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LIGHT-FRAME TIMBER SHEAR-WALLS WITH DIAGONAL BOARD SHEATHING: EXPERIMENTAL AND NUMERICAL INVESTIGATION

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ABSTRACT: The lateral stability of Light-frame timber (LFT) shear-walls is typically ensured by the nailed connection between the wooden frame elements and the sheathing wooden panels, e.g. Oriented Strain Boards (OSB), whereas hold-downs and angle brackets are typically adopted to limit the rigid body rotation and sliding of shear-walls. A valuable alternative to wooden panels in the sheathing of LFT shear-walls is given by diagonal boards. Wooden boards are typically 45° inclined and fastened to frame members, on one or both sides, by means of nails. This paper presents a numerical and experimental study of LFT shear-walls with diagonal board sheathing through extensive experimental campaign and finite element modelling. Monotonic tests were performed on six full-scale LFT shear-walls whereas monotonic shear tests were conducted at connection level to determine the mechanical properties of sheathing-to-framing nailed connections. The results showed that shear-walls sheathed on two sides, with boards which are 90° oriented one to the other, are characterized by significant in-plane shear resistance and can be considered as a feasible alternative to traditional LTF with wooden sheathing panels.

KEYWORDS: Light-frame shear-wall, diagonal sheathing board, lateral load, timber constructions, nailed connection.

1 INTRODUCTION

Light-frame timber (LFT) structures are certainly one of the most spread structural system used for low-to-mid-rise timber buildings worldwide. The lateral load resisting systems in such structural type is typically composed of LFT platform-type shear-walls which are designed to resist both vertical and lateral loads. The lateral stability of LFT shear-walls is typically ensured by nailed connection between wooden frame and sheathing elements, typically wooden panels such as e.g. Oriented Strain Boards (OSB) or plywood. Hold-downs and angle brackets are typically adopted to limit the rigid body rotation and sliding of shear-walls, especially in seismic prone areas, where lateral forces can reach particularly severe intensity.

Several comprehensive studies have been conducted to investigate the mechanical behaviour of traditional LFT shear-wall subjected to lateral loads. The seismic behaviour of shear-walls sheathed with OSB panels was investigated in the CUREE project ([1], [2]) through full-scale shear-wall tests and full-size building shake table tests. Salenikovich and Dolan [3] investigated the monotonic and cyclic behaviour of shear-walls with different aspect ratios showing that sheathing-to-framing connection mechanical properties and connection spacing have large influence on the shear-walls' response. The NEESWood project [4] provided essential outcomes obtained from shake table tests on two-storey and six storey buildings that lead to define the seismic behaviour of low-to-midrise LFT buildings in North America, through the development of a performance-based seismic design approach. Tomasi et al. [5] and Casagrande et al. [6], within the SERIES project, investigated the seismic response of two three-storey LTF buildings braced with LFT shear-walls sheathed with OSB or gypsum panels by means of shaking table tests.

A valuable alternative to wooden panels, for sheathing LFT shear-walls, is given by diagonal boards. Wooden boards are typically 45° inclined and fastened to the frame members by means of nails (Figure 1a). LFT walls with diagonal board sheathing had been largely used for many decades in the construction of low-rise buildings primarily in North America and a revival of such structural type is being observed also in Europe nowadays. However, limited information on their mechanical behaviour is available in literature. Partial designed models have been developed and limited provisions have been included in Standard documents [7] The mechanical behaviour of light-frame timber walls with diagonal board sheathing has been studied since the late 1950.

