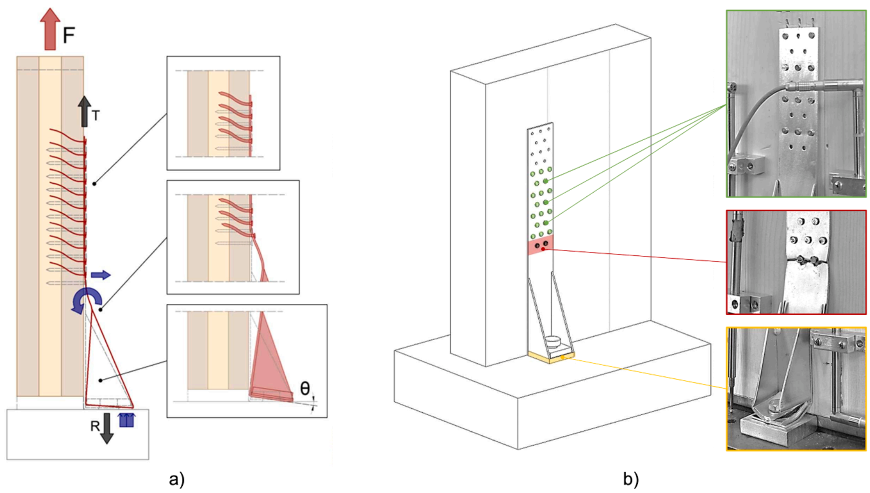


Fig. 4. Traditional hold-downs under vertical-tensile load: a) deformation mechanisms and b) potential failure modes.

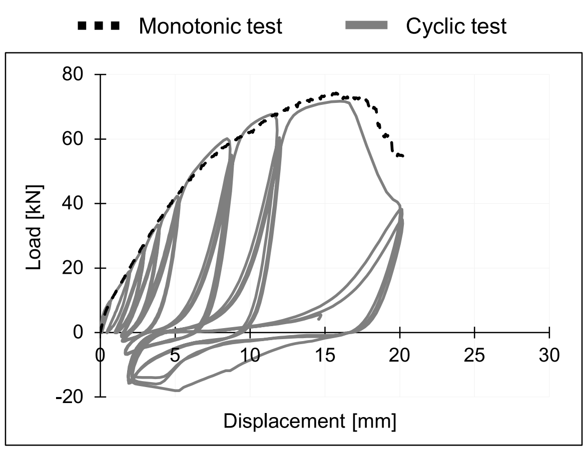


Fig. 5. Monotonic and cyclic load–displacement curves for the HD440-S hold-down with full nailing pattern, based on data from the Mechanical Testing Laboratory archive of the Institute of Bioeconomy, National Research Council of Italy (CNR-IBE).

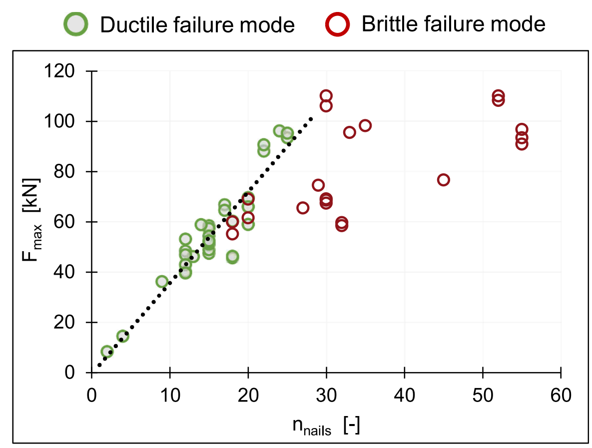


Fig. 6. Traditional hold-downs under vertical-tensile load: peak load vs number of nails.

3.3. Summary of results

This section presents an overview of the values of mechanical parameters reported in the annotated catalogue and obtained from experimental tests available in literature in terms of: i) peak load (F_{max}); ii) stiffness (K_{el}) iii) ductility (μ), and iv) ultimate displacement (v_u). The data from both monotonic and cyclic tests under vertical-tensile load were analysed in order to establish a relationship between such mechanical properties and the number of fasteners. For this purpose, the results obtained from hold-downs with Ø4⌀60 mm ring shank nails dimensions were examined.

An almost linear trend between the peak load (F_{max}) and the number of nails (n_{nails}) can be established for hold-downs that exhibit a ductile failure mode related to yielding of the nails, as shown in Fig. 6. The values of peak load are between 40 kN and 100 kN corresponding approximately to a number of nails between 10 and 25. Conversely, the number of nails does not have a significant influence on the peak load value when the failure in the vertical steel plate is reached. The value of peak load is related to the resistance of the net cross section of the vertical steel plate and ranges from 60 to 80 kN for smaller hold-down (type-S) and from 90 and 110 kN for larger hold-down (type-L). A brittle failure mode is typically observed for a number of nails approximately equal to or greater than 20 and 30 for type-S and type-L hold-downs, respectively, as shown in Fig. 6.

The analytical relationship between the peak load and number of nails for hold-downs with a ductile failure mode can be obtained through a linear interpolation as expressed by Eq. (1).

$$F_{\max} = 3.62 \bullet n_{nails} - 0.58 \quad [kN] \quad R^2 = 0.87 \tag{1}$$

It is noteworthy to observe that, despite the dimensions and the grade of the vertical steel plates are similar for hold-down types (i.e. type-S or type-L), a considerable variability of the values of peak load is obtained when the failure mode in the steel plate is reached (peak load is between 60 and 80 kN for type-S). Such variability may be attributed to the bending deformation of the vertical steel plate (caused by the rigid rotation of the bottom part of the hold-down) that promotes a concentration of axial and bending stresses in the area around the lowest row of nails. The nails act like a restraint to the rotation of the bottom part of the hold-down and, as a result, the location of the lowest row of nails may influence the failure mode of the steel plate (Fig. 7).

In Figs. 8 and 9 the values of stiffness (K_{el}) and ductility (μ) are reported as function of the number of nails (n_{nails}), respectively.

Fig. 8 shows a large scatter of values of stiffness when the number of nails is less than 20, as it ranges between 4 kN/mm and 10 kN/mm. Conversely, the values of stiffness are almost proportional linear to the number of nails when they are larger than 20. Through a linear interpolation, an analytical relationship between stiffness and the number of nails can be expressed according to Eq. 2 for a number of nails equal to or greater than 20.

