Connections for steel-timber hybrid prefabricated buildings.
Part II: Innovative modular structures

Construction and Building Materials

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Abstract: This paper aims at the development of multi-storey prefabricated modular buildings. Particularly, this document deals with a novel versatile construction system in which the main structural members, made by combining timber with steel, are highly engineered and can be produced in the factory. The research demonstrates the potential of steel-timber hybrid structures in terms of sustainability, providing lightweight modern seismic-resistant constructions. This document presents the results of a comprehensive experimental campaign, including tests on several innovative connections, and of the numerical FEM analyses of the structural components. The paper also provides prototypes of new highly-industrialized steel-timber hybrid shear wall and floor components.

Keywords: Steel-timber connections; Hybrid structures; Composite beams; Wood-based structures; Modular constructions; Prefabrication; Sustainability; Shear walls.

1. Introduction

The main structural systems used to build timber buildings are typically based on panel construction elements, whether they are made of lightweight wood frame panels [1] or cross-laminated timber panels, (CLT) [2]. In both these systems, structural panels can simultaneously have a load-bearing function and be considered as the building envelope. Such systems can be easily built using prefabricated modular elements, produced in a factory, and subsequently joined on site using mechanical connections. Even though the industrial manufacturing quality, construction methods and joining technologies have evolved in recent years, there are several structural outstanding issues that must be resolved for the construction of modern timber buildings. Structural problems are mainly associated to the compliance with current performance-based code requirements, as for instance in case of fire [3,4], earthquakes [5] or acoustic and vibration serviceability ([6,7]). These and other technical issues have pushed designers to find new structural hybrid-type construction systems, obtained using wood in conjunction with other structural materials, such as concrete and steel. In Europe for example some symbolic buildings have recently been constructed under pilot projects [8-10] and other structures are under construction [11].

Hybrid construction systems can be of strategic importance for the high-rise building sector ([12,13]). In particular, the advent of hybrid wood-based construction systems has moved the interest towards the research for practical, sustainable, and above all, energy-efficient solutions, to compete with the most widely used structural systems assembled using traditional materials. In this work buildings are erected adopting an innovative steel-timber hybrid-based construction system, developed under an exclusive industrial research programme. As demonstrated in [14], different levels of collaboration between wood and steel may define different construction systems, which differ in either the construction elements or in parts of the structure. In other words, a wide range of steel-timber hybrid-based construction systems can be designed. The 'Linea Nova' building in Rotterdam [15] is an example of a hybrid system with materials which vary according to the height, with the first four storeys of concrete and the other sixteen built using steel and timber members. The 'Hybrid timber-steel retail structure' [16] is an example of a commercial building realized in England with columns and bracing systems made of steel and wooden beams. Similarly, the Japanize 'Kanazawa M. Building' [17] represents an example of full hybridization of the construction system. Pillars and other construction elements are composite elements made of wood and steel. The last building typology here considered is 'Scotia Place' [18] in New Zealand, realized by joining steel frames with timber diaphragms. The 12-storey building is stabilized by concentric diagonal bracings and the floors are produced by joining together laminated timber decks. Although the use of hybrid structures is rapidly increasing, the current knowledge is still too limited to exploit all the potential benefits and further research needs to be carried out in order to develop more reliable and well-engineered solutions. This paper deals with a contemporary integrated and sustainable construction technology for new residential buildings. In particular, this work concerns a new hybrid construction system that allows a quicker assembly of the construction elements, mainly prefabricated in the factory, reducing the time of on-site assembling operations and the construction costs. Under these conditions, the component manufacturing is highly industrialized and permits the building processes to take place also in areas exposed to harsh weather conditions. The proposed hybrid system is innovative not just in adopting modern engineered wood products, such as CLT panels, but also in using modular standardized construction elements: composite steel-timber floors and mixed steel-timber shear walls. In addition, the connections between the steel elements and the CLT panels have been specifically developed under this research. This structure offers structural benefits in terms of load-carrying and deformation capacity, as well as being inherently lightweight, which minimizes the effects induced by earthquakes.

The paper describes the reference building and its construction system in Section 2. This Section also discusses the advantages offered by composite steel-timber floors compared to other common technologies. Section 3 presents some innovative connections designed to develop structural interaction between materials, providing solutions for composite floors and hybrid-based bracing walls. The structural behaviour of the connections, evaluated via experimental tests, is also reported in this Section. Section 4 shows three different numerical studies carried out for the evaluation of the ductile behaviour of hybrid shear walls and the in- and out-of-plane structural capacity of the composite floors. For each study we present the engineered connection solutions recommended to ensure both that the building is quick to assemble on site and that composite elements can be easily prefabricated. Section 5 reports on the prototyping of steel-timber composite floor and mixed bracing wall components to build earthquake-resistant structures. Conclusions and recommendations for future research then follow in Section 6.

2. Innovative prefabricated buildings

Figure 1 shows a three-dimensional view of the reference building. The structure is built by combining steel frames and beams with cross-laminated timber panels (CLT), in order to develop a construction system with excellent performance and architectonic flexibility. The structure effectively exploits both the highly industrialized technology typical of steel construction systems and the advantages offered by the use of solid wood-based panels, such as lightness, structural in-plane stability, and low environmental impact, as well as the possibility to recycle and quickly replace degraded elements. The construction system is modular and easily repeatable in space, making the fabrication of new residential complexes possible in a short time.

