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Seismic Response of a Platform-Frame System with Steel Columns

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Abstract: Timber platform-frame shear walls are characterized by high ductility and diffuse energy dissipation but limited in-plane shear resistance. A novel lightweight constructive system composed of steel columns braced with oriented strand board (OSB) panels was conceived and tested. Preliminary laboratory tests were performed to study the OSB-to-column connections with self-drilling screws. Then, the seismic response of a shear wall was determined performing a quasi-static cyclic-loading test of a full-scale specimen. Results presented in this work in terms of force-displacement capacity show that this system confers to shear walls high in-plane strength and stiffness with good ductility and dissipative capacity. Therefore, the incorporation of steel columns within OSB bracing panels results in a strong and stiff platform-frame system with high potential for low- and medium-rise buildings in seismic-prone areas.

Keywords: timber structures; platform frame; seismic response; hybrid structures; light-frame shear wall

1. Introduction

Timber building systems, based on platform-frame technology, were born and have been developed in the USA, but are recently spreading across the world. In high seismicity regions, i.e., North America, Italy, Japan and New Zealand, the application of the platform-frame technology as a seismic force resisting system (SFRS) has already proved to be very effective, thanks to its lightness and intrinsic dissipative capacity, when properly designed [1,2]. Platform-frame systems generally benefit from high shear deformability and their dissipative capacity is mainly diffused in connections (mainly nails, screws or staples) between frame and bracing panels.

Also, hybrid systems are of great interest in the current construction practice. Coupling different materials allows the exploitation of their intrinsic properties and the reduction of their limits, improving the overall behaviour of the building. Steel and timber can be integrated into components and/or in a layered construction system (e.g., steel connections with timber frames or timber walls, hybrid frames, steel frames with timber panels) [3,4]. Examples of hybrid construction systems have already been built and tested: steel beams or frames combined with cross-laminated timber (CLT) panels [4–7] or with timber-frame shear walls ([8–10]) were studied through experimental tests and numerical modelling. Dubina [11] analysed the results of monotonic and cyclic-loading tests of full-scale shear walls realized with cold-formed steel frames braced with oriented strand board (OSB) panel or corrugated sheet. These systems showed a high ductility and proved to be reliable as SFRSs.

As a response to users' needs, new hybrid light frames have been proposed for optimizing their performance [12]. The innovation is primarily in the materials used for the frame, for the bracing system and for the ductile connections to fasten the panel to the frame. The bracing panels are usually realized with timber-based materials (usually OSB sheets), whereas gypsum or plaster of cement are used as a finishing layer. The influence of these brittle materials on the performance of the timber-frame shear walls is reported in [13]. Another variation of hybrid systems is the use of cold-formed steel components for the internal frame.

For a safe use of these innovative systems in seismic areas, it is necessary to evaluate their mechanical properties by means of experimental tests. In [14] a summary of experimental and modelling studies of light-frame timber shear walls performed in the last two decades of the 20th century is presented. More recently in the USA, the seismic behaviour of typical platform-frame systems has been studied [15] for improving their design procedure and for proposing solutions for retrofitting existing buildings [16].

The hybrid system analysed in this work is composed of an internal frame realized with tubular steel columns and timber beams, which is externally braced with OSB panels on both sides, fastened to the frame with proper dowel-type fasteners. The new building system is an evolution of the one tested and described in [10]. The previous system was composed by a wood frame coupled to tubular steel columns, which supported the vertical loads. In the new system, the steel columns are part of the seismic-resisting frame, in addition to supporting, as previously, the vertical loads. This simplification optimizes the system, making the assembly easier and cheaper with respect to the previous one [10], due to the lack of the timber frame. The removal of the vertical timber posts means that the strength, stiffness and ductility of the new hybrid system rely on the behaviour of the steel-to-panel connections realized with self-drilling screws.

For this purpose, a shear wall was tested according to the quasi-static cyclic loading protocol of EN 12512 [17] with the aim of characterizing this structural system in terms of strength, stiffness, ductility and hysteretic behaviour.