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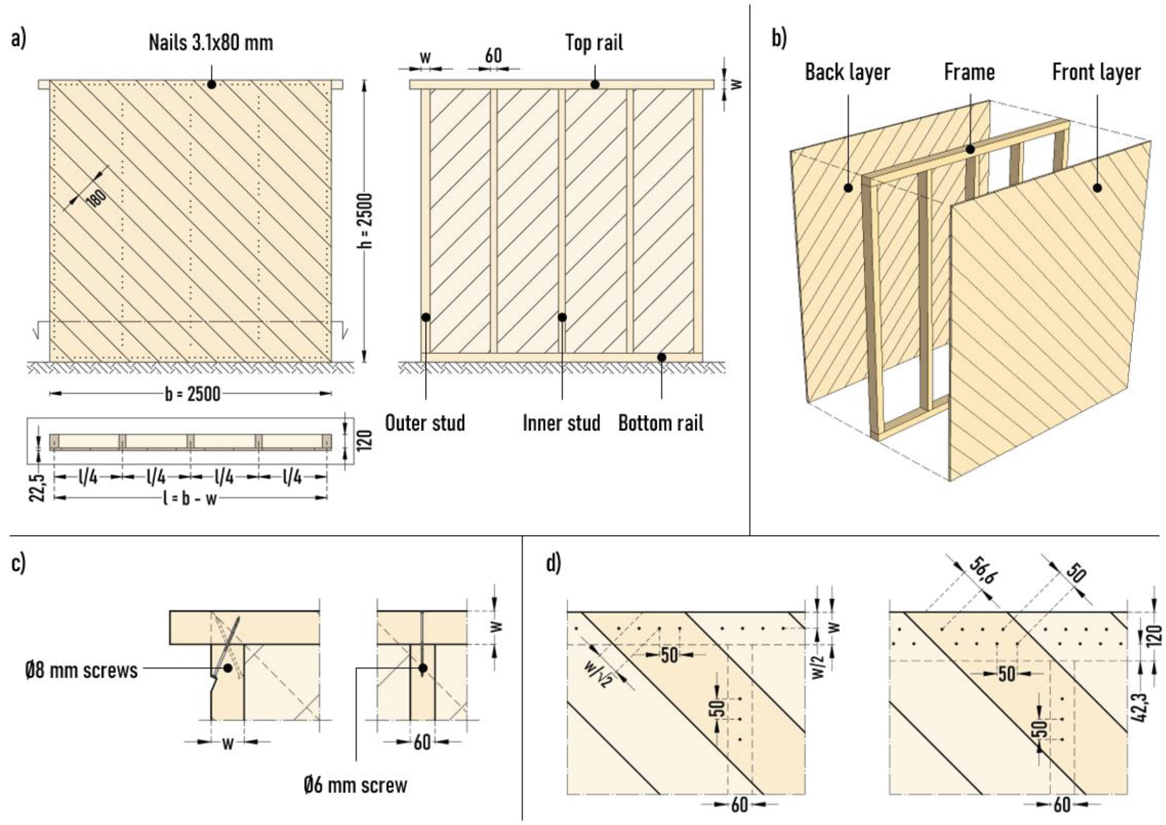

Figure 1: *Shear-wall types and geometries: a) single sheathing board layer (1s) and general dimensions; b) double crossed sheathing board layers (2s); c) frame member connections; d) sheathing-to-framing nail layouts.*

Doyle [8] conducted an experimental campaign on seventeen shear-walls characterized by: different number of sheathing layer (single or double layers), type of boards (full-length and jointed) and corner with or without reinforcements The results provided by single sheathing layer walls showed that the overall mechanical behaviour depended on stresses which are acting within the boards: shear-walls with boards subjected to tension were stiffer and stronger than shear-walls with boards subjected to compression. Similar outcomes were obtained by Ni and Karacabeyli [9] in Canada by means of both monotonic and cyclic tests on sixteen shear-walls. The influence of different parameters was studied: sheathing layer configurations (either single or double layer), type of mechanical anchor, magnitude of vertical loads and reinforcements.

This paper presents a numerical and experimental study of the mechanical behaviour of LFT shear-walls with diagonal boards sheathing through experimental and numerical analyses. Monotonic tests were performed on six LFT shear-walls whereas monotonic shear tests were conducted at connection level to determine the mechanical properties of sheathing-to-framing nailed connections. Numerical finite element models were implemented to predict the mechanical behaviour of tested shear-walls.

