Earthquake-resistant CLT buildings stiffened with vertical steel ties

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Abstract The interest in multi-storey CLT buildings in seismic areas is leading to the development of new strategies to increase the lateral stiffness of shear walls and to resist high tensile forces due to rocking. Both these purposes can be achieved with vertical steel ties placed at each shear-wall end, to transfer directly tensile forces from each storey to foundation. Three technologies are proposed for transferring forces from the CLT panels to the ties: the use of nailed plates, of the screwed connection X-RAD, or directly by contact with a thick plate at the top of each storey wall. The dynamic behaviour of CLT shear walls, representing the bracing system of a building, anchored with the aforementioned technologies has been investigated by means of dynamic analyses and a comparison with the use of common nailed plates or screwed connections without ties is presented. Results, varying the number of storeys and the seismic mass, show that the proposed technology is an effective strategy to increase the feasibility of multi-storey CLT buildings. Moreover, the substantial increase of the lateral strength makes possible to decrease the ductility demand, reducing lateral displacements and damages.

Keywords Cross-Laminated Timber (CLT), seismic design, steel ties, tall timber buildings, timber structures

1. Introduction

Wood-based engineered products like Cross-Laminated Timber (CLT) are characterized by high in-plane strength and stiffness of the timber panel conferred by the cross-lamination of solid-wood boards [1], which give the multi-storey structure good resistance to vertical and lateral loads [2].

A key role in the mechanical behaviour of CLT structures subjected to lateral loads (wind or earthquake) is assumed by connections, generally hold-downs and angle brackets, which are assigned the function of connecting the panels together and to concrete foundation. These nailed brackets were originally developed for the use in light-frame systems and were not originally conceived to be particularly strong and ductile [3]. The adoption of these connections in CLT structures has required to increase thickness, steel grade, number of nails and to add new stiffeners to reduce local deformations. Nevertheless, it has been demonstrated that stiffness and load-bearing capacity of such connections may not be sufficient to realize CLT buildings taller than five or six storeys in high-seismicity areas, due to the very high tensile forces at hold-downs and the excessive lateral flexibility of the structure [4,5]. The rocking contribution takes on great importance with increasing seismic action, height and slenderness of the shear walls. Finally, the increasing architectural needs for internal free spaces are leading to optimize the number of shear walls in the building, increasing further the strength demand to connections.

Various strategies have therefore been developed for slender multi-storey CLT buildings in high-seismicity areas [6,7]:

1. Hybrid timber-concrete [8] or timber-steel systems [9,10];
2. Use of post-tensioning bars, to reduce tensile forces at connections and assuring recentering of shear walls [11–16];
3. Rocking coupled shear walls with vertical joints [17,18];
4. Reduction of inertial forces using connections with high ductility and dissipative capacity, employing the post-elastic hysteretic behaviour of steel or the friction to dissipate energy during a strong earthquake, preserving the structure from damages [19–25].

All these strategies require the adoption of special technologies, innovative connections or the coupling of different materials, and the consequent development of new design methodologies, normally not implemented by regulations.

In this work, an original earthquake-resistant system based on the use of vertical steel ties as alternative or in addition to traditional nailed plates or screwed connections is analysed. According to this technology, high-strength vertical CLT cantilevers can be realized to brace the structure, optimizing the amount of connections in the building, without the use of prestress, special connections or hybrid structures.

A technology based on the use of strong and stiff steel ties may reduce the dissipative capacity of the structure, compared to the use of nailed plates or the dissipative connections mentioned above. For this reason, a non-dissipative seismic design of the wall systems has been followed in the calculations presented in the following Sections, adopting a behaviour factor q equal to 1.5 according to [26]. It is worth noting anyway that this choice does not penalize particularly the design of the structure, as current regulations limit significantly the behaviour factor associated to CLT buildings. For example, current European Seismic Code gives a maximum value equal to 2 [26], and current Italian Code for Constructions equal to 2.5 [27]. The design with a behaviour factor equal to 1.5, i.e., in accordance with low-dissipative structural behaviour concept [26], leads to a significant increase of the lateral stiffness of the structure: damages after an earthquake are limited and the structure is less flexible also for wind action, with a clear benefit also in terms of reduced discomfort for wind-induced vibrations in high-rise buildings [28].

2. Description of the technology

In low- and mid-rise CLT structures, tensile and shear forces, which are generated at each storey level from lateral loads, are generally resisted respectively by hold-downs and angle brackets. Alternative connections can be used, suitably designed to optimize one or more targets: strength, ductility, dissipative capacity. Independently from the type of connection, they are attributed both to resist the abovementioned forces and to limit the lateral displacements of the structure. As the number of storeys of the building increases, there is an increase in tensile forces at connections especially for slender shear walls. As already mentioned, the rocking contribution to lateral displacements takes on increasingly importance as the height of the building grows, with a consequent increase in the deformability of the structure.

To limit the lateral displacements and to withstand the high tensile forces in multi-storey CLT buildings, connections with high stiffness and strength are needed. Traditional hold-downs, composed of a steel plate and many nails or screws, may not be enough. For this reason, steel ties, which may consist of steel plates, bars or profiles have been analysed, to work in addition to or in place of traditional hold-downs or other connections with the same purpose.

In detail, three original connection systems are proposed to resist the tensile forces due to rocking:

a) steel ties coupled with nailed plates;
b) steel ties coupled with the screwed connection X-RAD [29];
c) steel ties only.

The proposed systems are conceived to withstand high seismic tensile forces that arise in multi-storey CLT buildings braced with a limited number of shear walls, limiting rocking, increasing the lateral strength and stiffness of the structure and therefore reducing inter-storey drifts and damages to structural and non-structural components, permitting to increase the height of the building, also in high-seismicity areas. The following subsections present the concept of these three technologies.

2.1. Steel ties coupled with nailed plates

The system is composed by steel ties welded to steel plates, which are nailed to the panels at each storey level, Fig. 1. In this way, the steel ties connect the nailed plates together and transmit the tensile forces derived from them directly to the concrete foundation. According to this strategy, the nailed plates work as hold-downs at each storey but have to withstand only the tensile force deriving from rocking at that storey, whereas the tensile forces derived from the storeys above are directly transferred to the foundation through the ties. This results in reducing the tensile forces at the nailed connections, permitting of overcoming the typical limitations in the realization of multi-storey buildings.

To avoid eccentricities, the nailed plates and the steel ties should be placed at both sides of the wall. Since the steel tie is continuous from the foundation to the top storey, it is necessary to realize a slot through the timber floors. Furthermore, if required, it is possible to realize the tie with various steel elements and then to restore its continuity by welding or bolting.