2. Structural Components of the Shear-Wall System

The investigated shear-wall system is a lightweight platform frame realized with steel columns having a square hollow section braced at both sides with OSB panels. The steel columns have the aim of supporting dead and live vertical loads, whereas the OSB panels allow the wall to withstand lateral loads thanks to self-drilling screws, nails and staples, which fasten the panels to the columns and to timber crossbeams within the wall. This precast system is conceived to be modular, with constant spacing of the columns equal to 125 cm. However, the width and height of the wall can be properly adapted to the geometry of the building or to special needs, changing the dimensions of the OSB sheets. Dimensions of the main components and spacing of the fasteners have to be specifically designed for each building. Here, a description of the main components is given. In the following sections, dimensions, thicknesses and spacing chosen for the tested specimen are listed.

The shear wall is composed of:

- Hollow steel columns realized with steel conforming to EN 10219-1 [18];
- Crossbeams realized with glued laminated timber conforming to EN 14080 [19];
- Steel brackets conforming to EN 10219-1 [18] to anchor the columns to the foundation and to the timber floor above, fastened to the columns with 6.3 mm × 19 mm self-drilling steel screws conforming to EN 15480 [20], steel class 9.8 according to ISO 898 [21];
- Bracing system realized with OSB/3 panels conforming to EN 300 [22];
- U-shaped bottom rail continuous for each façade of the building to transmit shear forces between OSB panels and foundation;
- OSB-to-steel joints (i.e., to columns and bottom rail) realized with 5.5 mm × 38 mm self-drilling steel screws conforming to EN 15480 [20], steel class 9.8 according to ISO 898 [21];

- OSB-to-timber joints realized with 4 mm × 60 mm ring shank nails according to the producer homologation document [23].

Figure 1 shows the details of the main components of the shear wall. All these structural elements confer to the wall the lateral stability and the necessary in-plane strength, stiffness and dissipative capacity to resist to earthquake action. The in-plane shear strength of the wall and the dissipation capacity are given by screws and nails, which fasten the OSB panels to the columns and to the crosspiece beams. The steel brackets are placed for supporting floors and to anchor columns at the foundation, in order to avoid the uplift of the shear wall due to rocking behaviour. These brackets are made of the same steel element of the column, diagonally cut, with a steel plate welded below. Finally, the U-shaped bottom rail is placed to transfer shear forces between OSB panels and foundation, avoiding sliding deformations of the wall. To comply with the capacity design approach, the steel brackets, the connections with the bottom rail and the anchoring to foundation should be sufficiently over-resistant with respect to the connections between the OSB panels and the hybrid frame, in order to confer to the wall a diffuse energy dissipation due to shear deformation and to avoid anticipated brittle failures.