Referring to Figure 1, the main force-resisting system (FRS) consists of a three-dimensional collection of steel and timber elements arranged horizontally and vertically. The gravity loads flow from the floors to the frames, loading first the secondary beams and then transferring relative vertical forces into the columns. These forces are later downloaded to the foundation. For the horizontal loads, each floor acts as a truss system transmitting the forces from the point of origin to the vertical bracing systems, which here are hybrid steel-timber walls placed in both main directions of the building. In the truss system, each steel secondary beam is braced by the CLT panels and their related beam-to-panel and panel-to-panel connections (see Part I) [19]. The steel beams also perform a stabilizing function preventing any possible out-of-plane instability of the CLT panels. The global behaviour of the whole system is mainly guided by the steel frames under gravity loads, while towards seismic and wind forces the resistant mechanism is governed by the interaction between the diaphragms (DIAs) and the shear walls (SWs). Both DIAs and SWs are assembled using modular prefabricated components manufactured via two novel steel-timber hybrid-based technologies, developed within this research.

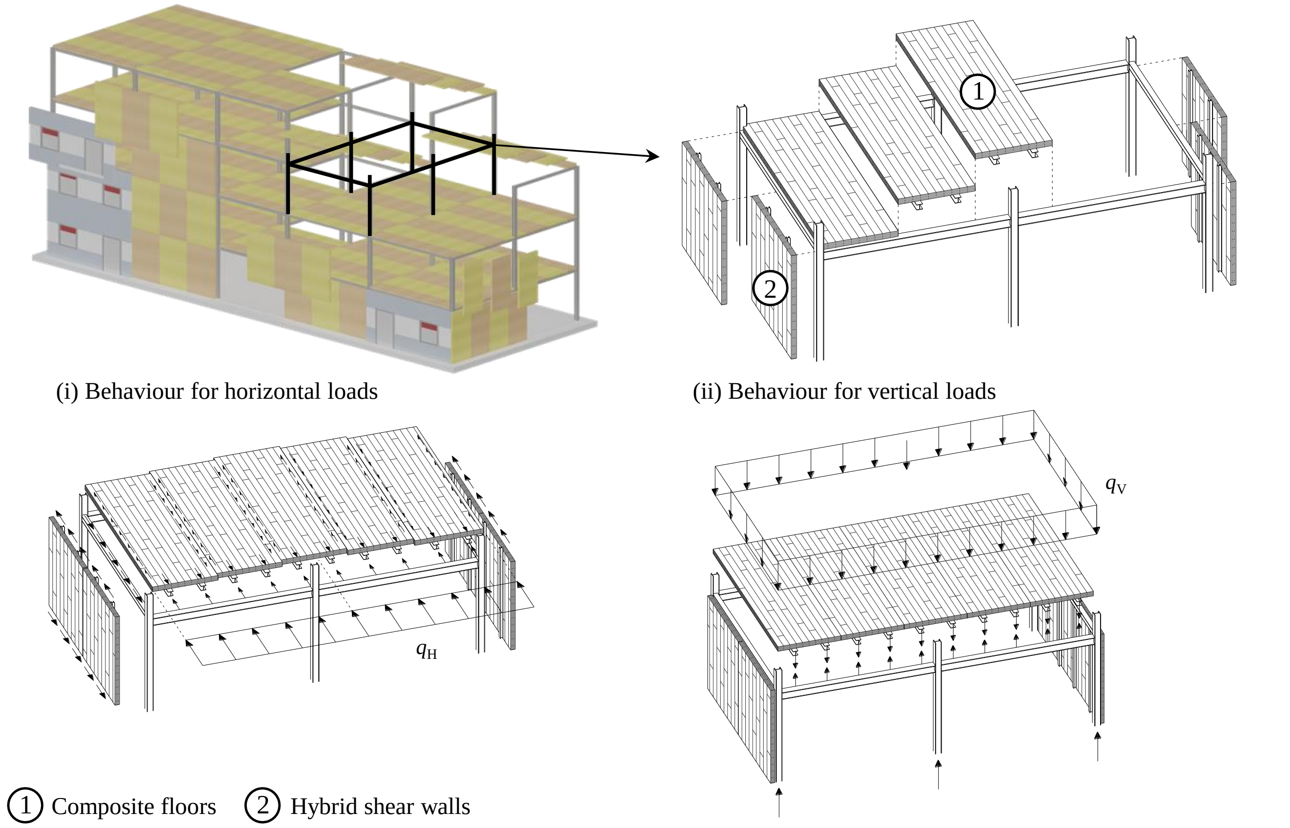


Fig. 1. Axonometric view of the building and exploded view of the hybrid structure with load path depicted for the vertical and horizontal loads.

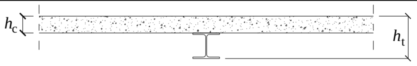
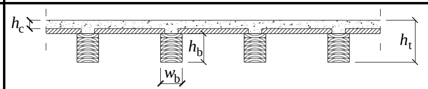
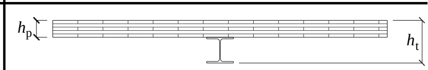
2.1 Composite steel-timber floors and their impact in the building market

As part of this innovative hybrid steel-timber prefabricated construction system, composite steel-timber floors with CLT slabs can be considered a new generation of structural technologies, especially for the realization of lightweight modern buildings. The structural efficiency of these composite steel-timber elements is remarkable, especially if compared to other traditional steel-concrete or timber-concrete composite technologies. This Section outlines some of the main results obtained from a comparative analysis that considers the flexural deformation and the load-carrying capacity of three different floors. In Table 1, the floor sections have been dimensioned [20,21] considering similar design requirements, expressed in terms of span (l), cross-section height (h_t), design loads (q_d) and serviceability deflection limits (l/250). Table 1 displays the estimated structural performance of the floors under two different assumptions: without composite action (connections stiffness k equal to 0) and with fully composite action (connections stiffness k equal to ∞). The design models used to evaluate the load-carrying capacity of the floors have been taken from Eurocodes [22-25], adopting characteristic values for the strength properties of materials. Table 1 also includes the capacity-to-self-weight ratio ($\eta_q = Q_{R\text{-}SLU}/Q_P$), to be used in the evaluation of the structural efficiency and also useful in the preliminary estimation of the costs.