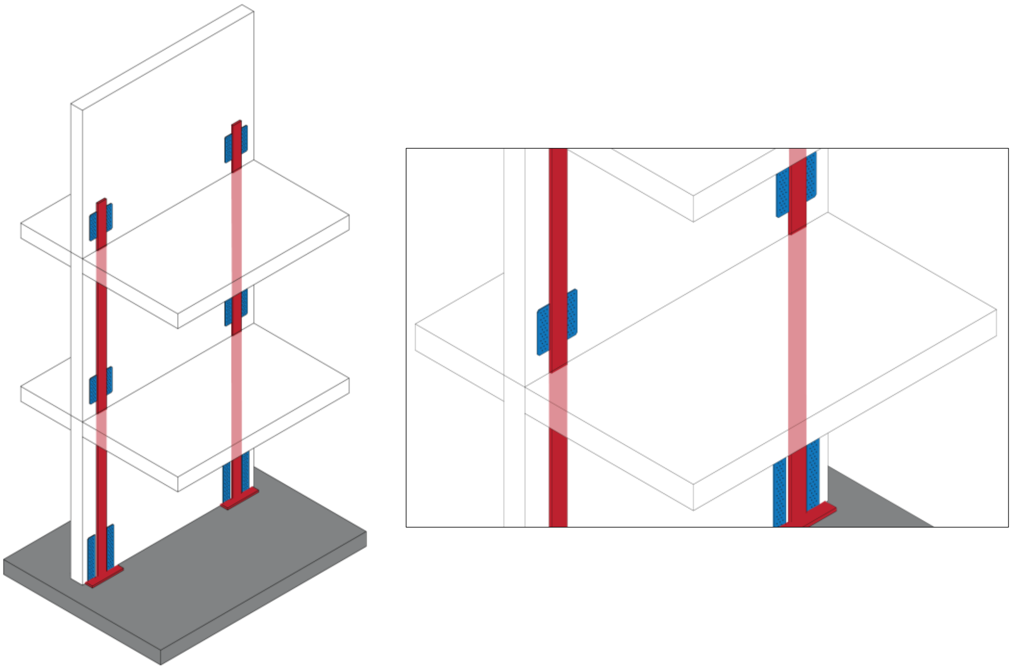


Fig. 1 – Conceptual representation of the steel ties coupled with nailed plates. 3D view and detail (only the front face is shown)

2.2. Steel ties coupled with the screwed connection X-RAD

The system is based on the combination of the screwed connection X-RAD [29] and the steel ties, which work together, Fig. 2. In detail, the steel ties are bolted to the X-RAD at each floor level. As the strategy described in Section 2.1, the ties are continuous along the height of the shear wall and can be composed by various bolted or welded elements. At the corners of the building, the steel ties can be opportunely realized with steel L profiles, whereas elsewhere steel plates can be placed at the narrow sides (thickness) of the wall, Fig. 2. According to this arrangement of the ties, eccentricities are avoided. As for the ties combined with nailed plates, the X-RAD for each storey must withstand only the tensile force deriving from rocking at that storey, whereas the tensile forces derived from the storeys above are directly transferred to the foundation through the tie.

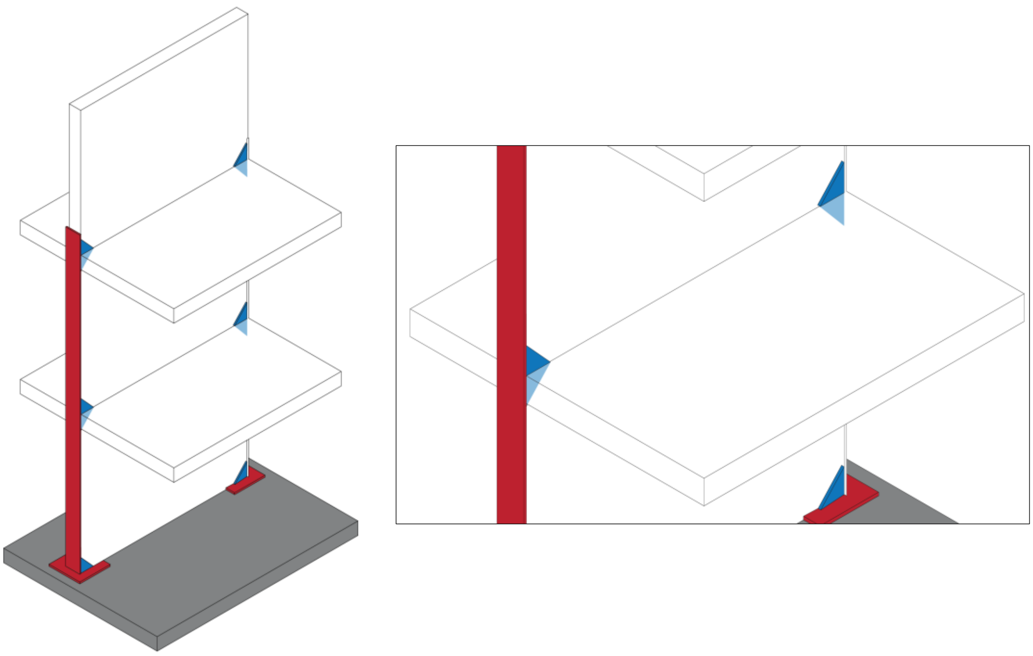


Fig. 2 – Conceptual representation of the steel ties coupled with X-RAD. 3D view and detail (only the front face is shown)

2.3. Steel ties only

This strategy, unlike the previous ones, does not involve the coupling of ties with other connections. The rocking behaviour at each storey is directly prevented by thick steel plates, in direct contact with the top narrow side of each wall panel. In this case, the ties can be realized with two steel bars placed at both sides of the wall; the bars are secured to the plates at each floor level, Fig. 3. Compared to the other two systems (i.e., nailed plates or X-RAD combined with steel ties), this is definitely the stiffest, since the deformation contribution related to nails or screws is avoided.

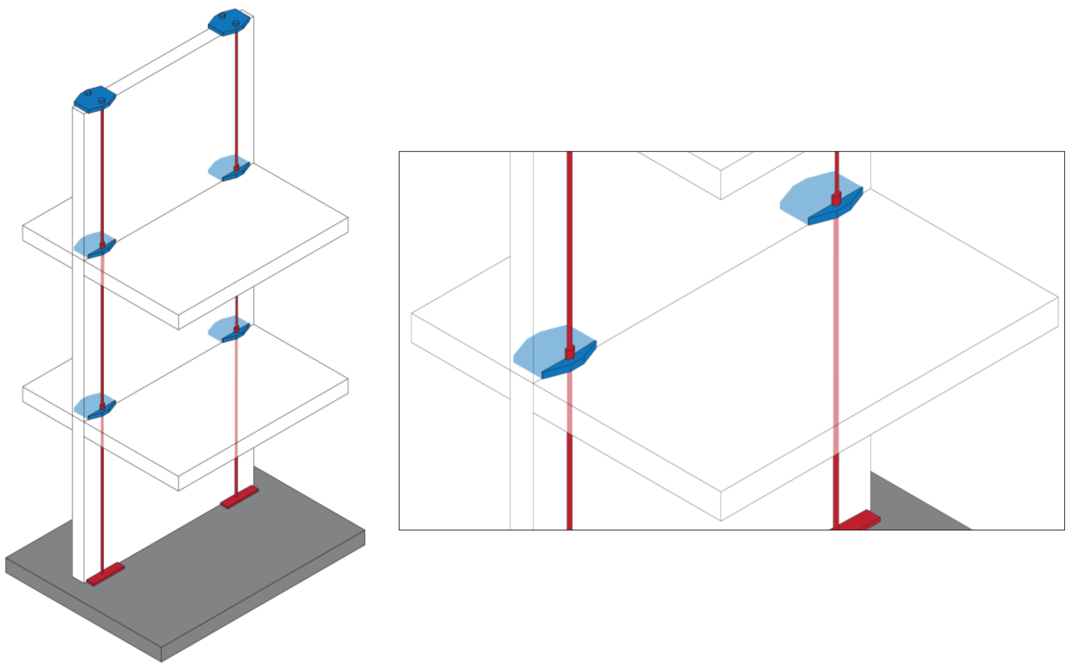


Fig. 3 – Conceptual representation of the steel ties. 3D view and detail (only the front face is shown)

3. Numerical model and analyses

In this Section, the effectiveness of the technology is evaluated, presenting parametric Linear Dynamic Analyses (LDA) of multi-storey CLT shear walls, which represent the bracing system of a building, supposed to be realized in an area with high earthquake hazard.