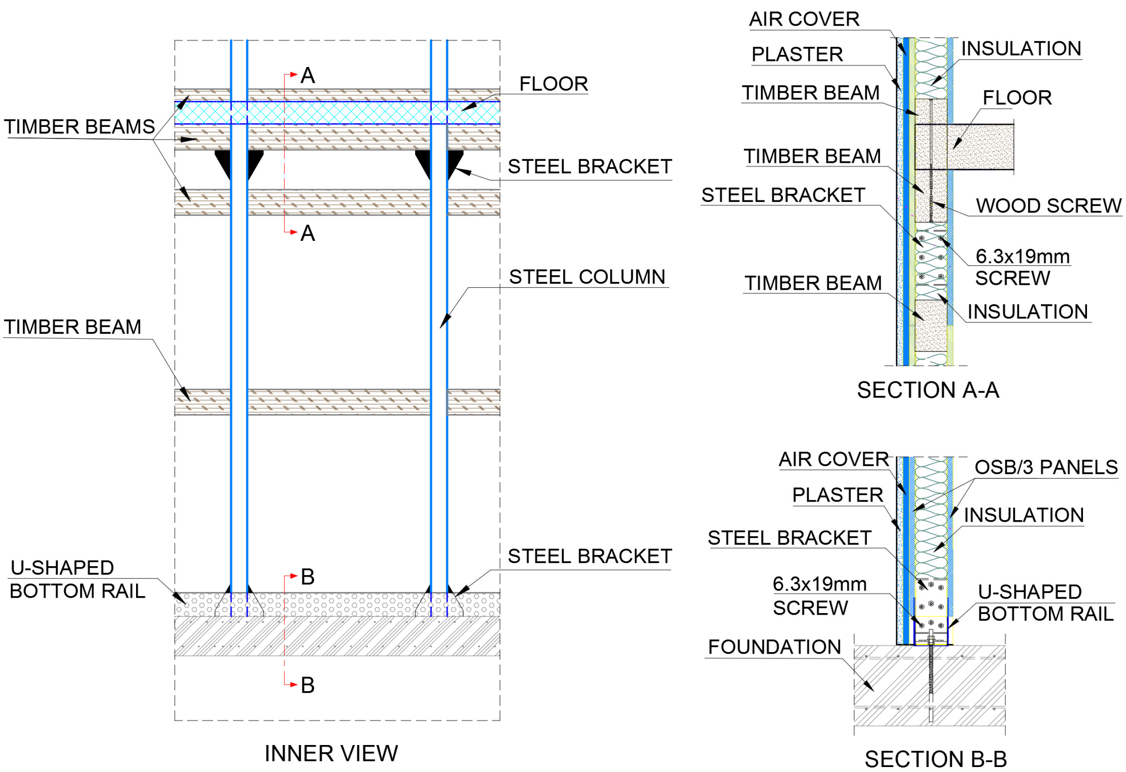


Figure 1. Details of the shear-wall system: inner view of a wall module and detailed sections.

3. Preliminary Laboratory Tests

Preliminary tests were performed at the testing laboratory for construction materials of the University of Padova to determine the mechanical behaviour of the OSB-to-column self-drilling screws subjected to a displacement-driven monotonic load applied at a rate of 2.0 mm/min. The tests were performed in a universal testing machine with a load capacity of up to 250 kN. A conventional push-out test configuration was adopted, which induces pure shear loading conditions in the connection by compression (Figure 2a).

The specimen was composed by a cold formed steel hollow section of 100 mm × 100 mm having thickness equal to 2 mm or 3 mm and two 18 mm thick OSB/3 panels. Each panel was connected to the tubular element with four 5.5 mm × 50 mm self-drilling screws. The use of two different thicknesses of the columns was chosen to analyse the possible different response of the connection. Figure 2b shows the geometrical details of the specimens. A symmetric disposition of the panels was chosen in accordance to the intended use of this system and to avoid out-of-plane deformations. A total of six specimens were tested, three for each analysed thickness of the column. This is the minimum number of specimens needed to compute the 5% characteristic value of a property according to EN1990 [24], if the coefficient of variation is unknown from prior knowledge. The test results are here reported in terms of failure load and force-displacement curves recorded for each specimen. Table 1 lists the main mechanical parameters and the mean and characteristic values among the three tests: yielding point (V_y, F_y), ultimate displacement V_u, maximum shear strength F_{max}, stiffness for the elastic and post-elastic branches (k_e, k_p), ductility μ. The 5% characteristic values were obtained using a k_n factor equal to 3.37, according to annex D on EN1990 [24] for a number of specimen equal to three. Figure 3 shows the force-displacement curves up to failure of the tested specimens.

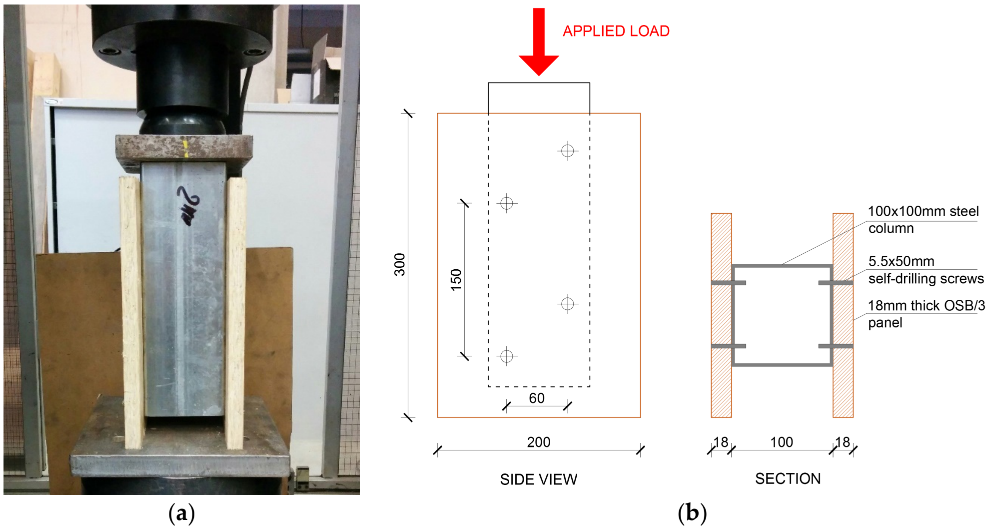


Figure 2. Preliminary laboratory tests: (**a**) photo of a specimen before the test; (**b**) Geometry of the specimens.