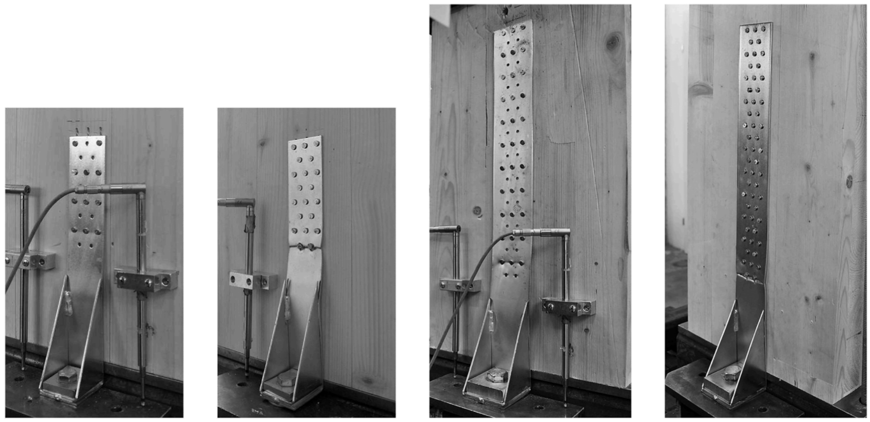


Fig. 7. Traditional hold-downs under vertical-tensile load at the end of the test. Hold-down type-S: a) 12 nails (partial nailing-pattern) and; b) 20 nails (full-nailing pattern). Hold-down type-L: c) 33 nails (partial nailing-pattern) and; b) 55 nails (full-nailing pattern).

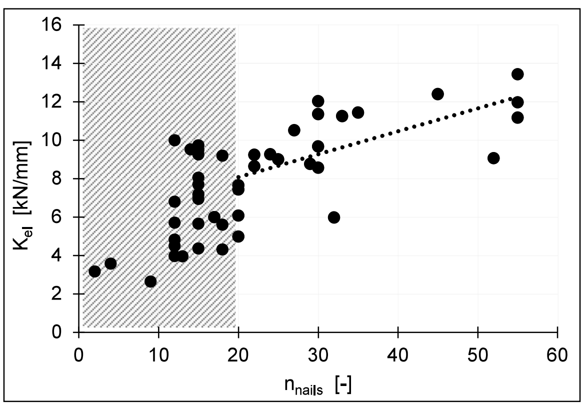


Fig. 8. Traditional hold-downs under vertical-tensile load: stiffness vs number of nails.

$$K_{el} = 0.12 \bullet n_{nails} + 5.69 \quad \left[\frac{kN}{mm}\right] \; n_{nails} \geq 20 \quad R^2 = 0.42 \tag{2}$$

Similarly, Fig. 9, reports a significant scatter in the ductility values for specimens with fewer than 20 nails, with results ranging from 2.5 to 5.0. However, when the number of nails is larger than 20 an almost constant trend of ductility, between 1.5 and 2.5, is evidenced.

A generally decreasing trend of the ultimate displacement (v_u) as function of the number of nails (n_{nails}) is observed in Fig. 10. For hold-downs with fewer than approximately 25 nails, the ultimate displacement shows a wide scatter, with values ranging between about 15 mm and 30 mm. Conversely, for specimens with more than approximately 25 nails, the values of v_u are confined within a narrower range, typically between 15 mm and 20 mm.

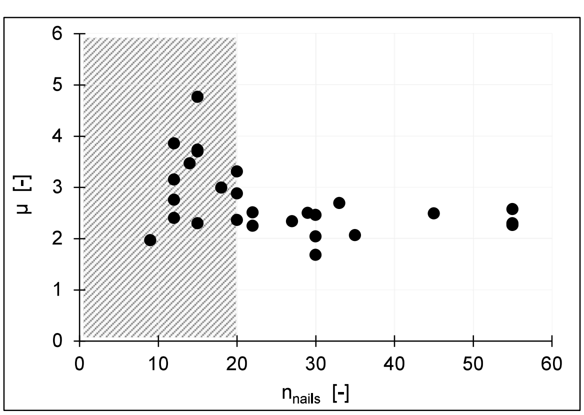


Fig. 9. Traditional hold-downs under vertical-tensile load: ductility vs number of nails.

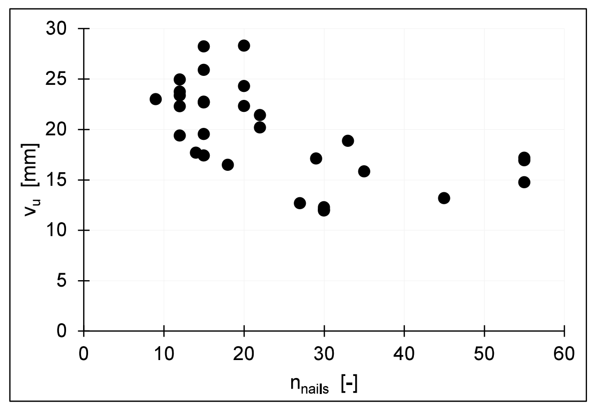


Fig. 10. Traditional hold-downs under vertical-tensile load: ultimate displacement vs number of nails.

4. Angle brackets

4.1. Geometrical and mechanical configurations

Angle brackets (Fig. 11) are three-dimensional mechanical anchors installed at the base of CLT walls primarily to prevent the sliding of shear walls. Fabricated from cold-formed steel, angle brackets consist of a vertical plate and a horizontal base plate. The vertical plate is connected to the CLT panels through small-diameter dowel-type fasteners such as ring shank nails or STSs. In wall-to-floor joint configurations (Fig. 11.a), the base plate is connected to the floor panels by the same type of fasteners adopted for the vertical steel plate. Conversely, in wall-to-foundation configuration joints (Fig. 11.b), mechanical or chemical anchors are typically adopted to connect the base plate to concrete foundation.

Differently from hold-downs, a substantial variability in geometrical dimensions exists in the market of traditional angle brackets primarily due to the significant evolution in the design and performance of such products over the past two decades. The first-generation of angle brackets was originally developed for low-rise light-frame timber structures and their dimensions were relatively compact and optimized