Table 1 demonstrates that composite steel-timber hybrid-base floors with CLT slabs have similar capacity compared with other technologies based on reinforced concrete (RC) slabs. The bearing capacity $Q_R^{(k=\infty)}$ ranges from 20.28 up to 26.52 kN/m^2. However, the capacity-to-self-weight ratio (η_q) of the steel-timber composite sections is remarkably higher compared with that from the other solutions which use RC slabs. In a more general sense, steel-timber composite systems provide the advantages of each material creating very slim light floor components, and therefore leading to lightweight constructions and reducing the use of raw materials. This provides multiple benefits, cutting-down the forces acting on the foundation and similarly reducing the seismic effects on the structure. In addition, the balanced use of natural and recyclable materials, mean that this kind of structural system supports sustainability in construction.

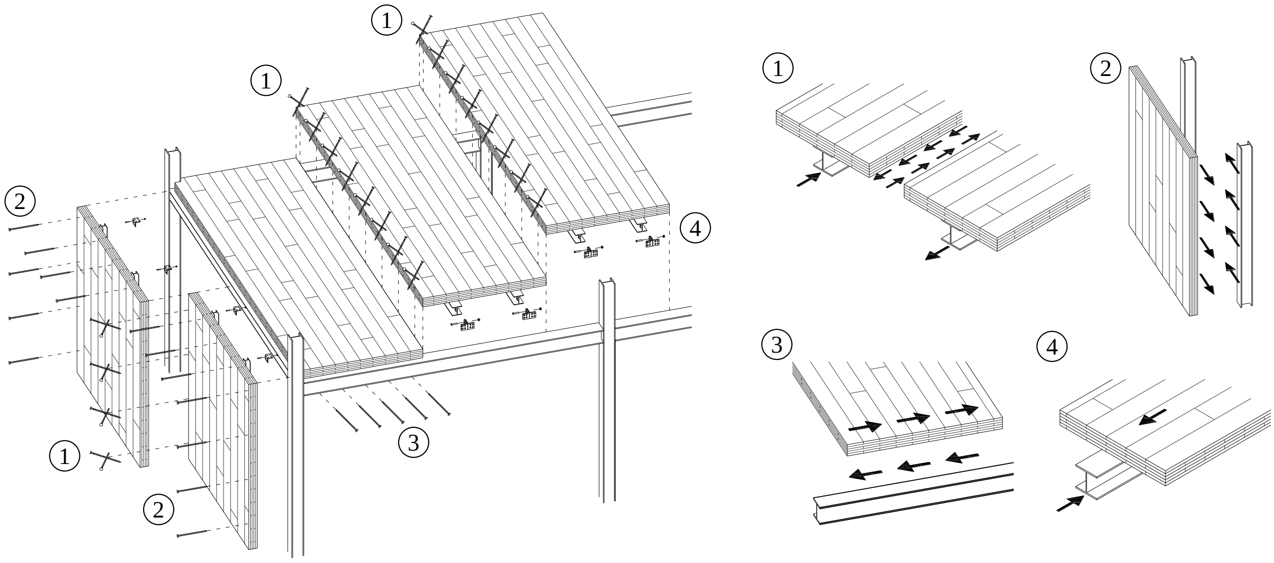
Table 1

Composite floors: comparison of three different types of constructions.

Steel-concrete composite system		**Timber-concrete composite system**		**Steel-timber composite system**	
					
Materials		**Materials**		**Materials**	
Steel beam grade [26]	S275	Timber beam grade [27]	GL24h	Steel beam grade [26]	S275
Concrete grade [22]	C25/30	Concrete grade [22]	C25/30	Timber boards grade [28]	C24
Geometry features of the beam		**Geometry features of the beam**		**Geometry features of the beam**	
Section profile (European section)	HEA160	Height h_b	170 mm	Section profile (European section)	HEA160
		Width w_b	130 mm		
Geometry features of the slab		**Geometry features of the slab**		**Geometry features of the slab**	
Depth h_c	100 mm	Depth h_c	50 mm	Depth h_p	5x20 mm
Structural main frame		**Structural main frame**		**Structural main frame**	
Free span length	6 m	Free span length	6 m	Free span length	6 m
Beam spacing	2 m	Beam spacing	0.5 m	Beam spacing	2 m
Cross section height h_t	252 mm	Cross section height h_t	250 mm	Cross section height h_t	252 mm
Bending stiffness		**Bending stiffness**		**Bending stiffness**	
$EJ^{(k=0)}$	$3\cdot10^{12}$ Nmm^2	$EJ^{(k=0)}$	$7\cdot10^{11}$ Nmm^2	$EJ^{(k=0)}$	$4.5\cdot10^{12}$ Nmm^2
$EJ^{(k=\infty)}$	$1.2\cdot10^{13}$ Nmm^2	$EJ^{(k=\infty)}$	$2.6\cdot10^{12}$ Nmm^2	$EJ^{(k=\infty)}$	$1.1\cdot10^{13}$ Nmm^2
Self-weight		**Self-weight**		**Self-weight**	
Q_P	2.65 kN/m^2	Q_P	1.41 kN/m^2	Q_P	0.56 kN/m^2
Design load carrying capacity		**Design load carrying capacity**		**Design load carrying capacity**	
$Q_R^{(k=0)}$	8 kN/m^2	$Q_R^{(k=0)}$	7.61 kN/m^2	$Q_R^{(k=0)}$	9.53 kN/m^2
$Q_R^{(k=\infty)}$	22.25 kN/m^2	$Q_R^{(k=\infty)}$	26.52 kN/m^2	$Q_R^{(k=\infty)}$	20.28 kN/m^2
load carrying capacity / self weight ratio		**load carrying capacity / self weight ratio**		**load carrying capacity / self weight ratio**	
$\eta_q^{(k=0)}$	3.02	$\eta_q^{(k=0)}$	5.4	$\eta_q^{(k=0)}$	17.02
$\eta_q^{(k=\infty)}$	8.4	$\eta_q^{(k=\infty)}$	18.8	$\eta_q^{(k=\infty)}$	36.21