3.1. Description of the buildings

Two CLT buildings have been considered, having rectangular plan with dimensions of 17.1 × 15.5 m (Building A) and 23.3 × 15.5 m (Building B), and a variable number of storeys (3, 5 and 7 storeys), with an inter-storey height of 3 m. The structure is braced with six shear walls per direction for a total of 12 shear walls in the building, symmetrically arranged in plan. The base length of the walls is equal to 6 m. They are intended to act only as lateral-load resisting systems, whereas all the vertical gravitational loads are resisted by a frame, which is supposed to be perfectly pendular, in order to neglect its effects in the lateral stiffness of the building. The perfect regularity of the buildings and the rigid floor diaphragms allowed to study the behaviour of the whole structure modelling only one shear wall, with the same number of storeys of the building and one sixth of the mass.
The mass was computed according to the seismic combination of EN 1990 [30], assuming floor and roof dead loads equal to 2.5 kN/m^2, floor live loads equal to 2 kN/m^2 and roof live loads equal to 0.5 kN/m^2. The seismic storey masses are listed in Table 1.

Table 1 – Seismic mass per storey relative to the modelled shear wall

	Building A	**Building B**
Dimensions	17.1 × 15.5 m	23.3 × 15.5 m
Floor mass	14.0 t	19.0 t
Roof mass	11.3 t	15.3 t
Wall mass	1.5 t	1.5 t

3.2. Earthquake-resistant connections

Two technologies have been preliminarily considered to connect the panels of the shear walls to the foundation and in-between storeys and to withstand the seismic forces: traditional hold-downs and angle brackets [31] or the innovative strong connection X-RAD [29]. Since for the considered high earthquake intensity, the seismic demand for the five-storey shear walls (in terms of force and/or stiffness) exceeded the capacity of these two connection systems, steel ties have been used to strengthen and stiffen the structure, according to the conceptual representations in Section 2. The use of steel ties allowed also to extend the analyses to the seven-storey shear walls.
Table 2 summarizes all the analysed configurations, with the following meaning of the labels:

- The first letter (A or B) identifies the dimension of the building, that is, the different storey mass assumed for the single shear wall;
- The number (3, 5 or 7) identifies the number of storeys of the shear wall;

- The last letters identify the connection system used or the combination of them: “H” is for hold-down, “X” is for X-RAD (i.e., screwed connection), “T” is for steel ties (see Section 2, Fig. 3), “NP+T” and “X+T” are for a combination of nailed plates or X-RAD with the steel ties (hereafter called also combined configurations or strengthened configurations, see Section 2, Fig. 1 and Fig. 2).

Table 2 – Analysed configurations of the shear walls and labels

Config.	Connection system		Labels					
			Building A			Building B		
	Rocking	Sliding	3 storeys	5 storeys	7 storeys	3 storeys	5 storeys	7 storeys
H	Hold-downs	Angle brackets	**A3H**	**A5H**	**-**	**B3H**	**B5H**	**-**
NP+T	Nailed plates and steel ties	Angle brackets	**-**	**A5NP+T**	**A7NP+T**	**-**	**B5NP+T**	**B7NP+T**
X	X-RAD	X-RAD	**A3X**	**A5X**	**-**	**B3X**	**B5X**	**-**
X+T	X-RAD and steel ties	X-RAD	**-**	**A5X+T**	**A7X+T**	**-**	**B5X+T**	**B7X+T**
T	Steel ties	Angle brackets	**-**	**A5T**	**A7T**	**-**	**B5T**	**B7T**

3.3. Numerical models

The numerical models of the buildings were developed and analyses were performed using the SAP2000 Finite Element code [32]. The following finite elements were used: two-dimensional elastic shell elements to model the CLT panels; uniaxial linear elements to simulate the actual stiffness of the connections; one-dimensional truss elements to simulate the steel ties. Fig. 4 shows the models of the shear walls, performed with the characteristics hereafter summarized.

In the configurations H and NP+T (Fig. 4 a,c), the connections have been modelled with uniaxial elements according to the component approach presented in [4]. It can be noted that, according to the technology presented in Section 2.1, in the model NP+T (Fig. 4 c), the tensile forces due to rocking at each storey level are firstly resisted by the nailed plates and then introduced in the steel tie and directly transferred to foundation. The values to be assigned to these models are: the axial stiffness of the hold-down $k_{H,i}$ or of the nailed plate $k_{NP,i}$; the shear stiffness of the element representing all the angle-brackets in a storey $k_{A,i}$; the Modulus of Elasticity E_T and the cross-section area A_T of the truss elements representing the steel ties. In the configurations X and X+T (Fig. 4 b,d), the X-RAD system that connects two panels at the same corner is represented by an assembly of four uniaxial elements: the two vertical elements simulate the stiffness at 90° to the horizontal ($k_{X,v}$); the other two elements the stiffness at 0° to the horizontal ($k_{X,h}$). The steel tie is represented also in this case with E_T and A_T. Finally, in the configurations T (Fig. 4 e), the steel ties are directly fixed to the top of each panel, for all the storeys, in order to transfer directly the force from the wall to the tie, according to the technology described in Section 2.3.