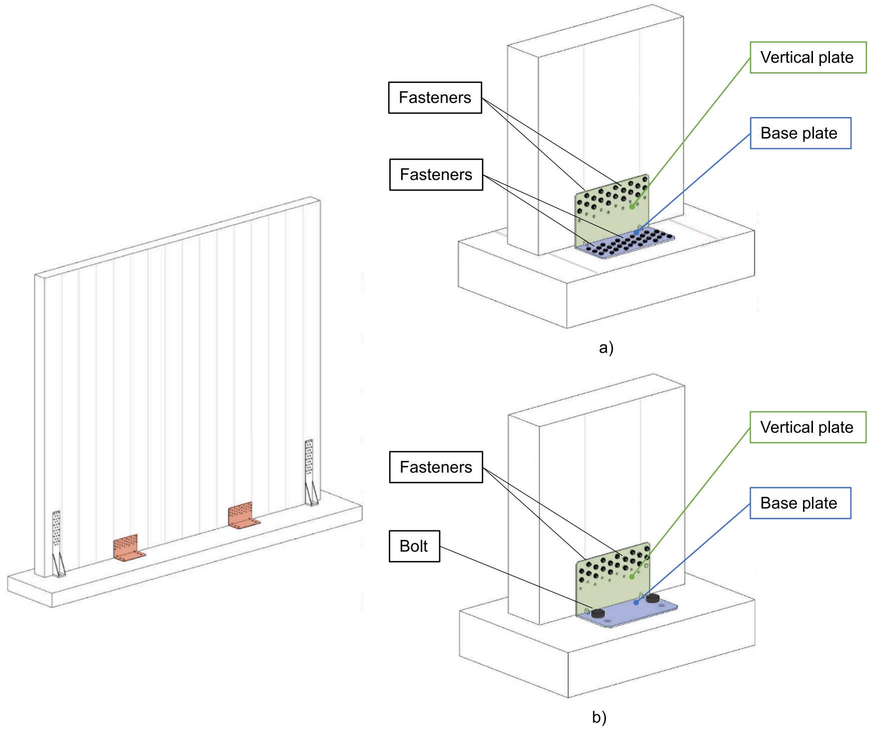


Fig. 11. Angle brackets installed in a CLT shear wall (left), and definition of the structural components (right): a) wall-to-floor and b) wall-to-foundation joint configuration.

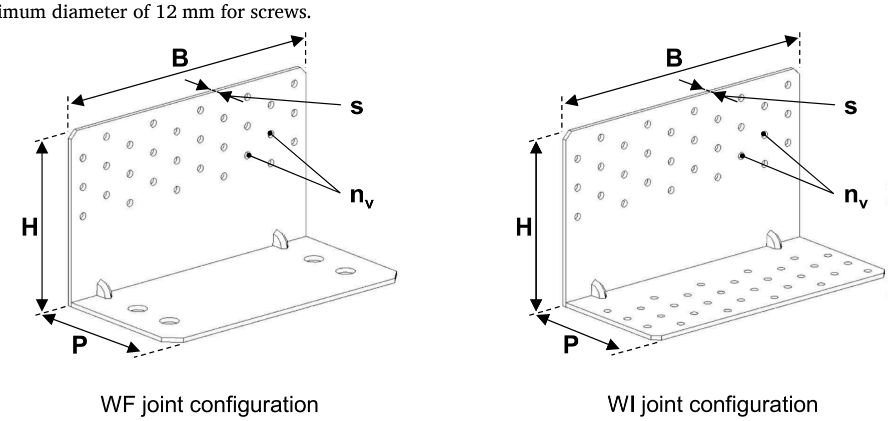
for limited values of shear forces. The second-generation of angle brackets was conversely designed to specifically withstand the forces acting on CLT shear walls increasing the geometrical dimensions of the steel plates as well as the number of fasteners.

A list of eleven different types of traditional angle brackets available in the market are reported in Table 3 according to four different manufactures, namely: Simpson Strong-Tie Int. Ltd. [55], Rotho Blaas s.r.l. [56,57], BB Stanz- und Umformtechnik [58] and GH-Baubeschläge

Table 3
Geometric parameters of traditional angle brackets from selected manufactures.

Generation	Angle brackets ID	Manufacturer	Joint configuration	H [mm]	B [mm]	P [mm]	s [mm]	n_v* [-]	AC
I°	AB55-S	[54]	WF+WI	70	55	70	2	6+1	N
	AB65-S	[58,59]	WF+WI	90	65	90	2.5	14+1	Y
	AB90-S	[54,58,59]	WF+WI	100–105	90	100–105	3	14+4	Y
	AB104-S	[57]	WF+WI	100	104	78	2.5	25	N
	AB116-S	[54]	WF+WI	90	116	48	3	18+3	Y
II°	AB200-L	[55]	WF	71–120	200	103	3	30+0	Y
			WI	71	200	71	3	30+0	Y
	AB240-L	[55]	WF	120	240	123	3	36+0	Y
			WI	120	240	83–93	3–4	36+0	Y
	AB255-L	[54,56]	WF+WI	120	255	100	3	52+2	N
	AB260-L	[59]	WF+WI	120	260	82	3	59	N

*The notation X + Y indicates the presence of different hole sizes. X represents the number of holes with a diameter of 5 mm for small fasteners, while Y indicates the number of holes with a minimum diameter of 12 mm for screws.



GmbH [59,60]. The same nomenclature (AB-XXX) used in the annotated catalogue is reported in Table 3, where *XXX* represents the width of the vertical plate in millimetres. The geometrical dimensions, the number of holes in the vertical plate, the joint configuration (i.e. wall-to-foundation (WF) and wall-to-floor (WI)) and the availability of experimental data in the annotated catalogued AC (either "Y" yes or "N" no) are reported. Angle brackets marked with "N" are included even without experimental data in the annotated catalogue, as they contribute to the geometrical overview that follows. It is noteworthy that the same geometrical configuration may have been adopted from different manufacturers (e.g. AB90-S from manufacturers [52,54 and 55]) and for two joint configurations (e.g. WF+WI). Either full- or partial-nailing patterns can be adopted also for angle brackets.

As reported in Table 3, the height (*H*) and depth (*P*) of the angle brackets generally range between 70 mm and 120 mm, while the diameter of the holes in the vertical plate varies between 4.7 mm and 5.0 mm. A clear distinction between the two generations of angle brackets can be established referring to the geometrical dimensions and number of holes n_v in the vertical plate. The first-generation (I°) of angle brackets are characterized by a width of the vertical plate (*B*) not greater than 116 mm and a maximum number of holes equal to 18. Conversely, for the second-generation (II°) of angle brackets the width of the base plates range from 200 mm to 240 mm and the number of holes in the vertical plate is not less than 30.

4.2. Mechanical behavior

The horizontal-shear force (*V*) is transferred through the small-diameter dowel-type fasteners from the CLT panel to the vertical steel plate of the angle bracket which in turn transmits the shear force from the steel plate to the anchoring bolts (or fasteners for a wall-to-floor joint configuration). Assuming that *V* is located in the geometrical centre of the group of fasteners of the vertical steel plate, two geometric eccentricities, namely e_z and e_y, can be identified between *V* and the horizontal forces (F_x) acting on the anchoring bolts at the base plate, as shown in Fig. 12. These eccentricities generate both a bending and a torsional moment which induces additional uplift (F_z) and out-of-plane shear (F_y) forces on the anchoring bolts (or fasteners for a wall-to-floor joint configuration).

Fig. 13 illustrates the typical deformation mechanisms of angle brackets for both wall-to-foundation and wall-to-floor joint configurations. For first-generation angle brackets, the failure mode, in both joint configurations, is predominantly governed by significant deformation of the steel plates. The excessive deformation, caused by bending and torsional moments on the base plates, has a critical impact on the failure mechanism. In wall-to-floor joint configurations, fasteners are pulled out from the base plate (Fig. 13.c). Conversely, in the wall-to-foundation joint configurations, localized bearing due to bolt anchoring in the base plate were observed (Fig. 13.c). In contrast, for the second-generation angle brackets, the increased size of the steel plates enhances the bending and torsional stiffness of the angle brackets, leading to a failure mode primarily governed by yielding of the fasteners in the vertical steel plates (Fig. 13.f).