3. Building Construction Method

The construction system is implemented using modular highly-prefabricated steel-timber components. With specific reference to Fig. 2, floor and bracing wall modular components are mounted by joining a CLT panel with a pair of steel beams, using *ad-hoc* mechanical devices. These elements are subsequently laid out and assembled to the main frames using particular mechanical connections arranged in timber-to-timber and steel-to-timber configurations. In addition, the beam ends are joined to the steel frames using common angle brackets with bolts.



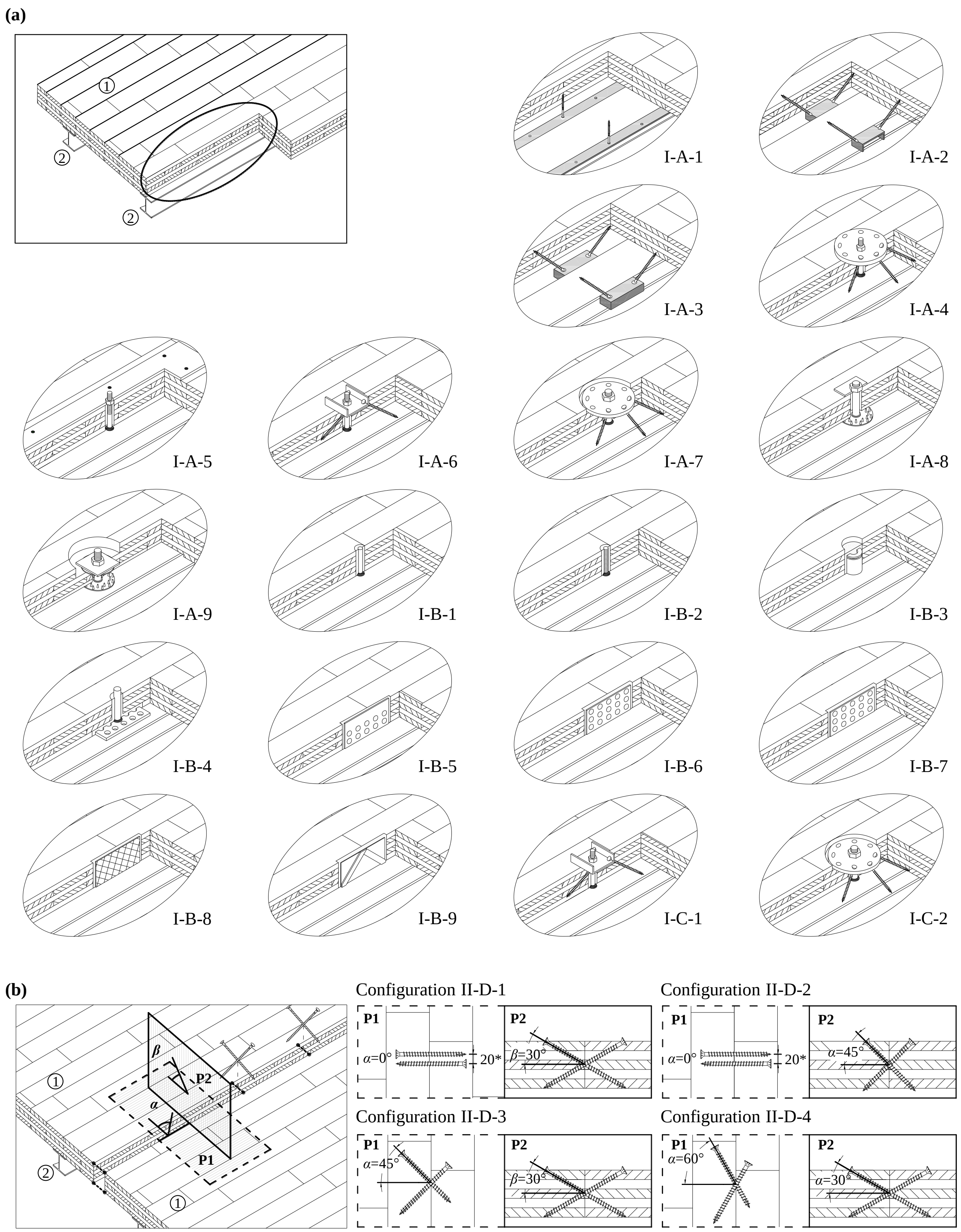
① Panel-to-panel connections ② Column-to-panel connections ③ Panel-to-main frame connections ④ Beam-to-panel connections

Fig. 2. Connections of the construction system. Different types of fasteners with steel-to-timber and timber-to-timber shear plane configurations.