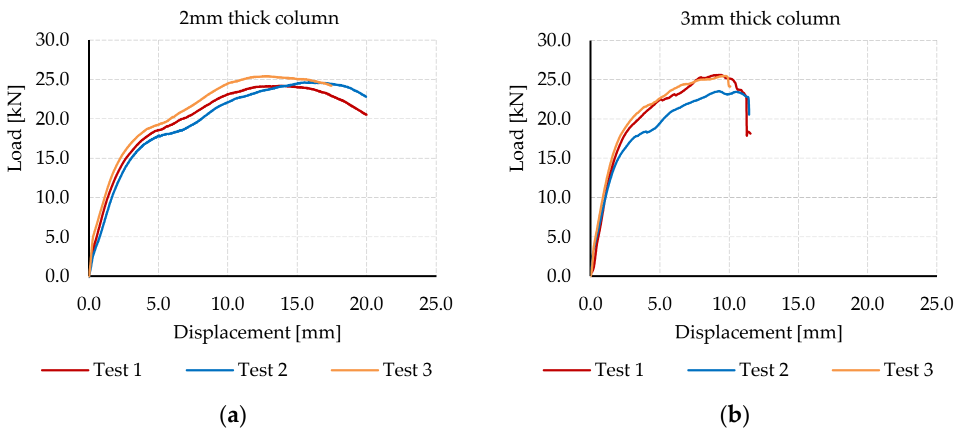
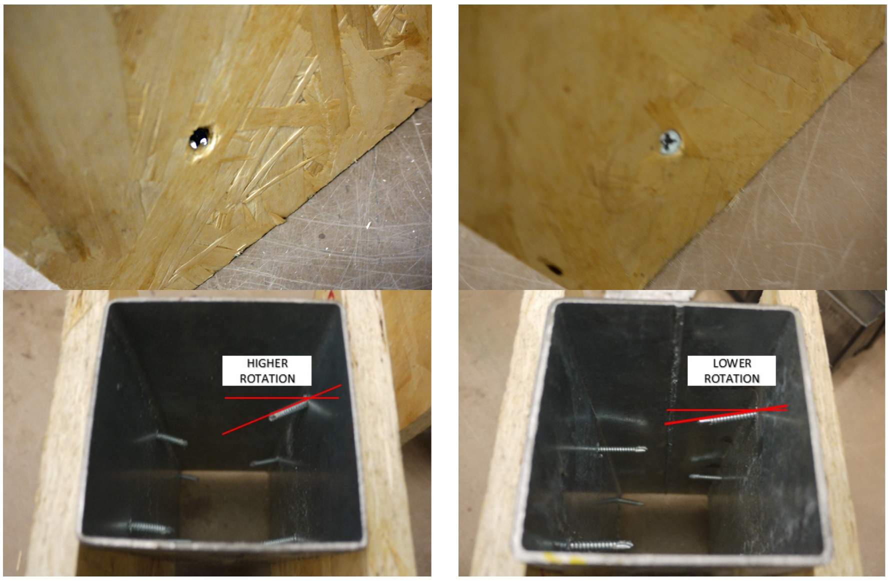


Figure 3. Force-displacement curves for the six tested specimens: (**a**) specimens with 2 mm thick steel column; (**b**) specimens with 3 mm thick steel column.

The results clearly show a good force-displacement response of the OSB-steel connection with a characteristic failure load per screw equal to 2.84 kN for the 2 mm thick column and 2.63 kN for the

3 mm thick column. It is worth noting that the thickness of the column does not influence the strength of the connection. Conversely, the displacement capacity recorded for the 2 mm column was higher than for the 3 mm column, and the failure mode was different. For the 2 mm column, a preliminary elastic phase was followed by wood embedment and formation of one plastic hinge near the screw head. Then, a second higher post-elastic stiffness was recorded due to rope effect, followed by a final softening branch after the maximum strength, and failure due to penetration of the screw head in the OSB panel. For the 3 mm column, a higher elastic stiffness was evidenced, together with a less ductile failure due to premature shear cutting of the connectors after slight wood embedment and formation of a plastic hinge. The failure modes are clearly shown in Figure 4. It can be seen in Figure 4a that the 2 mm specimens showed a marked rotation of the screws and penetration of the head. Conversely, the screws in the 3 mm column showed reduced rotation and failed for shear (Figure 4b).