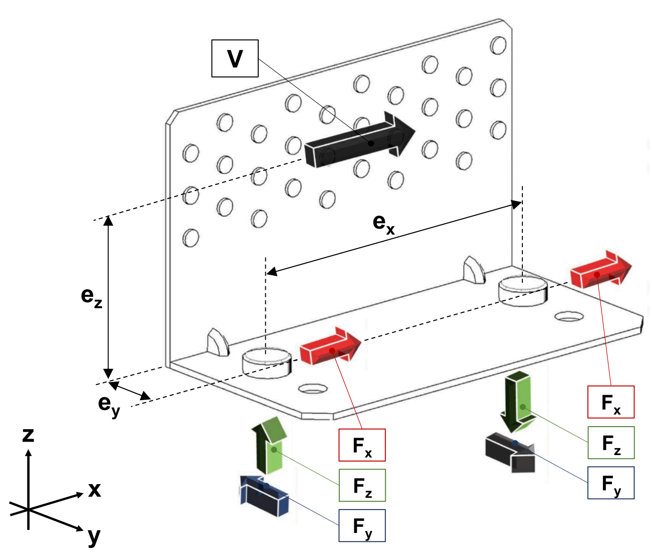


Fig. 12. Traditional angle bracket in a wall-to-foundation joint configuration under horizontal-shear load: typical force distribution on anchor bolts.

The introduction of second-generation angle brackets showed a considerable increase in stiffness and load capacity, preventing the substantial deformations of the vertical and base steel plates typically observed in the first-generation angle brackets. Figs. 14 and 15 present, as an example, typical monotonic and cyclic load-displacement curves obtained from two angle brackets in a wall-to-foundation joint configuration, representing the two different generations: AB116-S with partial-nailing pattern of 11 ring shank nails and AB240-L with full-nailing pattern of 36 ring shank nails.

A comparison of the cyclic load-displacement curves (Fig. 14 and Fig. 15) demonstrates that second-generation angle brackets exhibit a more pronounced elastic phase compared to their first-generation counterparts. Specifically, the average yield displacement increases from approximately 7.0 mm (related average yield load of 21 kN) for first-generation angle brackets to approximately 8.5 mm (related average yield load of 54 kN) for second-generation angle brackets. However, the ultimate displacement observed in second-generation angle brackets is generally lower than that of first-generation, which typically exhibit a limited plastic phase. The ductility values for second-generation angle brackets range from 1.7 to 4.5, whereas first-generation angle brackets show a wider range, from 2.0 to 10.0. It is noteworthy to mention that in many studies, the assessment of ultimate displacement has not adequately accounted for the influence of low-cycle degradation. This phenomenon plays a critical role in defining the ductility of angle brackets, particularly when significant ultimate displacements are observed during cyclic tests. The hysteretic behaviour of both first- and second-generations is characterized by pronounced pinching phenomena, attributed to localized slips and material deformation during load reversals. No significant differences are typically observed between the load-displacement curve obtained from a monotonic test and the backbone curve derived from a cyclic test for both first- and second-generations of angle brackets.

4.3. Summary of results

This section provides an overview of the key mechanical parameters (i.e. peak load - F_{max}, stiffness - K_{el}, ductility – μ, and ultimate displacement – v_u) reported in the annotated catalogue and obtained from experimental tests available in literature for traditional angle brackets subjected to horizontal-shear load. The data from both monotonic and cyclic tests were analysed in order to establish a relationship between such mechanical properties and the number of fasteners. For this study, results obtained using ring shank nails (Ø4÷60 mm) as fasteners in CLT wall panels were considered. Additionally, data from both wall-to-foundation and wall-to-floor joint configurations were included in the analysis. It is noteworthy to mention that the number of data collected on the first-generation of angle brackets is much larger than that on the second-generation.

Fig. 16 demonstrates that data trends for both first- and second-generation of angle brackets is well represented by a linear relationship between the peak load (F_{max}) and the number of nails (n_{nails}). For the first-generation angle brackets, peak load ranges between 10 kN and 50 kN corresponding approximately 8–18 nails. Conversely, the second-generation angle brackets exhibit significantly higher peak load due to their wider steel plates and the ability to accommodate a greater number

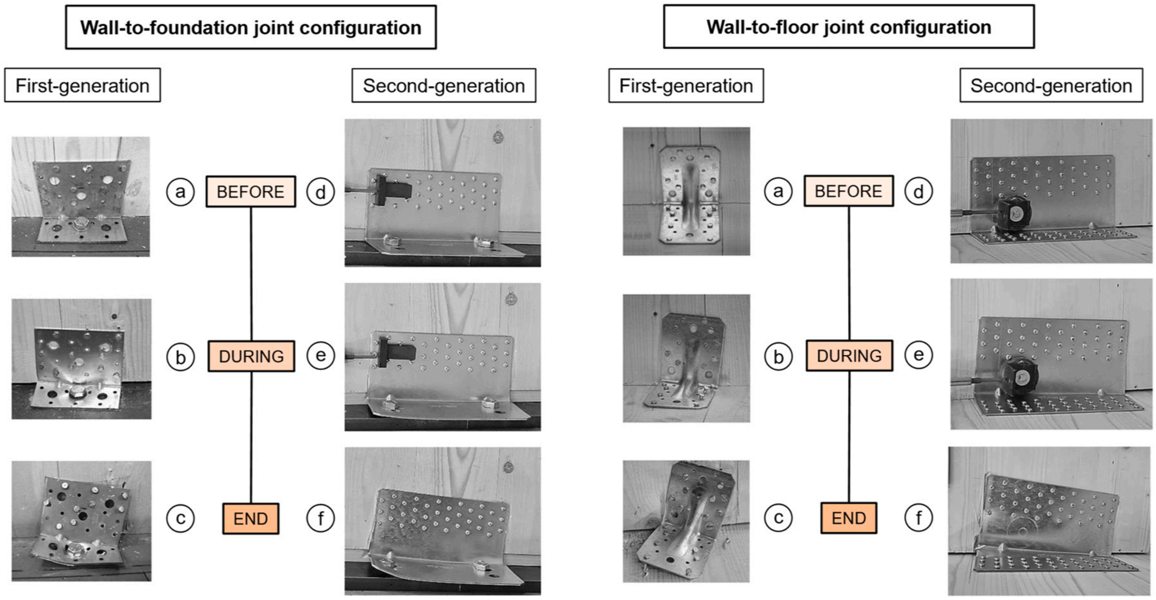


Fig. 13. Traditional angle bracket in a wall-to-foundation and wall-to-floor joint configuration under horizontal-shear load: potential progression of failure modes observed during laboratory testing.