Connectors for the floor components have to transfer the shear forces among the steel and timber elements for both their in- and out-of-plane behaviour. In particular, under bending deformation, the connectors have to avoid the activation of any possible detachment mechanisms of the elements. Connections for shear wall components must also provide the deformation and energy dissipation capacity required by Eurocode 8 [29] for earthquake-resistant structures. The connections introduced in this document have been engineered considering the current practice in the field of timber and steel construction ([30-32]). More specifically, the connections have been developed considering the installation tolerances and the practical aspects of the mounting sequence, as well as cost. These solutions can be considered at the cutting-edge of the building technologies not only for steel-timber hybrid systems, but also for other wood-based structures assembled using CLT panels or other engineered wood products.

3.1 Nonlinear behaviour of the connections

The load-slip curves were recorded via experimental tests carried out with the collaborating industrial partner. Test methods were described in detail in Part I [19]. In Fig. 3, Set I of steel-timber connections covers 20 different configurations, while Set II includes 4 distinctive arrangements of timber-timber connections. The whole experimental campaign consists of 24 monotonic displacement-controlled tests (MDS) in accordance with EN 26891 [33], 20 cyclic load-controlled tests (CLC) in accordance with Eurocode 4 [24], and 24 cyclic push (-) and pull (+) displacement-controlled tests (CDC) in accordance with EN 12512 [34]. A total of 52 specimens were built, 12 for timber-timber connections and the other 40 for the steel-timber connections. MDS tests were used for calibrating the loading procedures of the CDC tests. For the connections in Set II, the CDC test method was modified by applying a pre-loading history. Using the same specimens, CLC tests were carried out first and then CDC tests were performed. The effect of the pre-loading procedure is only in one direction (in compression (-)).



Note

* measures in mm ① CLT panel ② steel beam **P1** parallel plane **P2** perpendicular plane ***α***, ***β*** angles of insertion of screws

Fig. 3. Connections for hybrid steel-CLT floors and shear walls; (a) beam-to-panel connections (b) panel-to-panel edge connections.

Figures 4 and 5 show the non-linear behaviour of each connection measured by the experimental tests. In the charts, black thick lines are the load-slip monotonic curves while the brown thick lines represent the envelope curves.

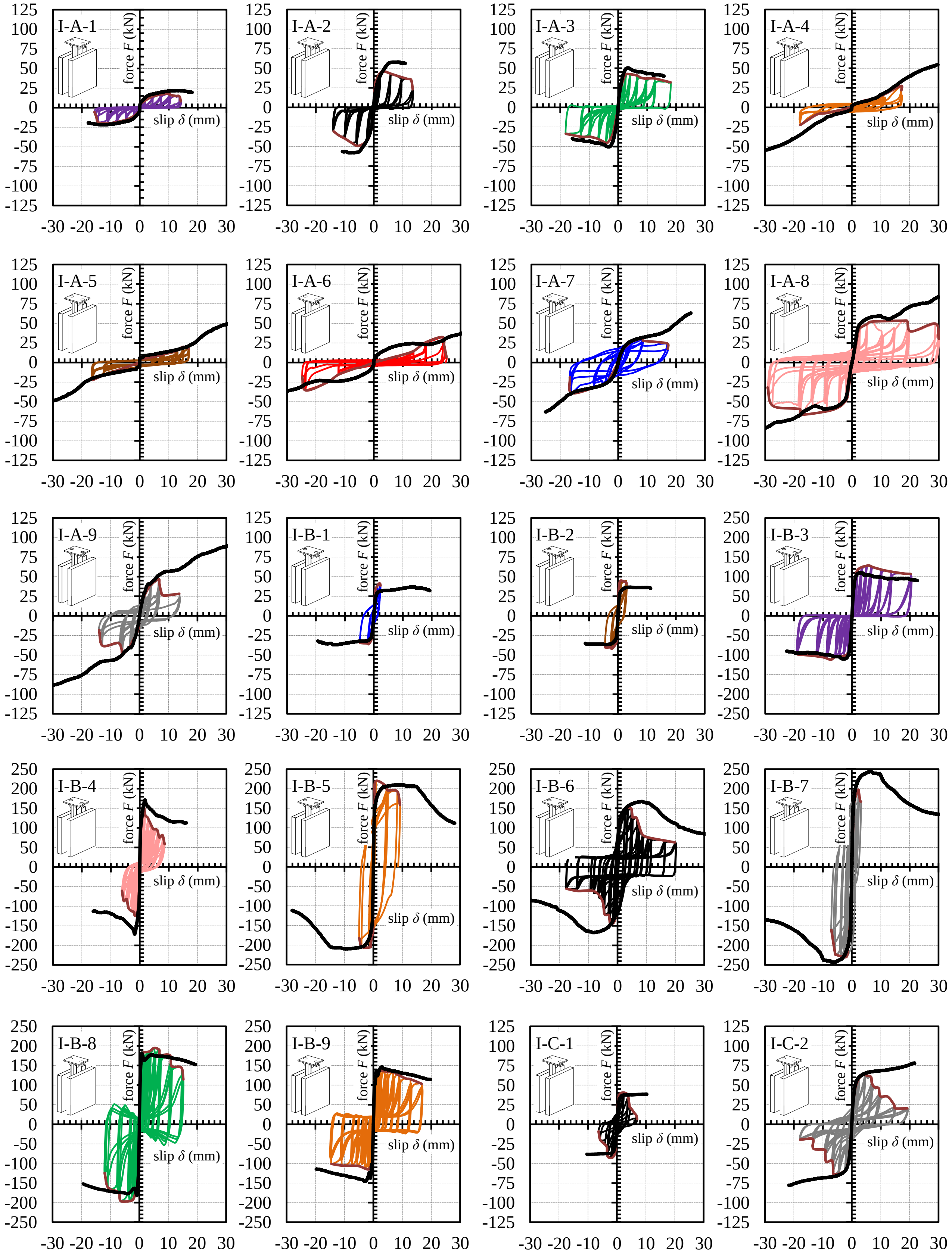


Fig. 4. Nonlinear load-slip (F-δ) response of the steel-timber connections measured by tests. In the charts: bold black lines are the monotonic curves while the bold brown lines are the envelope curves.