(**a**) (**b**)

Figure 4. Type of failure evidenced during tests: (**a**) Specimens with 2 mm thick steel column; (**b**) specimens with 3 mm thick steel column.

Table 1. Main mechanical parameters of the tested specimens according to EN 12512 method b [17]; characteristic values according to EN1990 [24].

Parameter (Units)	2 mm Thick Column					3 mm Thick Column				
	TEST 1	TEST 2	TEST 3	X_{mean}	$k_{0.05\%}$	TEST 1	TEST 2	TEST 3	X_{mean}	$k_{0.05\%}$
V_u (mm)	19.8	19.8	17.4	19.0	15.0	10.9	11.2	9.6	10.6	7.9
F_{max} (kN)	24.1	24.6	25.4	24.7	22.8	25.4	23.4	25.4	24.7	21.2
k_e (kN/mm)	6.7	5.5	7.6	6.6	3.4	10.4	8.2	10.5	9.7	5.8
k_p (kN/mm)	1.1	0.9	1.3	1.1	0.6	1.7	1.4	1.8	1.6	1.0
V_y (mm)	2.1	2.7	1.9	2.2	1.0	1.7	1.7	1.6	1.7	1.4
F_y (kN)	15.4	15.6	15.7	15.6	15.1	16.9	15.6	17.5	16.7	13.7
μ (-)	9.2	7.4	9.2	8.6	5.4	6.4	6.4	6.0	6.3	5.6

These preliminary results led us to choose the 2 mm steel column for the following full-scale test, to obtain a better seismic response of the shear wall. It is clear that the thickness of the column has to be verified also for withstanding vertical loads. Another outcome of these tests was that, using a 2 mm thick steel column, the response of the OSB-steel connection is independent from the length of the screw, as can be noticed in Figure 4a. This led us to choose shorter 5.5 × 38 mm screws for the full-scale test.

4. Full-Scale Test

4.1. Description of the Specimen

A quasi-static cyclic-loading test of a full-scale specimen was performed according to EN12512 [17] to define the seismic response of a shear wall. The specimen was anchored to a reinforced concrete (RC) foundation to reproduce actual conditions and was subjected to a displacement-driven cyclic loading at the top. The wall specimen had dimensions equal to 260 cm × 329 cm, i.e., two 125 cm × 329 cm wall modules. It was composed by three 2 mm thick steel columns and four 18 mm OSB/3 panels at each side, i.e., two 125 cm × 250 cm OSB panels and another two 125 cm × 68 cm panels above. To connect the panels among them and to transmit the lateral loads from the actuator to the foundation, six timber GL24h-class crossbeams were placed inside the wall: two 10 cm × 28 cm beams above the wall, fastened to the 125 cm × 68 cm OSB panels, to distribute lateral and vertical loads (upper beams), two 10 cm × 20 cm beams to transmit the shear forces between the upper and the lower panels (connecting beams), and two additional 10 cm × 8 cm beams in the middle of the 125 cm × 250 cm panels (additional beams) to avoid buckling of the panels. The shear wall was anchored to the foundation with two brackets for each column by means of eight 6.3 mm × 19 mm self-drilling screws and a M12 × 150 mm steel bar (class 4.8) for each bracket. The same brackets were also used to connect the three columns with the upper beams and therefore with the actuator. The following fasteners were used to connect the OSB panels to the columns and the beams: 5.5 mm × 38 mm/15 cm self-drilling screws between columns and panels; 4 mm × 60 mm/15 cm ring shank nails between upper beams and panels and between connecting beams and panels; 1.4 mm × 1.65 mm × 50 mm/10 cm staples between additional beams and panels. Finally, a 2 mm thick U-shaped bottom rail (S235JR steel) was placed at the base of the wall to transmit shear forces between OSB panels and the foundation, through 5.5 mm × 38 mm/15 cm self-drilling screws. Table 2 lists the dimensions and materials of the main structural components for a complete description of the specimen and Figure 5 shows an inner, an external and a side view of the specimen with main dimensions and test setup.