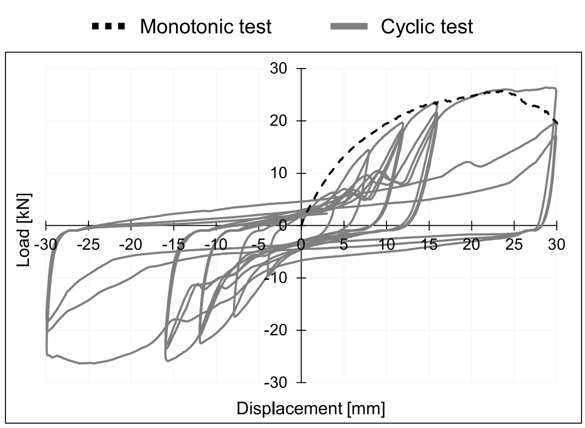


Fig. 14. Monotonic and cyclic load–displacement curves for the AB116-S angle bracket with partial nailing pattern, based on data from the Mechanical Testing Laboratory archive of the Institute of Bioeconomy, National Research Council of Italy (CNR-IBE).

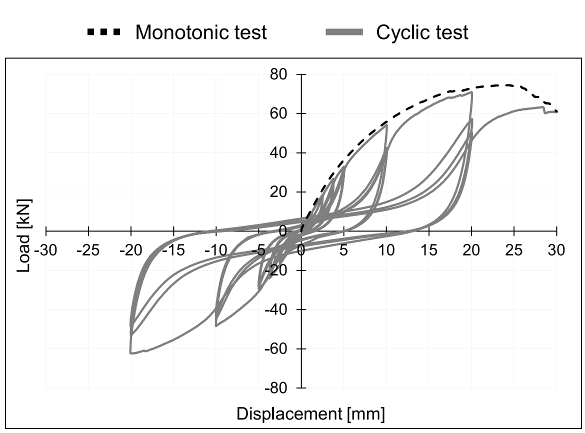


Fig. 15. Monotonic and cyclic load–displacement curves for the AB240-L angle bracket with full nailing pattern, based on data from the Mechanical Testing Laboratory archive of the Department of Civil Engineering, University of Bologna, Italy.

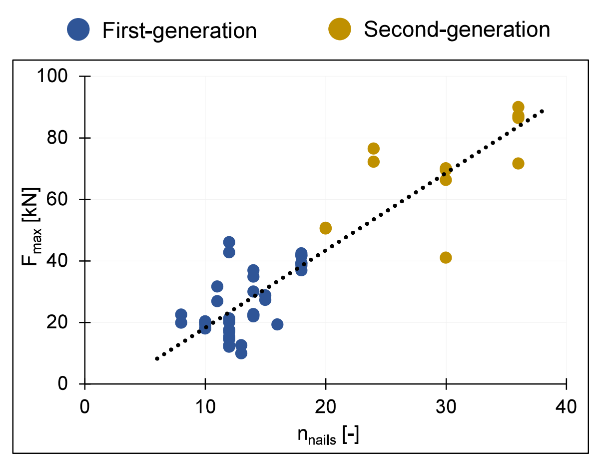


Fig. 16. Traditional angle brackets under horizontal-shear load: peak load vs number of nails.

of nails. Specifically, the values of peak load ranges between 40 kN and 90 kN for a number of nails approximately between 20 and 36.

An analytical relationship can be established between the peak load and number of nails for angle brackets through a linear interpolation as expressed by Eq. (3).

$$F_{\max} = 2.52 \bullet n_{nails} - 6.87 \quad [kN] \quad n_{nails} \geq 8 \quad R^2 = 0.82 \tag{3}$$

Figs. 17 and 18 present the stiffness (K_{el}) and ductility (μ) values as a function of the number of nails (n_{nails}), respectively. Overall, an increasing trend in stiffness with respect to the number of nails can be observed across all collected data for angle brackets, regardless of product generation. Although first- and second-generation angle brackets show different values of stiffness, their experimental data exhibit a consistent global trend when considered together.

Through a linear interpolation, an analytical relationship between stiffness and the number of nails can be expressed for the entire set of angle brackets, as reported in Eq. 4.

$$K_{el} = 0.30 \bullet n_{nails} - 1.52 \quad \left[\frac{kN}{mm}\right] \; 8 \leq n_{nails} \leq 36 \quad R^2 = 0.70 \tag{4}$$

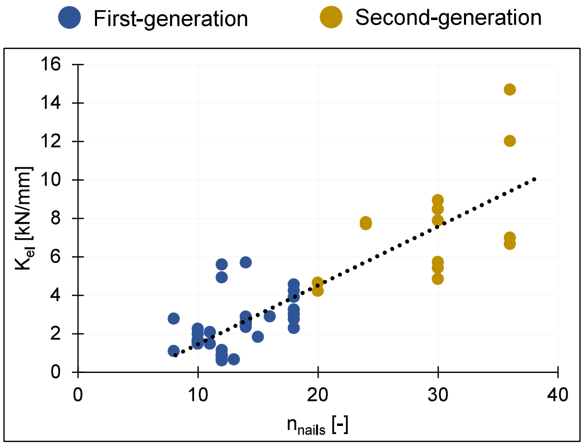


Fig. 17. Traditional angle brackets under horizontal-shear load: stiffness vs number of nails.

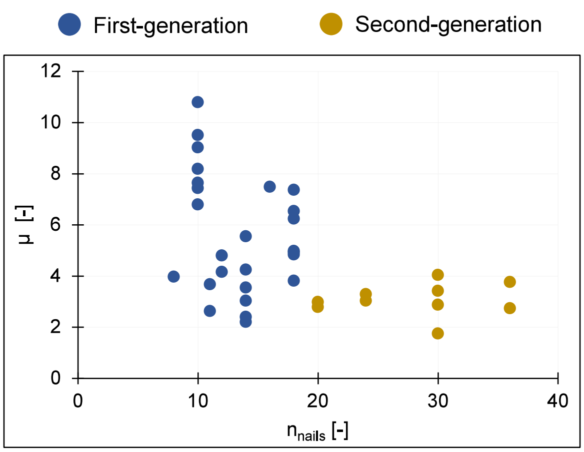


Fig. 18. Traditional angle brackets under horizontal-shear load: ductility vs number of nails.

Regarding ductility, first-generation angle brackets exhibit a considerable scatter in values, ranging between 2.0 and 11.0. Conversely, second-generation angle brackets show a more consistent distribution, with ductility values ranging between 2.0 and 4.0, as observed in Fig. 18.

The ultimate displacement does not exhibit a clear dependence on the number of nails, see Fig. 19. With the exception of few cases, the values of v_u for angle brackets with more than 10 nails are between 20 mm and 35 mm.