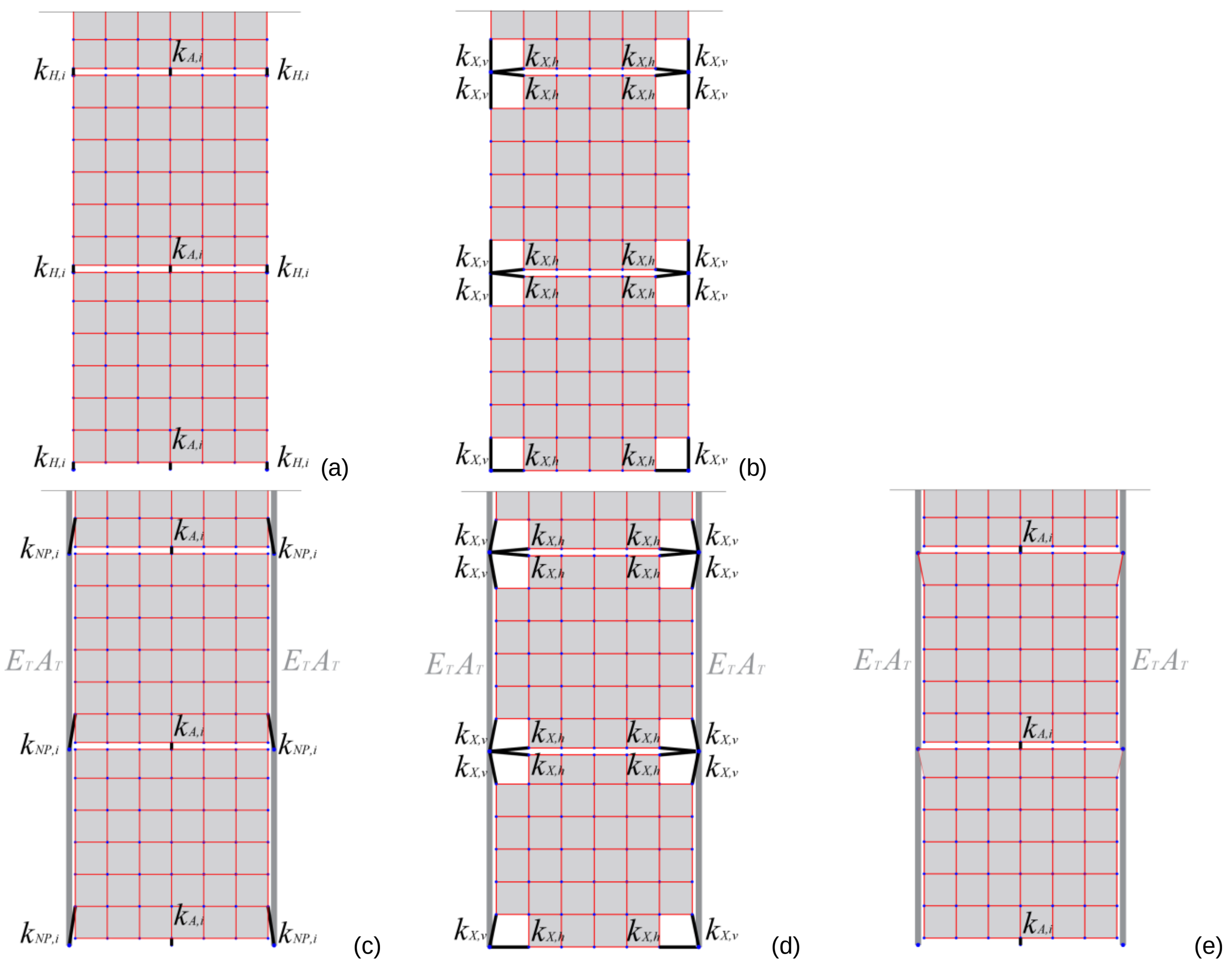


Fig. 4 – Numerical models of the shear walls. Configurations with: (a) hold-downs “H”; (b) X-RAD “X”; (c) nailed plates and steel ties “NP+T”; (d) X-RAD and steel ties “X+T”; (e) steel ties “T”

3.4. Design of the shear walls and stiffness of the connection systems

The shear walls were designed performing iterative LDA [26], assuring, for each iteration, the consistency between strength and stiffness of connections in the model, up to fulfilment of verifications for Ultimate Limit State (ULS) and Damage Limitation State (DLS) [33]. As anticipated in Section 3.2, H and X configurations for the five-storey buildings did not fully comply with these verifications and the strengthening with the ties was required. Nevertheless, the results are anyway shown also for these configurations for comparative purposes.

The site location of Perugia in the region of Umbria has been chosen because it is representative of a high seismic hazard zone in Italy. It is characterised by the seismic frequency spectra reported in Fig. 5, according to Italian Regulation for Constructions [27], assuming building foundations resting on ground type C and behaviour factor *q* equal to 1.5 or 2.0. A *q* equal to 2.0 was assumed for all the non-strengthened configurations (H and X), according to a dissipative structural behaviour in medium ductility class [26]. For the configurations with vertical steel ties (NP+T, X+T and T), it was assumed equal to 1.5. As mentioned above, the choice of designing these configurations according to a low-dissipative structural behaviour [26] was made since a reduced dissipative capacity is expected by using steel ties, which represent the leading earthquake-resistant system in the building. The following connections have been considered in the models:

- Hold-downs WHT340 with 20 nails or WHT620 with 52 nails [34];
- Angle brackets TTF200 for floor-wall joints or TCF200 for wall-to-foundation joint, with 30 nails [35];
- 4-mm thick nailed plates with 12 nails (NP12) or 26 nails (NP26);
- X-RAD [36].

Table 3 lists strength and stiffness values assumed for connections, according to values given in [4] for hold-downs and angle brackets and in [29] for X-RAD. Strength and stiffness of nailed plates were obtained multiplying the values available in [37] relative to a steel-timber connection with one 4×60-mm ring-shank nail, by twelve or twenty-six nails for NP12 and NP26 respectively. The steel ties were assumed to be realized with rectangular steel plates (or alternative shapes such as L or UPN profiles), realized in class S355, Modulus of Elasticity E_T equal to 210000 MPa [38].

The iterative design procedure resulted in the arrangement of connections listed in Table 4 for the configurations H, NP+T and T. With reference to the configurations X and X+T, one X-RAD per panel corner was assumed for all configurations and storeys, with the exception of the 7-storey shear walls (A7X+T and B7X+T), for which a couple of X-RAD was added for all the storeys at the middle of the panel base and panel top, to increase the shear strength and comply with the seismic demand. For all the configurations with steel ties, an overall cross-section area of the ties A_T equal to 1500 mm^2 was assumed for the 5-storey shear wall, and equal to 2250 mm^2 for the 7-storey shear wall.

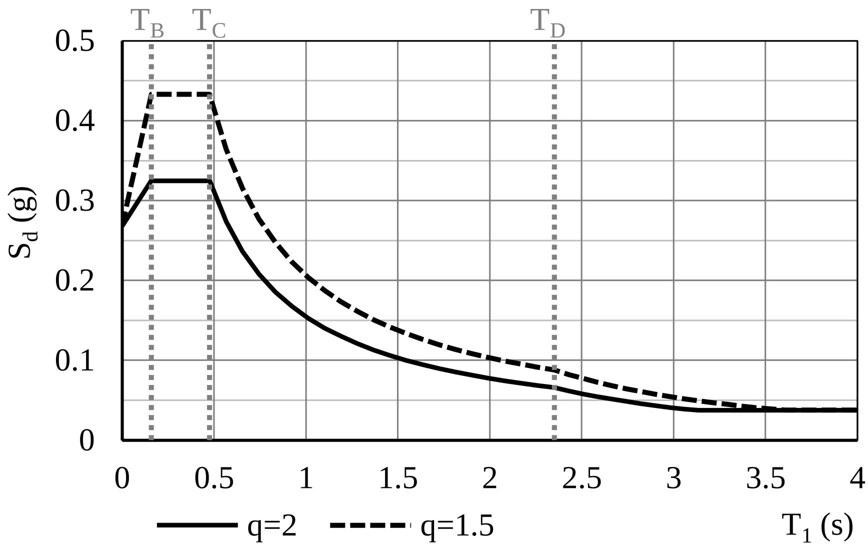


Fig. 5 – Design seismic spectra of Perugia in Italy, ground type C, behaviour factors *q* equal to 1.5 or 2