2 EXPERIMENTAL CAMPAIGN

2.1 SHEAR-WALL TESTS

The shear-walls were assembled at the mechanical test laboratory of the Institute of Bioeconomy (formerly Ivalsa) of the National Research Council of Italy. Monotonic tests were carried out on six 2.50x2.50 m LFT shear-walls sheathed with boards (Figure 1a and 1b). Either 80x120 mm (type A) or 120x120 mm (type B) solid wood members were used as frame outer members (rails and outer studs) whereas 60x120 mm members were adopted for inner studs. The specimens were assembled using C24 wooden members; studs were spaced at quarter length of shear-wall (l) defined as the difference between the base of the wall (b) and the width of the frame perimeter member (w). A couple of crossed and inclined semi-threaded screws (8x160 or 8x200 mm) were adopted to ensure connection between rails and outer studs whereas a single semi-threaded screw (6x160 or 6x200 mm) was adopted as connection between rails and inner studs (Figure 1c). C16 boards (width 180±20 mm and thickness 22.5mm) were placed inclined 45° with respect to the timber frame and connected to the frame by means of either four or eight 3.1x80 mm ring nails. Three nails were used to connect the boards to the inner studs (Figure 1d). Sheathing-to-framing nailed connection layouts were adopted in accordance with the provisions reported in Eurocode 5 [10]. Sheathing boards were placed on either one side (1s) or both sides (2s) of the wall frame; this latter

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configuration had boards placed on one side 90° oriented from boards on other side of the frame (double crossed layers) as shown in Figure 1. Test set-up used for full-scale shear-wall experimental campaign is shown in Figure 2. Specimens were anchored to the steel foundation by means of a couple of special mechanical anchors designed to fully prevent sliding and rocking of the specimen. Such anchors consisted of 15 mm thick steel plates provided with slotted holes where a special hold-down was connected by a steel dowel in order to allow free rotation of the specimen. These steel plates were bolted to the steel foundation element and the special hold-downs were connected to lateral side of outer studs by means of 5x70 mm LBS screws (30 or 60 screws were used for 1s or 2s shear-wall type respectively). As shown in Figure 2, lateral load was applied by means of a horizontal hydraulic jack through a steel top beam equipped with thick angular steel plates at top wooden rail ends to transfer the load from the hydraulic actuator to the top rail of the wall. Additional restraints were provided at the top of the wall specimen, on both sides, to prevent out-of-plane displacements of the wall and buckling.

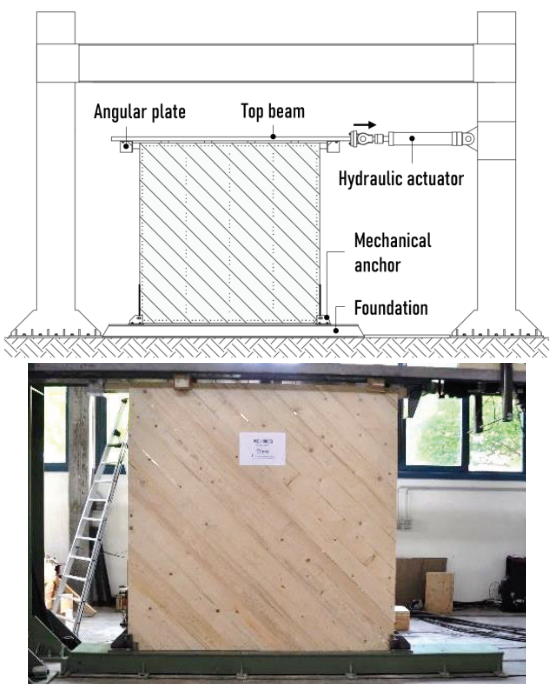

Figure 2: Set-up of full-scale shear-wall tests.

The instrumentation layout is represented in Figure 3. Linear Variable Displacement Transductors (LVDT) were used to measure the top horizontal displacement δ_T (M1), and the sliding of the specimen δ_B (M2) at the foundation level; two vertical LVDTs were placed in correspondence of ground anchor to measure the up-lift δ_V (M3 and M4) of the specimen. Finally, two diagonal LVDTs were placed to measure diagonal deformation of the frame (M5 and M6). The list of specimens used for experimental campaign is reported in Table 1.

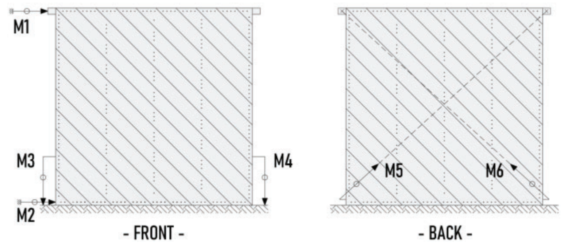

***Figure 3:** Instrumentation layout for full-scale shear-wall tests.*

Each specimen reported an alphanumeric label: the sheathing layer configuration (1s or 2s) was identified; the next letter ("C" or "T") depended on stress which was acting within diagonal boards during the test (compression or tension respectively), see Figure 4; the last part identified the sheathing-to-framing nail configuration which depended on both the number of nails adopted (either four or eight) and on the dimensions of frame perimeter members (defined with letters "A" and "B").