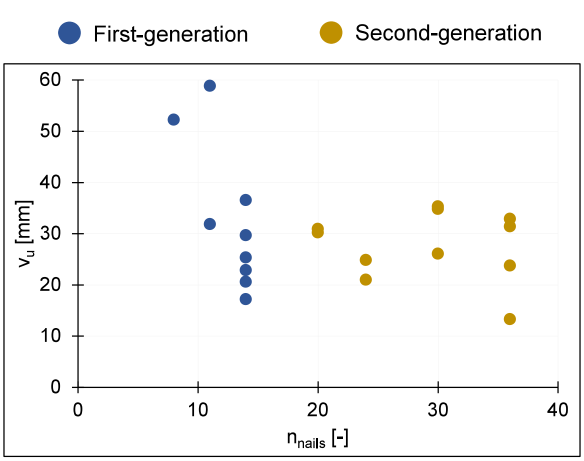


Fig. 19. Traditional angle brackets under horizontal-shear load: ultimate displacement vs number of nails.

5. Conclusions

The present study provides a comprehensive review of the seismic performances of traditional hold-downs and angle brackets in CLT structures. 39 experimental campaigns conducted worldwide were analyses to highlight differences and similarities in their mechanical performance, considering different set-up configurations, load protocols, fastener type, number and pattern, as well as the use of different wood species. The experimental findings and the mechanical properties are summarized and into an annotated catalogue.

The main outcomes from the analysis of the geometrical and the mechanical properties of traditional hold-downs can be summarized as follows:

- The failure mode is significantly influenced by the nailing pattern. A full-nailing pattern typically promotes a brittle failure of the vertical steel plate near the lowest row of fasteners while ductile failure primarily governed by fasteners yielding is observed when partial-nailing patterns are adopted.
- The deformation mechanism of hold-downs is typically characterized by a rotation at the base which causes a significant bending deformation in the vertical steel plate near the lowest row of fasteners. Such deformation plays a crucial role in the potential engagement of brittle failure modes.
- Experimental values of peak load generally range between 40 kN and 100 kN for ductile nail yielding failure mode corresponding to a number of nails from approximately 10–25. When brittle failure modes occur, the peak loads are between 60 kN and 110 kN depending on hold-down's base size.
- A linear relationship between the number of nails and peak load can be established for hold-downs with a ductile failure mode.
- For a number of nails not lower than 20, the stiffness is almost linear proportional to the number of nails and the ductility ranges between 1.5 and 2.5.
- The ultimate displacement decreases with increasing number of nails, ranging from approximately 15–30 mm for hold-downs with fewer than 25 nails, and from approximately 15–20 mm for hold-downs with a higher number of nails.

For traditional angle brackets the main outcomes can be summarized as follows:

- Two generations of angle brackets have been identified: a first-generation of angle brackets originally designed for light-frame timber structures and a second-generation of angle brackets specifically developed for CLT structures. An increase of dimensions and mechanical performance has been observed from the first- to the second- generation.
- In the first-generation of angle brackets a significant deformation of the steel plates as well as the fastener pull-out at the base plate were observed. The increase of size of the steel plates in the second-generation of angle brackets reduced the steel plate deformation primarily engaging ductile failure modes characterized by the yielding of fasteners.
- The second-generation of angle brackets exhibits higher mechanical performance than the first-generation. The values of peak loads and stiffness range from 40 to 90 kN and between 5 and 14 kN/mm for the second-generation of angle brackets, respectively. Values from 10 to 50 kN and 2–6 kN/mm were conversely observed for the first-generation angle brackets. The values of ductility in the first-generation range from 2.0 to 11.0, whereas the second-generation is characterized by a less spread range of values, namely from 2.0 to 4.5.
- A linear relationship between the number of nails and mechanical parameters (i.e. peak load and stiffness) can be established for angle brackets.

- The ultimate displacement for angle brackets does not show a clear dependence on the number of nails and generally remains within a range between about 20 and 35 mm when number of nails between is approximately between 10 and 36.

CRediT authorship contribution statement

Daniele Casagrande: Writing – review & editing, Validation, Resources, Methodology, Conceptualization. **Valentino Nicolussi:** Writing – review & editing, Writing – original draft, Validation, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **Andrea Polastri:** Writing – review & editing, Validation, Resources, Methodology. **Luca Pozza:** Writing – review & editing, Validation, Resources, Methodology.

Declaration of Competing Interest

Valentino Nicolussi declares that his PhD scholarship is co-funded by Rotho Blaas s.r.l. However, the company had no role in the design, execution, interpretation, or writing of this study. The authors declare no other competing interests.

Acknowledgements

The authors Valentino Nicolussi and Daniele Casagrande acknowledge the COST Action (CA20139) - *Holistic design of taller timber buildings (HELEN)* for supporting discussion within Working Group 3 (*Accidental Loads*) regarding the use of mechanical anchors in taller timber buildings.

The author Daniele Casagrande also acknowledges the Italian Ministry of Universities and Research 378 (MUR), in the framework of the project DICAM-EXC (Departments of Excellence 2023–2027, grant L232/2016).

Funded by the European Union - Next Generation EU, Mission 4, Component 2, Investment 3.3 (Ministerial Decree 352/2022) CUP J33C22001470009.

Appendix A. Supporting information

Supplementary data associated with this article can be found in the online version at doi:10.1016/j.istruc.2026.111622.

Data Availability

The annotated catalogue generated during the current study is available as open-access supplementary material in the form of an Excel spreadsheet [Annotated Catalogue.xlsx].