Table 3 – Strength and stiffness of the earthquake-resistant connections

	Rocking				Sliding		Rocking and sliding
Metal connector	WHT340 hold-down [4]	WHT620 hold-down [4]	NP12 nailed plate [37]	NP26 nailed plate [37]	TTF200 angle bracket [4]	TCF200 angle bracket [4]	X-RAD [29]
Elastic stiffness (kN/mm)	5.70	13.25	5.80	12.58	8.21	8.48	15.2 (0°/90°) 17.3 (180°/270°)
Strength (kN)	46.20	93.70	41.58	90.09	39.10	39.10	*Capacity domain*

Table 4 – Arrangement of the earthquake-resistant connections per panel of the multi-storey shear wall. Configurations H, NP+T and T

Storey	1	2	3	4	5	6	7
A3H	4 WHT620 3 TCF200	4 WHT340 3 TTF200	2 WHT340 2 TTF200	–	–	–	–
A5H	4 WHT620 3 TCF200	4 WHT620 3 TTF200	4 WHT340 2 TTF200	4 WHT340 2 TTF200	2 WHT340 2 TTF200	–	–
A5NP+T	4 NP26 5 TCF200	2 NP26 5 TTF200	2 NP26 4 TTF200	2 NP12 3 TTF200	2 NP12 2 TTF200	–	–
A5T	5 TCF200	5 TTF200	4 TTF200	3 TTF200	2 TTF200	–	–
A7NP+T	4 NP26 7 TCF200	4 NP26 6 TTF200	4 NP26 6 TTF200	4 NP26 5 TTF200	2 NP26 4 TTF200	2 NP26 3 TTF200	2 NP12 2 TTF200
A7T	7 TCF200	6 TTF200	6 TTF200	5 TTF200	4 TTF200	3 TTF200	2 TTF200
B3H	4 WHT620 3 TCF200	4 WHT340 3 TTF200	2 WHT340 2 TTF200	–	–	–	–
B5H	4 WHT620 3 TCF200	4 WHT620 3 TTF200	4 WHT340 2 TTF200	4 WHT340 2 TTF200	2 WHT340 2 TTF200	–	–
B5NP+T	4 NP26 7 TCF200	2 NP26 6 TTF200	2 NP26 5 TTF200	2 NP12 4 TTF200	2 NP12 3 TTF200	–	–
B5T	7 TCF200	6 TTF200	5 TTF200	4 TTF200	3 TTF200	–	–
B7NP+T	4 NP26 8 TCF200	4 NP26 7 TTF200	4 NP26 7 TTF200	4 NP26 6 TTF200	2 NP26 5 TTF200	2 NP26 4 TTF200	2 NP12 3 TTF200
B7T	8 TCF200	7 TTF200	7 TTF200	6 TTF200	5 TTF200	4 TTF200	3 TTF200

To compare the configurations in terms of resulting amount of connections per panel (Table 4), it is necessary to remember that the strengthening with the steel ties leads to an increase of the seismic demand due to an increase of stiffness and consequent decrease of periods of vibration and due to the lower behaviour factor used, Fig. 5. The consequent increase of strength demand clearly results in an increase of the number of angle-brackets per storey to resist higher shear forces, as can be noted by comparing results among the five-storey configurations. However, despite this apparent disadvantage, the addition of steel ties actually leads to a significant advantage in terms of rocking behaviour. Contrary to the non-strengthened five-storey buildings, both ULS and DLS verifications are fulfilled for all the five- and seven-storey NP+T and X+T configurations. Finally, the T configuration seems to be the most effective strategy in terms of optimization in the number of connections.

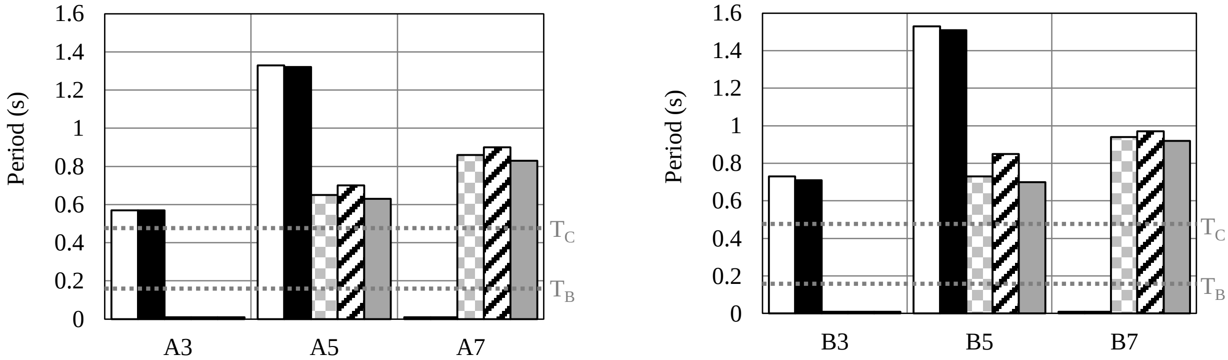
4. Results

In this Section, results from LDA are summarized and discussed in terms of fundamental periods of vibration, tensile forces at connections, inter-storey drifts and displacements.

4.1. Fundamental periods of vibration

Fig. 6 shows the results of the analyses in terms of fundamental period of vibration of the structure. The periods T_B and T_C of the seismic spectra defined by Eurocode 8 [26] (see Fig. 5) are also represented to identify the limit of the constant spectral acceleration branch (plateau).

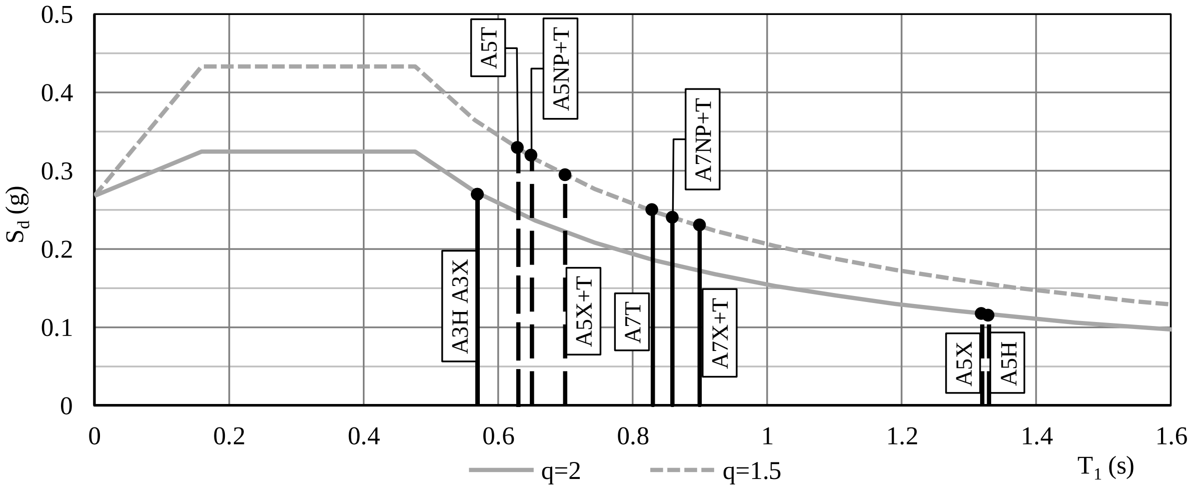
The configurations without steel ties (H and X) for both the buildings show similar fundamental periods of vibrations. The use of steel ties combined or not with the nailed plates and X-RAD (NP+T, X+T, T) increases strongly the global stiffness of the shear walls, resulting in a substantially lower fundamental period and a higher spectral acceleration. This is evident comparing results for the five-storey configurations in Fig. 7 (dashed black lines), where the fundamental periods of vibration have been superimposed to the design spectra.