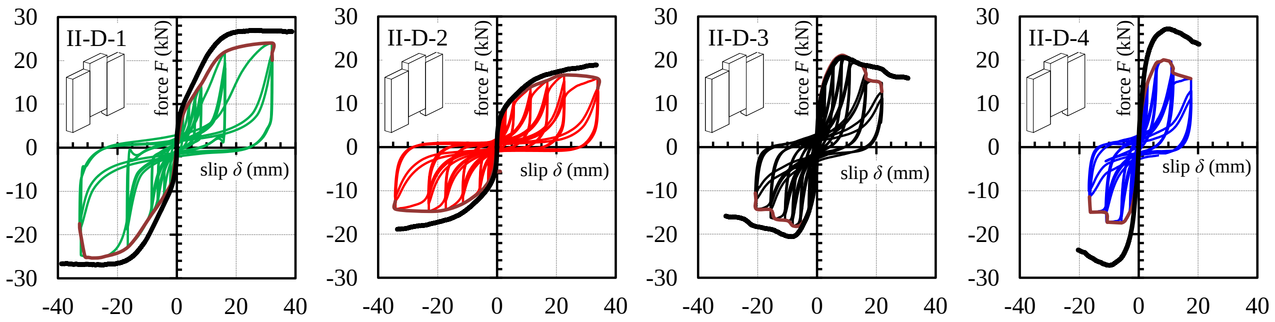


Fig. 5. Nonlinear load-slip (F-δ) response of the timber-timber connections recorded by tests. In the charts: bold black lines are the monotonic curves while the bold brown lines are the envelope curves.

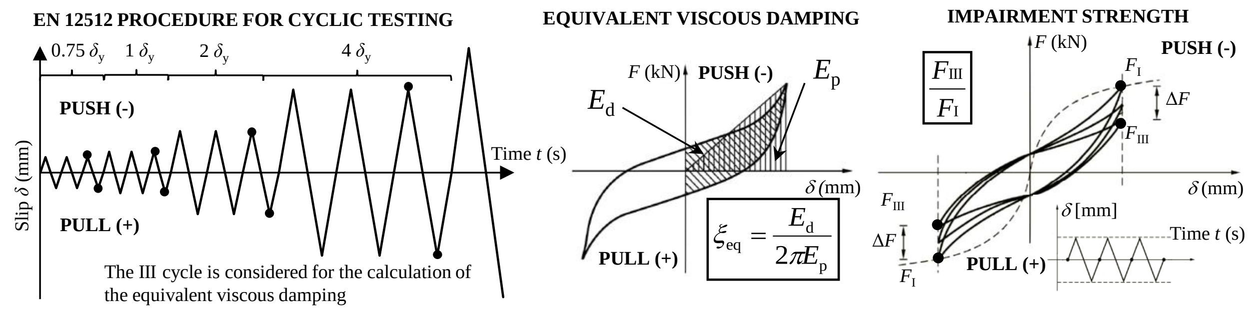
With specific attention to the steel-timber connections, the monotonic and envelope curves of Fig. 4 show that the behaviour is basically ductile, although the deformation and energy dissipation capacity varies considerably. In graphs of Figure 4, an unexpected result occurred in the configurations that use epoxy resin: I-B-1, I-B-2, I-B-3, I-B-8 and I-C-1. In fact, there is an abnormal deviation between the monotonic curves and the envelope of the cyclic curves. The trend is reversed compared to the expected situation and shows a bearing capacity higher in the cyclic case than in the monotonic one. This phenomenon is explained by the process of specimen production. The specimens were formed starting from different elements, tolerances in steel-timber holes and the process of pouring the epoxy resin. Therefore, even though the manufacture process has been subjected to quality control, the glued connections have not a uniform thickness of the resin between steel and timber parts.

With reference to Fig. 5, for timber-timber connections, the cyclic behaviour is characterized by a pinching effect which reduces their energy dissipation capacity. Connections II-D-1 and 2, when laterally loaded can accept large deformations without reduction of strength and stiffness, while connections II-D-3 and 4 when subjected to a combination of axial and lateral loads exhibit evident impairment of strength as the slip demand increases.

Being a comparative experimental campaign, only the connections with the preferable structural behaviour will be considered in the next Sections and the evaluated characteristics of these connections are provided in Table 2.

Table 2

Test results in accordance with EN 12512 [34]. List of the evaluated performance parameters.