References

[1] Tannert T, Follesa M, Fragiacomo M, González P, Isoda H, Moroder D, Xiong H, Van De Lindt J. Seismic design of cross-laminated timber buildings. Wood Fiber Sci 2018;50:3–26. https://doi.org/10.22382/WFS-2018-037.
[2] Brandner R, Flatscher G, Ringhofer A, Schickhofer G, Thiel A. Cross laminated timber (CLT): overview and development. Eur J Wood Prod 2016;74:331–51. https://doi.org/10.1007/s00107-015-0999-5.
[3] Sandoli A, D'Ambra C, Ceraldi C, Calderoni B, Prota A. Sustainable cross-laminated timber structures in a seismic area: overview and future trends. Appl Sci 2021;11 (5):1–24. https://doi.org/10.3390/app11052078.
[4] Chen Z, Popovski M. Mechanics-based analytical models for balloon-type cross-laminated timber (CLT) shear walls under lateral loads. Eng Struct 2020;208: 109916. https://doi.org/10.1016/j.engstruct.2019.109916.
[5] Izzi M, Casagrande D, Bezzi S, Pasca D, Follesa M, Tomasi R. Seismic behaviour of cross-laminated timber structures: a state-of-the-art review. Eng Struct 2018;170: 42–52. https://doi.org/10.1016/j.engstruct.2018.05.060.
[6] Ceccotti A. New technologies for construction of medium-rise buildings in seismic regions: The XLAM case. Struct Eng Int J Int Assoc Bridge Struct Eng (IABSE) 2008; 18(2):156–65. https://doi.org/10.2749/101686608784218680.
[7] Ceccotti A, Sandhaas C, Okabe M, Yasumura M, Minowa C, Kawai N. SOFIE project - 3D shaking table test on a seven-storey full-scale cross-laminated timber building. Earthq Eng Struct Dyn 2013;42(13):2003–21. https://doi.org/10.1002/eqe.2309.
[8] Popovski M., Gavric I., Schneider J., (2014) Performance of two-storey CLT house subjected to lateral load. World Conference on Timber Engineering, WCTE 2014, Quebec City, Canada. http://dx.doi.org/10.13140/RG.2.1.3582.9280.
[9] Matos F, Branco J, Rocha P, Demschner T, Lourenço P. Quasi-static tests on a two-story CLT building. Eng Struct 2019;201. https://doi.org/10.1016/j.engstruct.2019.109806.
[10] Van de Lindt J, Furley J, Amini M, Pei S, Tamagnone G, Barbosa A, Rammer D, Line P, Fragiacomo M, Popovski M. Experimental seismic behavior of a two-story CLT platform building. Eng Struct 2019;183:408–22. https://doi.org/10.1016/j.engstruct.2018.12.079.
[11] Zhang X, Isoda H, Sumida K, Araki Y, Nakashima S, Nakagawa T, Akiyama N. Seismic performance of three-story cross-laminated timber structures in Japan. J Struct Eng 2021;147(2). https://doi.org/10.1061/(ASCE)ST.1943-541X.0002897.
[12] Hristovski V, Dujic B, Stojmanovska M, Mircevska V. Full-scale shaking-table tests of XLam panel systems and numerical verification: specimen 1. J Struct Eng 2012; 139(11):2010–8. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000754.
[13] Hristovski V, Mircevska V, Dujic B, Garevski M. Comparative dynamic investigation of cross-laminated wooden panel systems: shaking-table tests and analysis. Adv Struct Eng 2018;21(10):1421–36. https://doi.org/10.1177/1369433217749766.
[14] Sustersic I, Fragiacomo M, Dujic B. Seismic analysis of cross-laminated multistory timber buildings using code-prescribed methods: influence of panel size, connection ductility, and schematization. J Struct Eng 2016;142(4). https://doi.org/10.1061/(ASCE)ST.1943-541X.0001344.
[15] Chen Z., (2025) A Web-Based Database for Supporting the Design, Modelling, and Analysis of Mass Timber Structures (1st Year) (with Research Snapshot). FPInnovations Report. Available at: ⟨https://www.woodresearchlibrary.com/en/permalink/catalogue4290⟩.
[16] Heidari A, Dang E, Rezaei Rad A. Design database development and comparative performance assessment of connections in cross-laminated timber construction. J Build Eng 2025;114:114278. https://doi.org/10.1016/j.jobe.2025.114278.
[17] Acler E. Indagine sperimentale e numerica del comportamento di diverse tipologie di sistemi di connessione impiegate nella tecnica delle costruzioni a pannello Xlam. Italy: University of Trento; 2012.
[18] Flatscher G, Bratulic K, Schickhofer G. Experimental tests on cross-laminated timber joints and walls. Proceedings of the Institution of Civil Engineers - Structures and Buildings, 168; 2015. p. 868–77. https://doi.org/10.1680/stbu.13.00085.
[19] Flatscher G. Evaluation and approximation of timber connection properties for displacement-based analysis of CLT wall systems. Monogr Ser TU Graz Timber Eng & Technol 2017;6.
[20] Gavric I, Fragiacomo M, Ceccotti A. Cyclic behaviour of typical metal connectors for cross-laminated (CLT) structures. Mater Struct/Mater Et Constr 2015;48(6): 1841–57. https://doi.org/10.1617/s11527-014-0278-7.
[21] Gavric I. Seismic Behaviour of Cross-Laminated Timber Buildings. Italy: University of Trieste; 2012.
[22] Hummel J. Displacement-based seismic design for multi-storey cross laminated timber buildings. Germany: University of Kassel; 2017.
[23] Polastri A., Casagrande D., Pinna M., Magnago D., (2019) Tests for the determination of mechanical properties of hold-down anchors WHT340, WHT440, WHT540 and WHT620 nailed to CLT panels. TECHNICAL REPORT, San Michele all'Adige, 03/12/2019.
[24] Masroor M, Doudak G, Casagrande D. Experimental investigation of dissipative connections in cross-laminated timber shearwalls. Constr Build Mater 2024;413: 134828. https://doi.org/10.1016/j.conbuildmat.2023.134828.
[25] Masroor M. Behaviour and design considerations for Multi-panel CLT shearwalls. Canada: University of Ottawa; 2023.
[26] Dong W, Li M, Ottenhaus L, Lim H. Ductility and overstrength of nailed CLT hold-down connections. Eng Struct 2020;215:110667. https://doi.org/10.1016/j.engstruct.2020.110667.
[27] Benedetti F, Rosales V, Opazo-Vega A, Norambuena-Contreras J, Jara-Cisterna A. Experimental and numerical evaluation of hold-down connections on radiata pine Cross-Laminated-Timber shear walls: a case study in Chile. Eur J Wood Wood Prod 2019;77(1):79–92. https://doi.org/10.1007/s00107-018-1365-1.
[28] Khai Tran D, Young Jeong G. Tension resistance properties of hold-down and angle-bracket connections on cross-laminated timber (CLT). Structures 2023;56: 104841. https://doi.org/10.1016/j.istruc.2023.07.031.
[29] Ferracuti B., Massari M., Pozza L., Savoia M., (2017) Experimental cyclic tests on CLT hold-down connections subjected to a combination of shear and tension. World Conference on Earthquake Engineering, WCEE 2017, Santiago de Chile, Chile. ⟨http://hdl.handle.net/11585/588994⟩.
[30] Pozza L, Ferracuti B, Massari M. Axial – Shear interaction on CLT hold-down connections – Experimental investigation. Eng Struct 2018;160:95–110. https://doi.org/10.1016/j.engstruct.2018.01.021.
[31] Liu J., Lam F., (2016) Experimental test of cross laminated timber connections under bi-directional loading. World Conference on Timber Engineering, WCTE 2016, Vienna, Austria. ⟨http://hdl.handle.net/20.500.12708/172⟩.
[32] Liu J, Lam F. Experimental test of coupling effect on CLT hold-down connections. Eng Struct 2019;586-602:178. https://doi.org/10.1016/j.engstruct.2018.10.06.
[33] Shen Y, Schneider J, Tesfamariam S, Stiemer S, Mu Z. Hysteresis behavior of bracket connection in cross-laminated-timber shear walls. Constr Build Mater 2013;48:980–91. https://doi.org/10.1016/j.conbuildmat.2013.07.050.
[34] Shen Y, Schneider J, Stiemer S, Ren X. Failure Modes and Mechanical Properties of Bracket Anchor Connections for Cross-Laminated-Timber. MATEC Web Conf 2019; 275:01011. https://doi.org/10.1051/matecconf/201927501011.