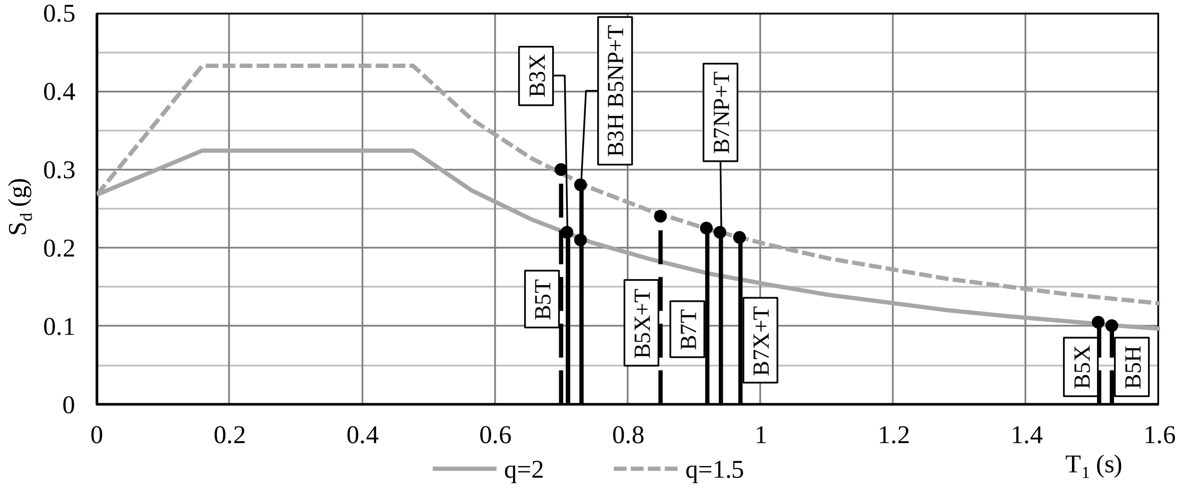
	H □	X ■	NP+T	X+T	T
A3	0.57	0.57	-	-	-
A5	1.33	1.32	0.65	0.70	0.63
A7	-	-	0.86	0.90	0.83

	H □	X ■	NP+T	X+T	T
B3	0.73	0.71	-	-	-
B5	1.53	1.51	0.73	0.85	0.70
B7	-	-	0.94	0.97	0.92

Fig. 6 – Fundamental periods of vibration. Units: s



(a)



(b)

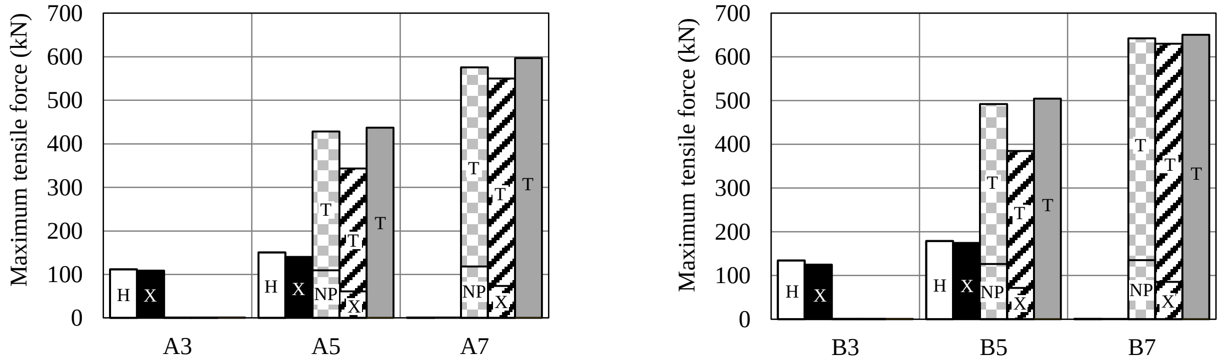
Fig. 7 – Fundamental periods of vibration and design spectra. (a) Building A; (b) Building B

4.2. Tensile forces

Fig. 8 shows the results of the analyses in terms of maximum tensile forces due to rocking. The maximum values were recorded always in the connections at the base of the shear walls. The tensile force is showed for the rate resisted by the hold-downs (H), by the X-RAD (X), by the nailed plates (NP) and by the steel tie (T).

The comparison between the configurations without steel ties (H and X) for both the buildings shows similar forces. It is worth noting that for the 5-storey shear walls, very high tensile forces in the hold-downs and X-RAD have been recorded, reaching about 180 kN in the hold-downs of the B5H configuration, which is very close to the limit of about 187 kN of a couple of WHT 620 in the wall end, see Table 3. Despite the higher spectral acceleration and consequent higher inertial forces due to the lower period of vibration and to the lower behaviour factor assumed, the strengthening with the steel ties allows to reduce the forces in the connections, and to have the possibility of increasing the number of storeys.

The T configurations behave similarly to the NP+T ones. The maximum tensile force at the base is almost the same, but in the case of the T configurations, the force must be resisted entirely by the tie. Conversely, in the NP+T and X+T configurations, tension in the ties is lower, due to the contribution of nailed plates and X-RAD at the base of the building, which are directly anchored to the foundation. The T configuration seems more effective than the NP+T and X+T configurations, since only the ties are used, avoiding additional nailed or screwed connections to withstand tensile forces. Nevertheless, the combined configurations may become further interesting, when used in combination with dissipative device (e.g., [19–25]). In this way, the dissipators permit to reduce at each storey the inertial force transmitted from the panels to the tie. It is worth recalling also the possibility of pre-assembling the connections to the panels, making the X+T configuration particularly advantageous in terms of prefabrication and fast installation of the ties to the structure, see Section 2.2.



	H □	X ■	NP+T	X+T	T
A3	112.1	109.0	-	-	-
A5	150.7	140.3	428.1 *NP 109.2* *T 318.9*	343.3 *X 61.5* *T 281.8*	436.9
A7	-	-	575.5 *NP 118.3* *T 457.2*	549.8 *X 73.4* *T 476.4*	597.3

	H □	X ■	NP+T	X+T	T
B3	134.2	125.0	-	-	-
B5	179.2	174.7	492.4 *NP 126.2* *T 366.2*	384.7 *X 71.5* *T 313.2*	504.3
B7	-	-	642.8 *NP 135.1* *T 507.7*	630.3 *X 85.3* *T 545.0*	650.4

Fig. 8 – Maximum tensile forces at hold-downs, nailed plates, X-RAD connections and steel ties. Units: kN