With regard to W001b, the specimen used in test W001 was strengthened by means of an angle bracket at the upper corner (Figure 5). The purpose of this solution was to create a corner reinforcement (CR) in order to prevent failure of the perimeter member connection, thereby increasing in-plane strength capacity.

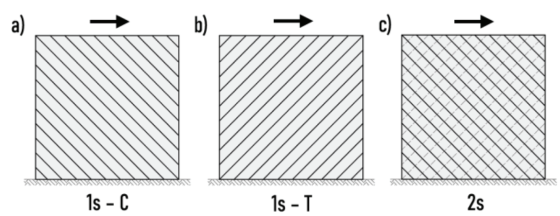

***Figure 4:** Laying direction of boards compared to load direction used for W001 and W001b (compression) (a), W002 (tension) (b) and W003, W004 and W005 (c).*

Monotonic test procedure was defined in accordance with EN594. The test protocol consisted in a linear displacement with a constant speed of 0.2 mm/s imposed to shear-wall until 100 mm displacement (measured from hydraulic actuator LVDT) was reached or failure of the specimen occurred.

***Table 1:** Full-scale shear-wall tests.*

Test	Label	Frame perimeter members
		width x depth
[-]	[-]	[mm]
W001	1s – C – 4A	80 x 120
W001b*	1s – C – 4A – CR	80 x 120
W002	1s – T – 4A	80 x 120
W003	2s – 4A	80 x 120
W004	2s – 4B	120 x 120
W005	2s – 8B	120 x 120

**specimen W001 stiffened with a further corner reinforcement (CR).*

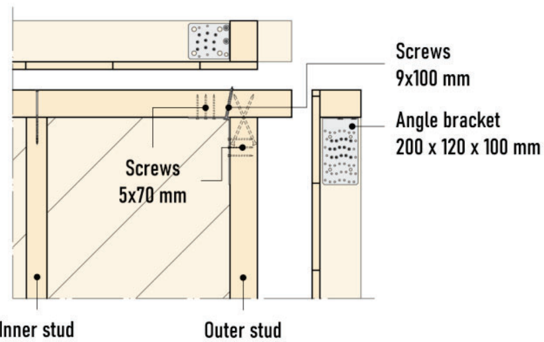

Figure 5: *Details of corner reinforcement used for W001b specimen (1s – C – 4A – CR).*

The results obtained with the experimental campaign are reported in
Table 2. Load-displacement curves are shown in Figure 6. The relative displacement δ was calculated with equation (1); in particular the sliding and the rocking of the specimen (δ_R) were subtracted from the top horizontal displacement.

$$\delta = \delta_T - (\delta_B + \delta_R) \quad (1)$$

The rocking displacement component δ_R is taken equal to δ_V as the aspect ratio of shear-walls is 1:1.
The results obtained from "1s" wall configuration (single sheathing layer) showed that in-plane strength capacity was dependent on the direction of the sheathing boards. As the matter of fact, two completely different behaviours from W001 (1s – C – 4A) and W002 (1s – T – 4A) were observed. During test W001, the shear deformation imposed by the external load was forcing the boards in compression stresses creating a diagonal compressed "strut" within the sheathing layer which led to the failure of the connection at the upper corner (Figure 7a). The concentrated forces caused relative displacements both in vertical and horizontal directions between the outer frame member. In addition, the shear-deformation caused the separation of the boards (Figure 8a).

Conversely, in test W002, the sheathing boards acted in tension. The stress configuration leads the frame perimeter members to compress each other (Figure 7b). Besides, shear deformation caused the contact between each board, generating collateral stresses in perpendicular direction which were acting in opposition to the movement of the specimen and which allowed the shear-wall to reach a maximum force almost four times greater than that obtained for shear-wall with boards subjected to compression (W001) (Figure 8b). The specimen has demonstrated a quite linear behaviour until the end of loading protocol and failure occurred in sheathing-to-framing connection,