[35] Shen Y, Schneider J, Tesfamariam S, Stiemer S, Chen Z. Cyclic behavior of bracket connections for cross-laminated timber (CLT): Assessment and comparison of experimental and numerical models studies. J Build Eng 2021;39:102197. https://doi.org/10.1016/j.jobe.2021.102197.

[36] Schneider J, Karacabeyli E, Popovski M, Stiemer S, Tesfamariam S. Damage Assessment of Connections Used in Cross-Laminated Timber Subject to Cyclic Loads. J Perform Constr Facil 2014;28(6):A4014008. https://doi.org/10.1061/(ASCE)CF.1943-5509.0000528.

[37] Rezvani S, Zhou L, Ni C. Experimental evaluation of angle bracket connections in CLT structures under in- and out-of-plane lateral loading. Eng Struct 2021;244:112787. https://doi.org/10.1016/j.engstruct.2021.112787.

[38] Mahdavifar V, Barbosa A, Sinha A, Muszynski L, Gupta R, Pryor S. Hysteretic response of metal connections on hybrid cross-laminated timber panels. J Struct Eng 2019;145(1). https://doi.org/10.1061/(ASCE)ST.1943-541X.0002222.

[39] Liu J, Lam F. Experimental test of coupling effect on CLT angle bracket connections. Eng Struct 2018;171:862–73. https://doi.org/10.1016/j.engstruct.2018.05.013.

[40] Pozza L, Saetta A, Savoia M, Talledo D. Angle bracket connections for CLT structures: Experimental characterization and numerical modelling. Constr Build Mater 2018;191:95–113. https://doi.org/10.1016/j.conbuildmat.2018.09.112.

[41] Tomasi R, Smith I. Experimental Characterization of Monotonic and Cyclic Loading Responses of CLT Panel-To-Foundation Angle Bracket Connections. J Mater Civ Eng 2015;27(6). https://doi.org/10.1061/(ASCE)MT.1943-5533.0001144.

[42] Fanti R, Polastri A, Casagrande D, Callegari E, Ramazzini A, Sestigiani L, Lang P. Experimental Characterization of the Multi-Directional Behaviour of Angle Brackets and Hold-Downs. 2023. World Conference on Timber Engineering. Oslo, Norway: WCTE; 2023. https://doi.org/10.52202/069179-0178.

[43] Brown J, Li M, Sarti F. Structural performance of CLT shear connections with castellations and angle brackets. Eng Struct 2021;240:112346. https://doi.org/10.1016/j.engstruct.2021.112346.

[44] Xiong H, Huynh A. Mechanical behaviour of connections between CLT panels under monotonic and cyclic loading. IOP Conference Series Earth Environmental Science 2018;153:042014. https://doi.org/10.1088/1755-1315/153/4/042014.

[45] D'Arenzo G, Rinaldin G, Fossetti M, Fragiacomo M. An innovative shear-tension angle bracket for Cross-Laminated Timber structures: Experimental tests and numerical modelling. Eng Struct 2019;197:109434. https://doi.org/10.1016/j.engstruct.2019.109434.

[46] D'Arenzo G, Seim W, Fossetti M. Experimental characterization of a biaxial behaviour connector for CLT wall-to-floor connections under different load directions. Constr Build Mater 2021;295:123666. https://doi.org/10.1016/j.conbuildmat.2021.123666.

[47] Casagrande D., Polastri A., Sartori T., Loss C., Chiodega M., (2016) Experimental campaign for the mechanical characterization of connection systems in the seismic design of timber buildings. World Conference on Timber Engineering, WCTE 2016, Vienna, Austria. ⟨https://hdl.handle.net/20.500.14243/335155⟩.

[48] ASTM E2126-19, Standard test methods for cyclic (reversed) load test for shear resistance of vertical elements of the lateral force resisting systems for buildings.

[49] EN 12512:2001/A1:2005, Timber structures – Test methods – cyclic testing of joints made with mechanical fasteners.

[50] ETA-07/0285. European technical assessment. Three-dimensional nailing plate (timber to timber and timber to concrete/steel hold downs and post bases). Simpson Strong-Tie Int. Ltd. Nordhavn, Denmark: ETA; 2018.

[51] ETA-11/0086. European technical assessment. Three-dimensional nailing plate (Angle brackets and hold-downs for timber-to-timber or timber-to-concrete or steel connections). Rotho Blaas s.r.l. Nordhavn, Denmark: ETA; 2015.

[52] ETA-14/0274. European technical assessment. Three-dimensional nailing plate (Angle brackets and hold-downs for timber-to-timber or timber-to-concrete or steel connections). ETA, Nordhavn, Denmark: Adolf Würth GmbH & Co; 2019.

[53] ETA-10/0010. European technical assessment. Three-dimensional nailing plate (Angle brackets and hold-downs for timber-to-timber or timber-to-concrete or steel connections). GH-Baubeschläge GmbH. Nordhavn, Denmark: ETA; 2022.

[54] Soltech srl Moltelabbate, Pesaro e Urbino, Italy. ⟨www.soltechonline.com⟩.

[55] ETA-06/0106. European technical assessment. Three-dimensional nailing plate (timber-to-timber/timber-to concrete angle bracket). Simpson Strong-Tie Int. Ldt. Nordhavn, Denmark: ETA; 2020.

[56] ETA-22/0089 (2024) European technical assessment. Rotho Blaas NINO Angle Brackets and WKR Hold Downs (Three-dimensional nailing plate: Angle Bracket or Hold Down for timber-to-timber or timber-to-concrete or steel connections). Rotho Blaas s.r.l. ETA, Nordhavn, Denmark.

[57] ETA-11/0496. European technical assessment. Three-dimensional nailing plate (Angle Bracket for timber-to-timber or timber-to-concrete or steel connections). Rotho Blaas s.r.l. Nordhavn, Denmark: ETA; 2014.

[58] ETA-08/0183. European technical assessment. Three-dimensional nailing plate (angle bracket for wood to wood connections). BB Stanz- und Umformtechnik. Charlottenlund, Denmark: ETA; 2008.

[59] ETA-09/0323 (2014) European technical assessment. Three-dimensional nailing plate (Angle brackets for timber-to-timber or timber-to-concrete or steel connections). GH-Baubeschläge GmbH. ETA, Nordhavn, Denmark.

[60] ETA-09/0324. European technical assessment. Three-dimensional nailing plate (angle bracket for wood to wood connections, timber-to-steel and timber-to-concrete connections). GH-Baubeschläge GmbH. Charlottenlund, Denmark: ETA; 2014.