Set I		Equivalent viscous damping ξ_{eq} (%)								Impairment strength F_{III}/F_I							
		$\mu = 1$	$\mu = 2$	$\mu = 4$	$\mu = 6$	$\mu = 8$	$\mu = 12$	$\mu = 16$	$\mu = 24$	$\mu = 1$	$\mu = 2$	$\mu = 4$	$\mu = 6$	$\mu = 8$	$\mu = 12$	$\mu = 16$	$\mu = 24$
I-A-1	Push	16.3	11.9	7.7	6.1	4.7				0.92	0.92	0.90	0.83	0.74			
	Pull	17.6	12.7	8.1	6.4	5.4				0.97	0.96	0.92	0.85	0.76			
I-A-3	Push	7.1	6.3	5.4	5.4					0.94	0.92	0.90	0.91				
	Pull	5.5	5.5	5.7	5.5					1.00	0.94	0.93	0.90				
I-A-8	Push	11.9	15.2	19.8	18.9	14.0				1.01	0.98	0.90	0.94	0.86			
	Pull	12.2	14.6	14.7	13.9	10.3				0.99	0.96	0.84	0.89	0.85			
I-B-2	Push	6.6	12.1	15.5	18.9					0.98	0.97	0.95	0.94				
	Pull	7.2	12.9	18.3	23.8					1.00	1.00	0.96	0.92				
I-B-5	Push	7.0	11.8	19.3	24.1					1.00	0.99	0.99	0.98				
	Pull	14.0	21.8	37.2	52.2					1.00	0.97	0.9	0.79				
I-B-6	Push	27.9	29.3	32.2						0.97	0.97	0.97					
	Pull	13.8	20.5	28.9						1.07	1.02	0.90					
I-B-7	Push	19.4	21.9	26.8	31.7					1.01	1.00	0.98	0.96				
	Pull	14.4	21.9	35.3	48.6					1.01	1.00	0.85	0.71				
I-B-8	Push	5.7	7.9	11.4	11.8	12.0	12.5	12.5	13.4	1.04	0.99	0.94	0.97	0.97	0.96	0.93	0.88
	Pull	6.8	10.8	13.5	15.1	15.6	16.1	16.6	17.5	0.98	0.98	0.98	0.96	0.95	0.91	0.86	0.76
I-B-9	Push	7.3	8.9	12.0	13.0	13.2	12.8	13.5	14.2	0.99	0.99	0.98	0.94	0.95	0.93	0.91	0.89
	Pull	10.1	12.4	15.2	15.3	15.3	15.6	15.6	15.5	0.96	0.97	0.93	0.88	0.88	0.89	0.87	0.86
I-C-2	Push	7.0	11.0	15.0	13.7					0.98	0.97	0.96	0.83				
	Pull	6.7	11.2	15.5	15.1					0.99	0.97	0.95	0.79				

Set II		Equivalent viscous damping ξ_{eq} (%)								Impairment strength F_{III}/F_I							
		$\mu = 1$	$\mu = 2$	$\mu = 4$	$\mu = 6$	$\mu = 8$	$\mu = 12$	$\mu = 16$	$\mu = 24$	$\mu = 1$	$\mu = 2$	$\mu = 4$	$\mu = 6$	$\mu = 8$	$\mu = 12$	$\mu = 16$	$\mu = 24$
II-D-1	Push	8.9	8.0	8.0	8.0					0.92	0.82	0.81	0.81				
	Pull	8.6	7.8	8.0	8.4					0.92	0.80	0.71	0.64				
II-D-2	Push	9.8	8.9	8.7	8.4	7.7	6.7			0.94	0.92	0.94	0.87	0.79	0.84		
	Pull	10.2	9.2	8.9	8.1	7.7	7.5			0.93	0.90	0.84	0.85	0.81	0.83		
II-D-3	Push	10.6	9.3	8.8	8.8	8.6	10.2			0.96	0.94	0.93	0.92	0.93	0.87		
	Pull	11.2	9.3	8.3	8.0	8.4	11.9			0.97	0.94	0.91	0.90	0.91	0.83		
II-D-4	Push	4.9	6.0	8.3	11.1					0.96	0.92	0.92	0.84				
	Pull	5.4	6.8	9.2	13.0					0.93	0.90	0.87	0.74				

For earthquake-resistant structures, connections have to provide sufficient ductility (at least 6) and energy dissipation capacity without an excessive loss of strength and stiffness. This means that the configurations to be considered are I-A-1, I-A-3, I-A-8, I-B-2, I-B-5, I-B-6, I-B-7, I-B-8, I-B-9 and I-C-2 for Set I, and all of Set II. From here on, only the aforementioned connections will be studied in the successive analyses. Figures 4 and 5, together with Table 2, collect all the data recorded and evaluated in accordance with EN 12512 [34] for the tested connections. For Set I, in all the charts of Figure 4 and in Table 2, two different behaviours are considered: in compression (-) and in tension (+). The behaviour in compression (-) takes into account the pre-loading phase simulated via CLC tests. This method of testing in compression (-) was defined to estimate the structural capacity of connections in composite floors, which are loaded by out-of-plane forces in elastic range (due to gravity loads) and in-plane forces that can take them into the inelastic field of deformation (due to strong seismic loads). The next Section will examine in detail the structural efficiency of the connections, comparing the behaviour of different construction components numerically simulated via FEM non-linear analyses.