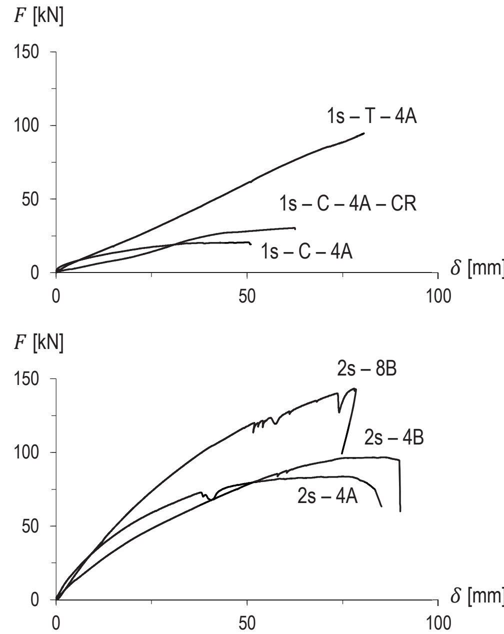

Figure 6: *Load-displacement curves from full-scale shear-wall tests.*

Table 2: *Full-scale shear-wall test results.*

Label	δ_{max}	F_{max}	δ_u	F_u	K	μ
[-]	[mm]	[kN]	[mm]	[kN]	[kN/mm]	[-]
1s – C – 4A	60.1	21.3	64.2	20.0	1.27	n/a*
1s – C – 4A – CR	62.4	30.4	62.5	29.5	n/a*	n/a*
1s – T – 4A	80.5	94.5	80.5	94.5	1.15	1.00
2s – 4A	75.0	83.7	84.6	67.0	3.07	3.98
2s – 4B	85.5	96.7	90.0	77.4	2.00	2.10
2s – 8B	78.1	143.5	78.3	114.8	3.10	1.97

- δ_{max}, F_{max} *peak point of load-displacement curve;*
- δ_u, F_u *ultimate curve point defined as the displacement at 80% of maximum load post peak or displacement limit of test protocol;*
- *K is stiffness assessed as the slope of the straight line passing from the origin of the axes to the yield point that was defined according to provisions reported in EN12512 (intersection between the secant between 10% and 40% of the maximum load and the tangent to the capacity curve with 1/6 slope of the secant line);*
- *μ ductility defined as the ultimate and yielding displacement ratio.*

**not reported as unreliable values (see Table 1).*

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It is worth mentioning that specimen W001b (1s – C – 4A – CR, wall with corner reinforcement) increased in-plane strength capacity of almost 50% compared to W001; failure occurred due to angular bracket yielding used to reinforce the corner connection (Figure 7c).
Significantly different responses were obtained from “2s” wall configurations (double crossed sheathing layers) tests compared to “1s” configurations, providing greater strength capacity and stiffness. Double crossed tests provided a strength capacity almost four-to-seven times greater than W001.

Figure 7: *Wall frame corner detail: a) failure of W001 frame perimeter members connection; b) no visible failure occurred for W002; c) failure of W001b by yielding of angle bracket. Pull-out of frame perimeter members connection for W003 (d), for W004 (e) and W005 (f).*

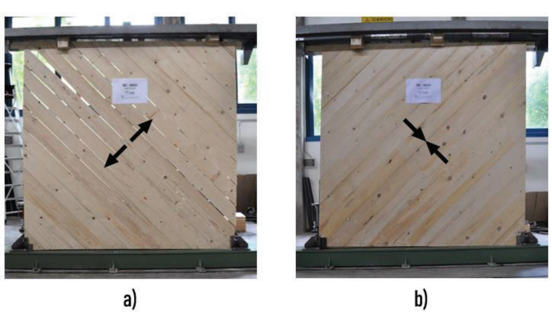

Figure 8: *Shear deformation: a) W001 sheathing boards tend to separate; b) W002 sheathing boards tend to contact.*

Load-displacement curves obtained from “2s” wall configuration tests showed a decreasing trend of the slope due to the progressive withdrawal of the screws which connected frame perimeter members on the left corner during the tests (Figure 7d, 7e and 7f). Similar mechanical behaviour with a decrease of stiffness during the test was observed in “1s” wall configuration with boards subjected to compression stresses. (see Figure 7a).