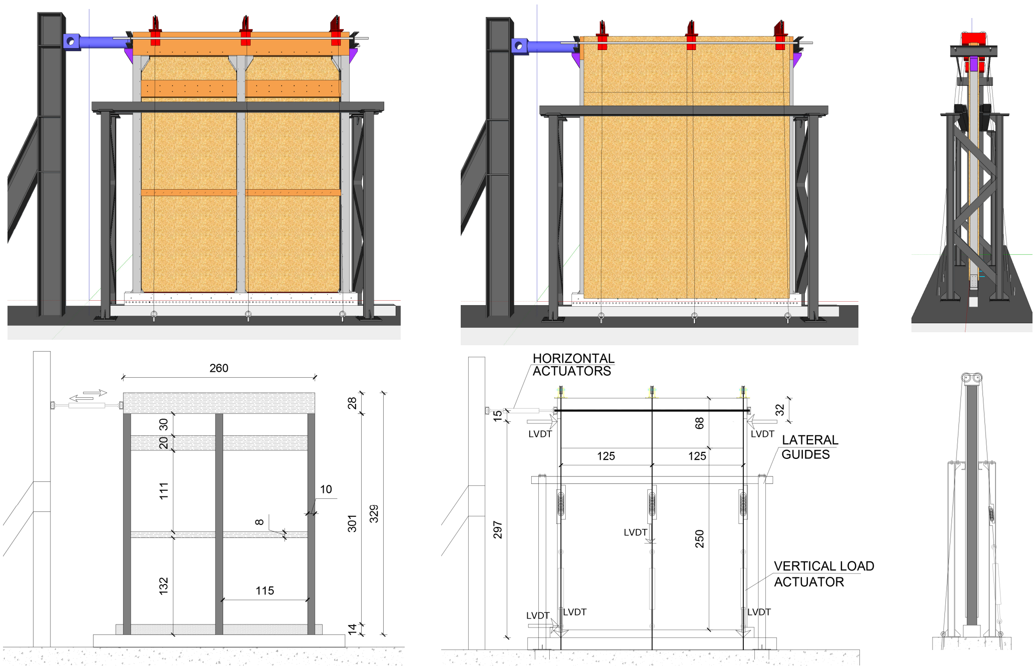
Table 2. Dimensions, fasteners and geometrical details of the specimen.

Structural Components		**Material**	**Dimensions**	
Bracing panels		OSB/3	Thickness	18 mm
Hollow columns		Steel S235JR	Section	100 mm × 100 mm × 2 mm
Crossbeams		Glulam GL24h	Section	10 cm × 28 cm 10 cm × 20 cm 10 cm × 8 cm
U-shaped bottom rail		Steel S235JR	Thickness	2 mm
Brackets		Steel S235JR	Section	100 mm × 100 mm× 3 mm
Fasteners	OSB-to-column	Self-drilling screws	Dimensions/spacing	Ø5.5 mm × 38 mm/15 cm
	OSB-to-upper beam OSB-to-connecting beam	Ring shank nails	Dimensions/spacing	Ø4 mm × 60 mm/15 cm
	Bracket-to-column	Self- drilling screws	Dimensions	Ø6.3 mm × 19 mm
	Bracket-to-upper beam	Wood screw	Dimensions	Ø8 mm× 100 mm
	OSB-to-bottom rail	Self- drilling screws	Dimensions/spacing	Ø5.5 mm × 38 mm/15 cm

4.2. Description of the Test Setup and Procedure

Figure 5 shows the test setup employed, which was chosen to be consistent with the previous experimental campaigns of other constructive systems [10,25]. A vertical constant load of 8.8 kN (reproducing gravitational loads at the first storey of a low-rise building with lightweight floors) was applied to each steel column by means of hydraulic actuators. Lateral restraints were positioned at the top of the specimen to avoid out-of-plane movements. Displacements of the walls were measured with

Linear Variable Displacement Transducer (LVDT), placed as illustrated in Figure 5b. A quasi-static cyclic-loading test was performed in displacement control, according to EN 12512 [17], following the protocol shown in Figure 6. Such testing protocol requires the definition of the yielding displacement V_y of the specimen assumed equal to 10 mm. Figure 7 shows photos of the specimen before the test.