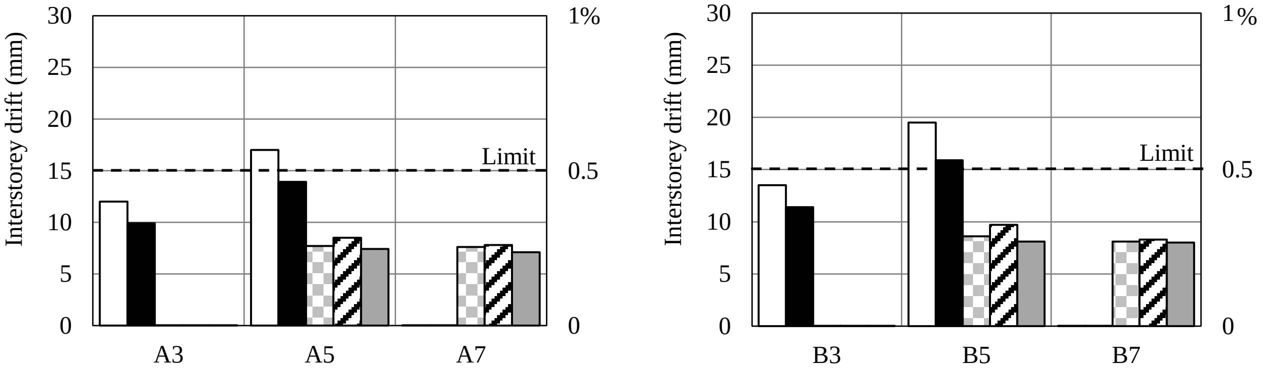
4.3. Displacements and inter-storey drifts

Fig. 9 and Fig. 10 show the results of the analyses in terms of maximum inter-storey lateral drift and maximum top displacement of the shear walls.

The configurations without steel ties (H and X) for both the buildings show similar top displacements, with an inter-storey drift slightly higher for the H configurations, mainly due to the greater rocking deformation. In these configurations, the global lateral displacements are mainly due to rocking for about an 80% rate and the contribution of shear deformations is limited to about 20%.

It can be noted that the use of the steel ties and the design of the structure according to a low-dissipative structural behaviour, result in limiting the lateral displacements of the building, and therefore in limiting the damage to structural and non-structural components.

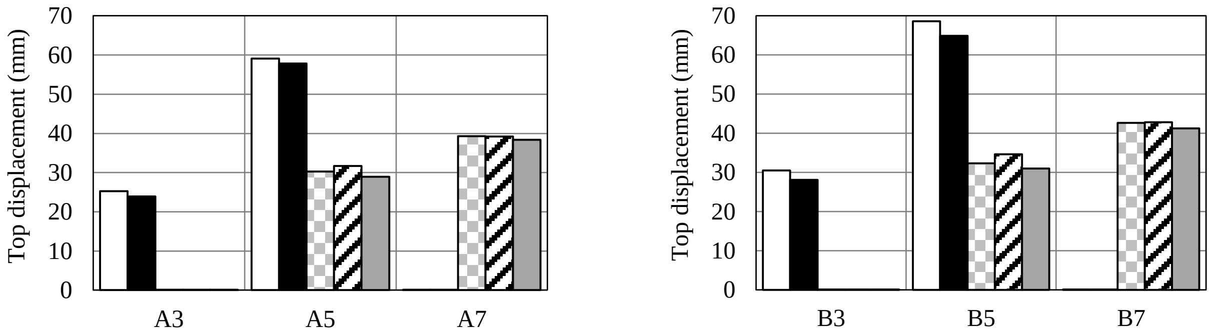
The 5-storey shear walls analysed in this work demonstrate that the addition of the steel ties almost halves the top displacements and inter-storey drifts, and, also for the 7-storey shear walls, displacements and drifts are smaller than the 5-storey configurations without ties and comply fully with the limit of 0.5% in Eurocode 8 [26]. In these strengthened configurations, shear deformations take on greater importance and contribute for about 50% to the global lateral displacements.



	H □	X ■	NP+T	X+T	T
A3	12.0	9.9	-	-	-
A5	17.0	13.9	7.7	8.5	7.4
A7	-	-	7.6	7.8	7.1

	H □	X ■	NP+T	X+T	T
B3	13.5	11.4	-	-	-
B5	19.5	15.9	8.6	9.7	8.1
B7	-	-	8.1	8.3	8.0

Fig. 9 – Maximum inter-storey drifts and DLS limit. Units: mm



	H □	X ■	NP+T	X+T	T
A3	25.3	23.9	-	-	-
A5	59.1	57.8	30.3	31.7	28.9
A7	-	-	39.3	39.2	38.4

	H □	X ■	NP+T	X+T	T
B3	30.5	28.1	-	-	-
B5	68.6	64.9	32.3	34.6	31.0
B7	-	-	42.6	42.8	41.2

Fig. 10 – Maximum top displacements. Units: mm

4.4. Effects of optimizing the capacity of steel ties per storey

In order to optimize the steel ties, the cross-section can be gradually reduced from the foundation to the top of the building, keeping almost constant among the storeys the ratio between the tensile strength demand to the capacity of the ties; this ratio is hereafter called "working rate".

To evaluate the influence of such optimization on the fundamental period of vibration, tensile forces and displacements, additional LDA were carried out for the strengthened configurations of the 5- and 7-storey Building A. The optimized configurations are marked with a star; the additional configurations are therefore: A5NP+T*; A5X+T*; A5T*; A7NP+T*; A7X+T*; A7T*. The cross-section areas of the ties resulting from the design and the relative working rates are listed in Table 5 and compared with the non-optimized counterparts. It is worth noting that the optimized design resulted in an important reduction of the cross-section of the ties for the upper storeys. Conversely, the number of the other connection elements (nailed plates, angle brackets and X-RAD), remained unchanged with respect to those of Table 4 and negligible variations occurred in terms of periods (below 12%) and therefore in forces acting in connections (below 12%), as can be seen in Table 6.

Finally, the decrease in the overall stiffness of the optimized structures resulted in a negligible increase of inter-storey drifts (below 15%) and top displacements (below 12%).

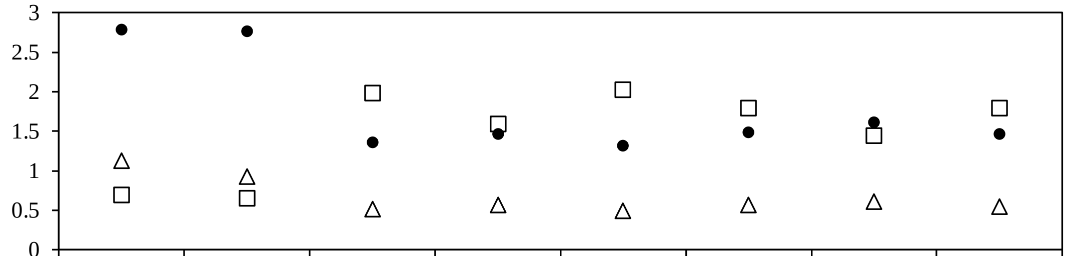
A final comparison among all the analysed configurations for the 5- and 7-storey Building A is given in Fig. 11. Three dimensionless parameters have been introduced, namely: the period index τ equal to T_1/T_c that shows how far the fundamental period of vibration is from the plateau; the tensile force index ϕ equal to $F_{max}/(M \cdot PGA)$; and the drift index δ equal to the actual drift divided by 0.5%. In these parameters, T_1 is the fundamental period of vibration, T_c is equal to 0.476 s, F_{max} is the maximum tensile force, M is the total seismic mass relative to the shear wall according to Table 1, PGA is the peak ground acceleration equal to 2.61 m/s^2, 0.5% is the drift limit for DLS.