2.2 CONNECTION LEVEL TESTS

Experimental monotonic tests were carried out at connection-level on sheathing-to-framing nailed connection. These tests were designed and performed in order to characterize the mechanical behaviour of board-to-stud nailed connection and with the aim of collecting input parameters for the numerical model. The three sheathing-to-framing connection configurations used on full-scale shear-wall experimental campaign (see Figure 1d) were tested as reported in Table 3. Three specimens were tested for each configuration. Monotonic tests were carried out in accordance with the loading protocols defined in EN26891 [11].
Test set-up adopted for experimental campaign is represented in Figure 9: four LVDTs were placed to measure relative slip of nails between frame wood element and the two boards. Test protocol consisted in a linear displacement with a constant speed of 0.08 mm/s imposed to top CLT element until failure occurred or 80% of load post peak was reached. Load-displacement curve was drawn through mean displacement and force measured by a 600 kN load.

Table 3: *Sheathing-to-framing connection tests.*

Test	Configuration	Solid wood element		Board	
		width	depth	width	depth
[-]	[-]	[mm]	[mm]	[mm]	[mm]
001	4A	80	120	180	22
002	4A	80	120	180	23
003	4A	77	120	173	21.5
004	4B	120	120	182	22.5
005	4B	120	120	184	22
006	4B	120	120	182	23
007	8B	120	120	180	22.5
008	8B	118	116	170	22
009	8B	118	116	195	22

The results and load-displacement curves are reported per single nail in Table 4 and Figure 10, respectively.
In Table 5 it has also reported the mean values of peak load and load measured at 15 mm displacement on the three sheathing-to-framing configurations. The mean values of the reached maximum strength decrease, increasing both the number of nails and the cross section of frame member. Load-displacement mean curves obtained from connection-level tests were used to characterize sheathing-to-framing connection behaviour for numerical analyses.

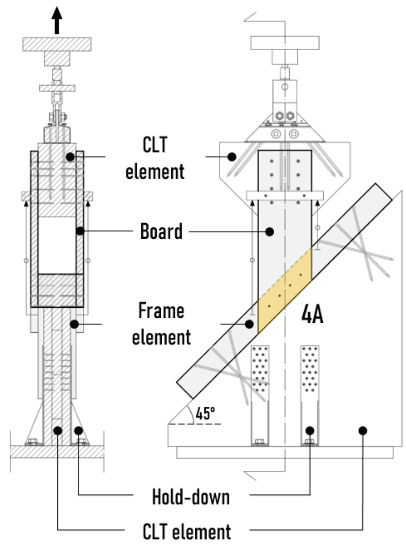

Figure 9: *Sheathing-to-framing connection test setup.*

Table 4: *Connection level test results per single nail.*

Test	Cf.	δ_{max}	F_{max}	δ_u	F_u	K
[-]		[mm]	[kN]	[mm]	[kN]	[kN/mm]
001	4A	17.3	2.16	29.7	1.73	0.22
002	4A	19.0	2.32	30.1	1.90	0.17
003	4A	18.8	1.69	26.9	1.35	0.28
004	4B	17.2	1.66	24.7	1.33	0.42
005	4B	17.3	1.57	28.6	1.26	0.20
006	4B	15.9	1.98	26.3	1.58	0.23
007	8B	18.4	1.65	31.1	1.32	0.32
008	8B	17.4	2.15	30.4	1.72	0.19
009	8B	14.7	2.07	23.4	1.66	0.24

- *δ_{max}, F_{max} peak point of load-displacement curve;*
- *δ_u, F_u ultimate curve point defined as the displacement at 80% of maximum load post peak;*
- *K is stiffness assessed as defined in full-scale shear-wall tests (see Table 2).*

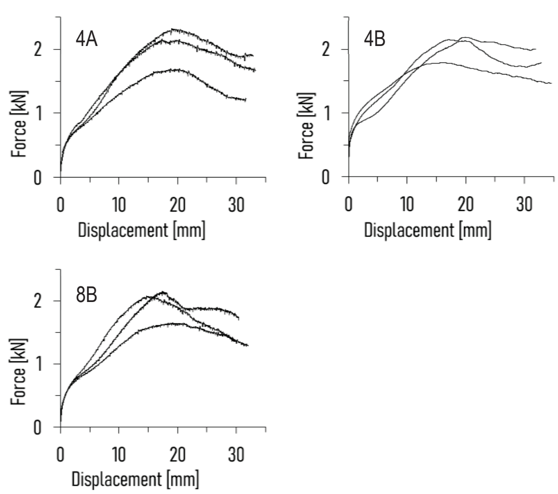

Figure 10: *Load-displacement curve of monotonic tests on sheathing-to-framing connection.*