(**a**) (**b**) (**c**)

Figure 5. Specimen and test setup: (**a**) inner view; (**b**) external view and setup; (**c**) side view. (Dimensions in cm).

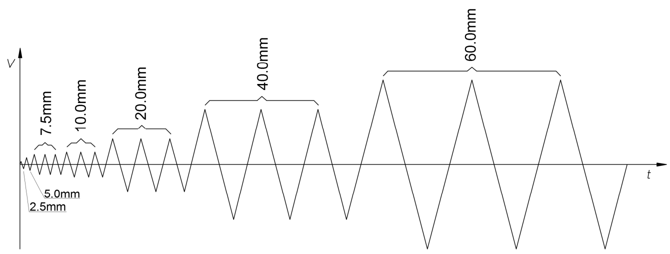


Figure 6. Adopted test protocol according to EN 12512 [17].

(**a**) (**b**)

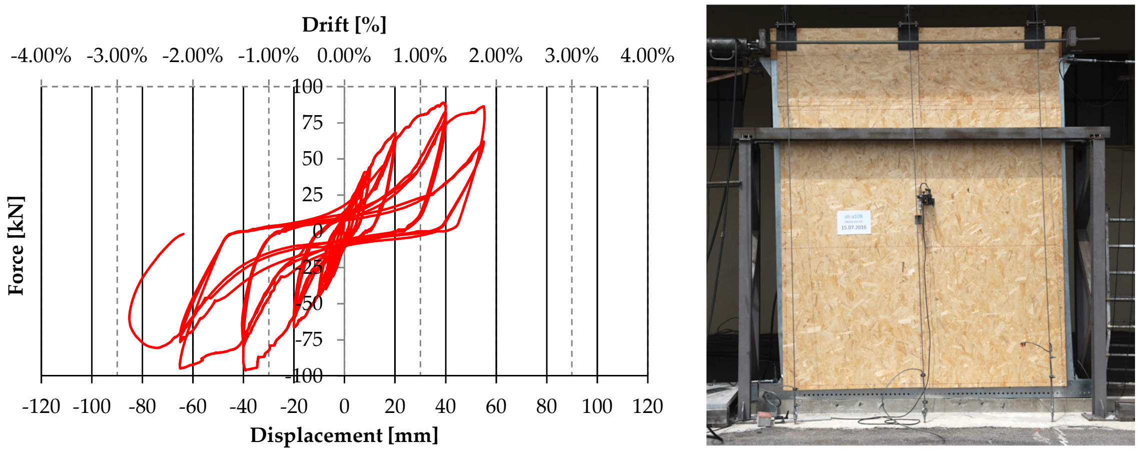
Figure 7. Photos of the specimen before the test: (**a**) front view; (**b**) back view.

5. Discussion

5.1. Test Results

Figure 8 plots the hysteresis cycles of the wall, i.e., base shear force vs. displacement at top LVDT, obtained from the test in terms of absolute and relative values (drift). The drift values have been plotted to provide results independent from the position of the top transducer.

Although characterized by an evident pinching behaviour, the shear wall showed a relatively good seismic response and displacement capacity. However, it can be seen that there is a relevant strength degradation already for 40 mm cycles. The resulting strength degradation between the first and the second 60 mm cycles exceeds 20%, which is the maximum allowable strength degradation corresponding to ultimate serviceability level (i.e., failure conditions) according to chapter 8.3 of Eurocode 8 [26].



(**a**) (**b**)

Figure 8. (**a**) Force vs. top displacement curve; (**b**) deformed specimen at 60 mm cycles.

The response of the shear wall was characterized by ductile failures localized in the steel-to-timber screws and the timber-to-timber nails. These fasteners behaved according to the preliminary tests described in Section 3, allowing appreciable relative slip among panels, as shown in Figure 9a,b. However, for higher displacements some brittle failures occurred at the base of the steel column consisting in withdrawal of OSB-to-column screws, localized buckling phenomena of the steel section at the base and screw head penetration (Figure 9c,d).