Table 5 – Cross-section areas A_T of the steel ties, units mm^2. In brackets the working rate of the steel ties

Storey	1	2	3	4	5	6	7
A5NP+T	1500 (60%)	1500 (60%)	1500 (38%)	1500 (21%)	1500 (8%)	-	-
A5NP+T*	1500 (54%)	1500 (54%)	950 (53%)	550 (53%)	200 (48%)	-	-
A5X+T	1500 (53%)	1500 (53%)	1500 (35%)	1500 (21%)	1500 (10%)	-	-
A5X+T*	1500 (48%)	1500 (48%)	1000 (47%)	600 (49%)	300 (46%)	-	-
A5T	1500 (82%)	1500 (58%)	1500 (38%)	1500 (21%)	1500 (7%)	-	-
A5T*	1500 (74%)	1100 (68%)	700 (67%)	400 (68%)	150 (65%)	-	-
A7NP+T	2250 (58%)	2250 (58%)	2250 (44%)	2250 (33%)	2250 (22%)	2250 (12%)	2250 (6%)
A7NP+T*	2250 (55%)	2250 (55%)	1750 (54%)	1300 (54%)	850 (56%)	500 (50%)	200 (48%)
A7X+T	2250 (60%)	2250 (60%)	2250 (47%)	2250 (35%)	2250 (24%)	2250 (14%)	2250 (7%)
A7X+T*	2250 (56%)	2250 (56%)	1750 (56%)	1300 (55%)	900 (55%)	600 (48%)	250 (49%)
A7T	2250 (75%)	2250 (65%)	2250 (50%)	2250 (36%)	2250 (23%)	2250 (13%)	2250 (5%)
A7T*	2250 (70%)	2000 (69%)	1500 (70%)	1100 (68%)	700 (70%)	400 (67%)	150 (66%)

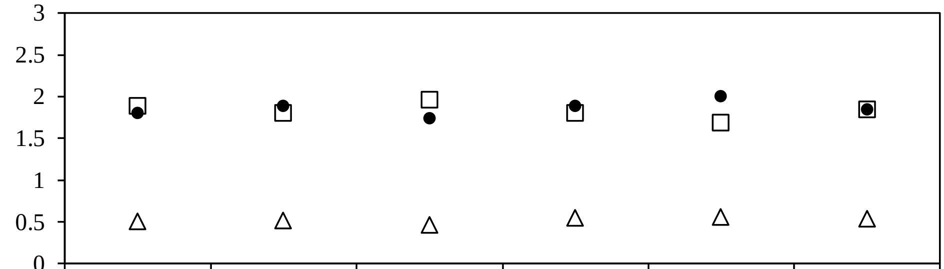
Table 6 – Results for the optimized configurations and percentage change from the non-optimized counterparts

	Period (s)		Maximum Tensile Force (kN)		Top displacement (mm)		Inter-storey drift (mm)	
A5NP+T*	0.71	*9.23%*	385.6	*-9.93%*	32.8	*8.25%*	8.6	*11.69%*
A5X+T*	0.77	*10.00%*	312.2	*-9.06%*	34.0	*7.26%*	9.2	*8.24%*
A5T*	0.70	*11.11%*	387.2	*-11.38%*	31.4	*8.65%*	8.3	*12.16%*
A7NP+T*	0.90	*4.65%*	549.7	*-4.48%*	43.1	*9.67%*	8.3	*9.21%*
A7X+T*	0.96	*6.67%*	514.2	*-6.48%*	43.6	*11.22%*	8.5	*8.97%*
A7T*	0.88	*6.02%*	560.9	*-6.09%*	42.8	*11.46%*	8.1	*14.08%*



	A5H	A5X	A5NP+T	A5X+T	A5T	A5NP+T*	A5X+T*	A5T*
● Period index τ	2.79	2.77	1.36	1.47	1.32	1.49	1.62	1.47
□ Force index ϕ	0.70	0.65	1.99	1.60	2.03	1.80	1.45	1.80
∆ Drift index δ	1.13	0.93	0.51	0.57	0.49	0.57	0.61	0.55

(a)



	A7NP+T	A7X+T	A7T	A7NP+T*	A7X+T*	A7T*
● Period index τ	1.81	1.89	1.74	1.89	2.02	1.85
□ Force index ϕ	1.90	1.81	1.97	1.81	1.69	1.85
∆ Drift index δ	0.51	0.52	0.47	0.55	0.57	0.54

(b)

Fig. 11 – Period, force and drift index for the 5- and 7-storey configurations of Building A. (a) 5 storeys; (b) 7 storeys

The comparison in Fig. 11 for the 5-storey building confirms the substantial decrease of fundamental periods of vibration and the increase of tensile forces for the strengthened shear walls. The improvement is evident in terms of DLS verification, confirming that it is fulfilled for all the strengthened configurations of the 5- and 7-storey shear walls. On the contrary, δ is higher than 1 for the A5H configuration, confirming that the DLS verification is not fulfilled and stiffening of the shear wall is needed. Finally, the results for the optimized configurations are confirmed to be similar to the non-optimized ones, with slight decrease of lateral stiffness and consequent increase of inter-storey drift.

5. Conclusions

Three earthquake-resistant systems for timber shear-wall structures have been presented, based on the use of vertical steel ties as alternative or in addition to traditional nailed plates or screwed connections. The proposed technologies are intended for multi-storey CLT buildings and have the aim of withstanding high tensile forces due to rocking of the timber panels, transmitting them directly from each storey level to the concrete foundation. At the same time, the ties stiffen the structure, reducing lateral displacements and inter-storey drifts. The fastening of the ties to the panels at each storey level can be realized with traditional nailed plates or screwed connections. As an alternative to the use of dowel-type fasteners, the ties can be directly secured to thick steel plates, placed in direct contact to the top narrow side of each wall panel.

Previous studies available in the literature proved that the main issues of multi-storey CLT buildings subjected to seismic action are the limited lateral stiffness and the high tensile forces concentrated in hold-down connections, which become the critical components of the building. The parametric dynamic analyses presented in this work have confirmed this conclusion and have demonstrated that the proposed use of steel ties to realize high-strength shear walls can address both these issues, improving the feasibility of multi-storey CLT buildings in high-seismicity areas. This strategy makes also possible a reduction of the number of earthquake-resistant walls and connections in the building.

The parametric analyses have shown that the spectral accelerations increase by adding the ties to the structure, due to an increase in the global stiffness of the shear walls and consequent decrease of periods of vibration. Nevertheless, the tensile forces in the connections significantly decrease, being partially or totally resisted by the steel ties, depending on the fastening strategy adopted. Results in terms of top displacements and inter-storey drifts have demonstrated that the use of steel ties improves significantly the stiffness and the elastic response of the building, with clear advantages in complying with requirements for damage limitation state.

Finally, the reduction of the cross-section area of the ties from the foundation to the top of the building according to the seismic demand leads to the optimization of the structural material without affecting the response of the building.

The earthquake-resistant systems presented in this work have proven a good seismic performance, making them worthy of further developments and research. Additional analyses can be performed to investigate the possible coupling of the proposed systems with ductile devices, able to dissipate energy, in order to reduce the inertial forces introduced in the steel ties at each floor level.

Acknowledgements

The authors would like to thank STP s.r.l. company for founding the research and Dr. Francesca Paoloni for the cooperation in the graphic representations of the technology.

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