Table 5: *Sheathing-to-framing configuration mean values and CoV.*

Configuration	F_{max}	CoV	F_{15mm}	CoV
[-]	[kN]	[%]	[kN]	[%]
4A	2.06	16%	1.89	14%
4B	2.03	11%	1.92	7%
8B	1.96	14%	1.96	14%

3 NUMERICAL ANALYSES

Numerical finite element analyses were conducted through the commercial software SAP2000 [13], see Figure 11, in order to predict the mechanical behaviour of LFT shear-walls with diagonal board sheathing.

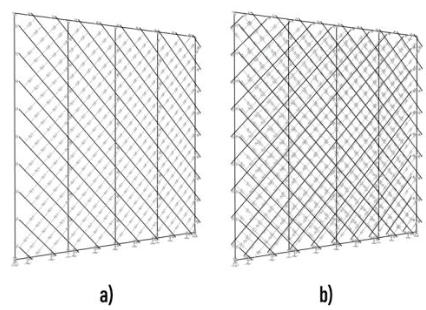

Figure 11: *Numerical model developed for "1s" (a) and "2s" (b) shear-walls.*

Orthotropic material was used to define the properties of wooden members (both frame member and diagonal lumber). The axial and bending stiffness of the solid wood members were set assuming an elastic modulus $E_{0,mean}$ = 12200 MPa .Furthermore, perpendicular to the grain and shear moduli were defined in accordance with EN338 [14]: $E_{90,mean}$ = 390 MPa and G_{mean} = 690 MPa.

Wooden frame and sheathing boards were modelled through "frame" elements. Outer and inner studs were connected to top and bottom rails by means of "non-linear links". Bottom rail was hinged to each end and provided by "line springs" along the frame object and resisting under compression only, in order to simulate the presence of the foundation. Sheathing boards were connected to the frame members by means of "non-linear links" whose mechanical behaviour was obtained by the sheathing-to-framing connection tests. Furthermore, boards were connected each other in perpendicular direction through "gaps" equal spaced in order to simulate the contact between the boards and taking into account the perpendicular to the grain compression forces due to shear-wall deformation.

Lateral increasing force in correspondence of top rail was imposed through non-linear static analyses. As an example, Figure 12 represents the comparison between experimental analysis and numerical results for specimen used in test W003 (2s – 4A).

It is noteworthy to mention that the distribution of stresses within sheathing boards was constant along each single board but with a decreasing trend moving gradually from the diagonal boards to the shorten ones.

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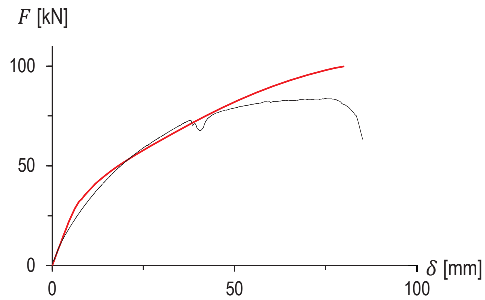

Figure 12: *Comparison between load-displacement curve obtained from test W003 (2s-4A) (black) and numerical result obtained from numerical model (red).*

4 CONCLUSIONS

An experimental campaign and a numerical investigation, on the mechanical behaviour of LFT walls with diagonal sheathing boards was conducted.

Results showed that shear-walls sheathed on one side (1s, single sheathing layer shear-walls) demonstrates a completely different behaviour in terms of resistance and stiffness, based on stresses which acting within sheathing boards, showing how the lateral resistance is strongly ruled by board laying direction. Corner reinforcement could be provided in order to avoid early failure in the connection between perimeter wood members of “1s” shear-wall, the reinforcement increases lateral resistance.

Regarding shear-walls sheathed on two sides with boards placed on one side 90° oriented from boards on other side of the frame (2s, double crossed sheathing layer shear-wall), the results showed that are characterized by significant in-plane shear resistance comparable to traditional LFT walls braced with wooden sheathing panels. The finite element model gave reliable results in terms of capacity curve if compared with these obtained from full-scale shear-wall tests, confirming that the model developed can be used to predict the mechanical behaviour of light-frame timber shear-walls with diagonal board sheathing.

5 ACKNOWLEDGEMENT

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