5.2. Analysis of Experimental Results

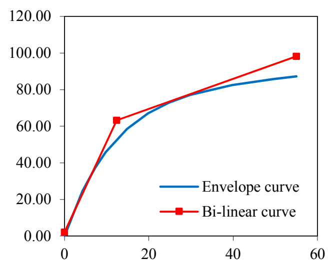
Main test results have been analysed according to EN 12512 [17] to obtain: yielding point (V_y, F_y), ultimate displacement V_u, maximum shear strength F_{max}, stiffness for the elastic and post-elastic branches (k_e, k_p), ductility μ, strength degradation ΔF and viscous damping ratio ν_{eq} (Table 3). The hysteresis cycles in Figure 8a were fitted by means of the envelope curve according to Foschi and Bonac [27]. Then, the mechanical parameters were obtained applying the bi-linearization method (b) of EN 12512 [17] to the envelope curve. The equivalent viscous damping in Table 3 was evaluated at the third cycle of the 40 mm amplitude, which is the highest cycle amplitude with strength degradation less than 20%, and was taken as the minimum between the positive and negative cycles.

Obtained ductility is higher than 4.0, which is the minimum value to be assured for the Medium Ductility Class, according to Eurocode 8 [26]. The equivalent viscous damping ν_{eq} equal to 8.3% confirms the good dissipative capability of this system.

Figure 9. Details of failures evidenced in test: (**a**,**b**) relative slip between panels; (**c**,**d**) withdrawal of self-drilling screws and crushing of the steel column.

Obtained results show that the tested wall was characterized by a quite high in-plane strength, quantified in about 35 kN/m. From a comparison with results in literature relative to traditional timber-frame systems, shear resistance equal to about 10 kN/m was obtained in [28] and equal to about 27 kN/m in [29], confirming the similar behaviour of this system in terms of strength. A comparison in terms of equivalent viscous damping with the mean value equal to 10.3% provided by [30], shows also a similar dissipative capacity. Finally, the comparison with the hybrid wall tested in [10] shows a slightly higher shear strength and stiffness, less displacement capacity, a more pronounced pinching behaviour and higher strength degradation for the system analysed in this work. However, the simplicity and cost effectiveness of this system, together with a good seismic response, make this system interesting and competitive for the use in seismic-prone areas.

Table 3. Test results and interpretation according to EN 12512 method [17].

Parameters	Notations (Units)	Values	Bi-Linear Curve
Ultimate displacement	V_u (mm)	55.0	
Maximum force	F_{max} (kN)	88.9	
Elastic stiffness	k_e (kN/mm)	4.9	
Hardening stiffness	k_p (kN/mm)	0.8	
Yielding displacement	V_y (mm)	12.4	
Yielding force	F_y (kN)	63.2	
Ductility ratio	$\mu = V_u/V_y$ (-)	4.4	
Equivalent viscous damping at 40 mm	ν_{eq} (-)	8.3%	
Strength degradation at 40 mm	ΔF (-)	19.5%	

6. Conclusions

A novel timber lightweight platform-frame system composed of steel columns braced with OSB panels has been presented. The use of steel columns, which can be continuous from the foundation to the roof, permits the avoidance of the compression perpendicular to the grain of the crossbeams, which is a typical issue of traditional multi-storey platform-frame buildings.

Preliminary monotonic tests were performed to analyse the behaviour of the OSB-to-column connections with self-drilling screws. Then, results from a cyclic-loading test of a full-scale shear wall showed that this system is characterized by high in-plane shear strength and good ductility and dissipative capacity, and is classifiable into the Medium Ductility Class according to the European Seismic Code. The good balance of these mechanical parameters with the simplicity, speed of construction and cost effectiveness makes this system an interesting alternative to traditional timber platform frames for low- and medium-rise buildings in seismic-prone areas. It has to be highlighted that only one full-scale specimen was tested. Therefore, to generalize the results and to compute characteristic mechanical parameters proper of the shear-wall system, further tests are needed.

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Author Contributions: All the authors contributed equally to this work. Roberto Scotta, Davide Trutalli and Luca Marchi performed the tests; Lorenzo De Stefani and Luca Pozza analysed the data; all authors discussed the results and wrote the paper; Roberto Scotta supervised all the phases.

Conflicts of Interest: The authors declare no conflict of interest.

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