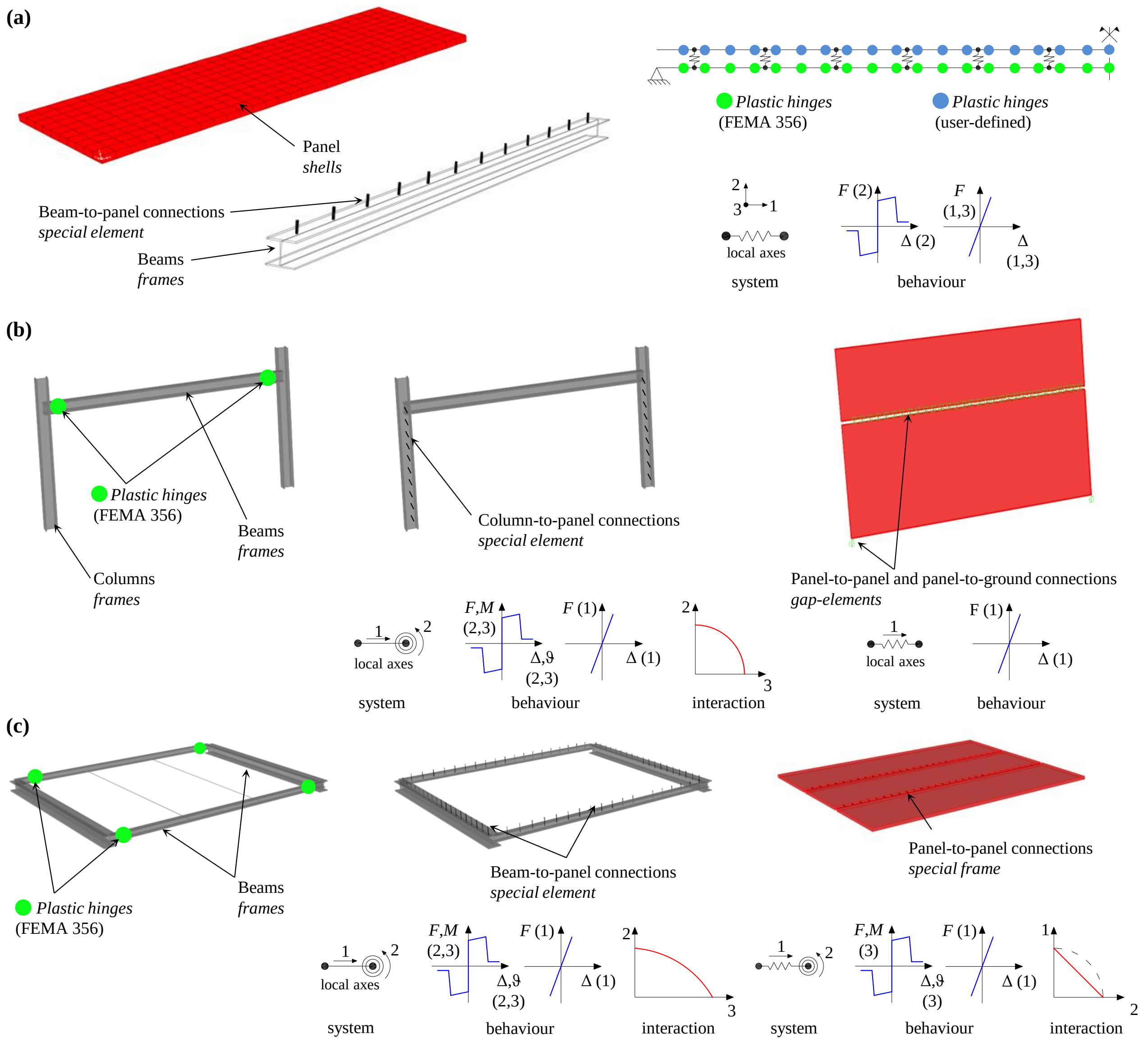
4. Innovative steel-timber hybrid-based floor and shear wall components

Two preliminary numerical studies were carried out in order to test the possibility of creating composite steel-timber collaborating floors that exhibit a rigid diaphragm behaviour. A complementary study was carried out to define ductile shear walls. With specific attention to the floors, the in-plane and out-of-plane behaviour were analysed separately. In each study, different combinations of Set I and Set II connections were considered, also varying their spacing and arrangement.

4.1 Nonlinear finite element analyses

Two refined FEM models were implemented in Sap 2000$^{(R)}$ [35]: FEM-I for the evaluation of the flexural behaviour of the composite steel-timber beams and FEM-II to study the in-plane nonlinear response of the floors and bracing walls. These models were developed taking into account the exact geometry of the structural members and connections, as well as considering each element with lumped plasticity. With reference to Figure 7, the bare steel frames and beams were modelled using one-dimensional elements while implementing two-dimensional shell elements for the CLT panels. The connections were reproduced with a set of special elements defined in order to represent their effective behaviour in all directions. The load-slip curves of the special elements were calibrated based on the experimental tests, assuming equivalent geometry and mechanical properties of materials. Both models were constructed considering plastic hinges located at the ends of the steel beams and columns. Plastic hinges in accordance with FEMA 356 [36] were considered for the steel elements, while user-defined hinges were considered for the timber elements. The inelastic response of the timber-to-timber and steel-to-timber connections was reproduced with plastic hinges placed at the top and bottom of the special elements. In order to replicate the shear mechanism of the connections in the stress direction, the load-slip response of the plastic hinges was defined by considering different surfaces of interaction (Figure 6). In the FEM models other special link elements were used to account for the interaction of the CLT panels at their edge surfaces or at the foundation level, as well as for friction.

In the models, CLT panels were made of 5 layers of C24 [28] timber boards (lamellar structure mm: 20/20/20/20/20), while the hot-rolled steel elements were made of steel S275 [26]. The stress-strain curve of the steel complies with Eurocode 3 [23], while the mechanical properties of the wood were modelled using an orthotropic elastic behaviour and an equivalent thickness that takes into consideration the effective elasticity and shear modulus ([37,38]).

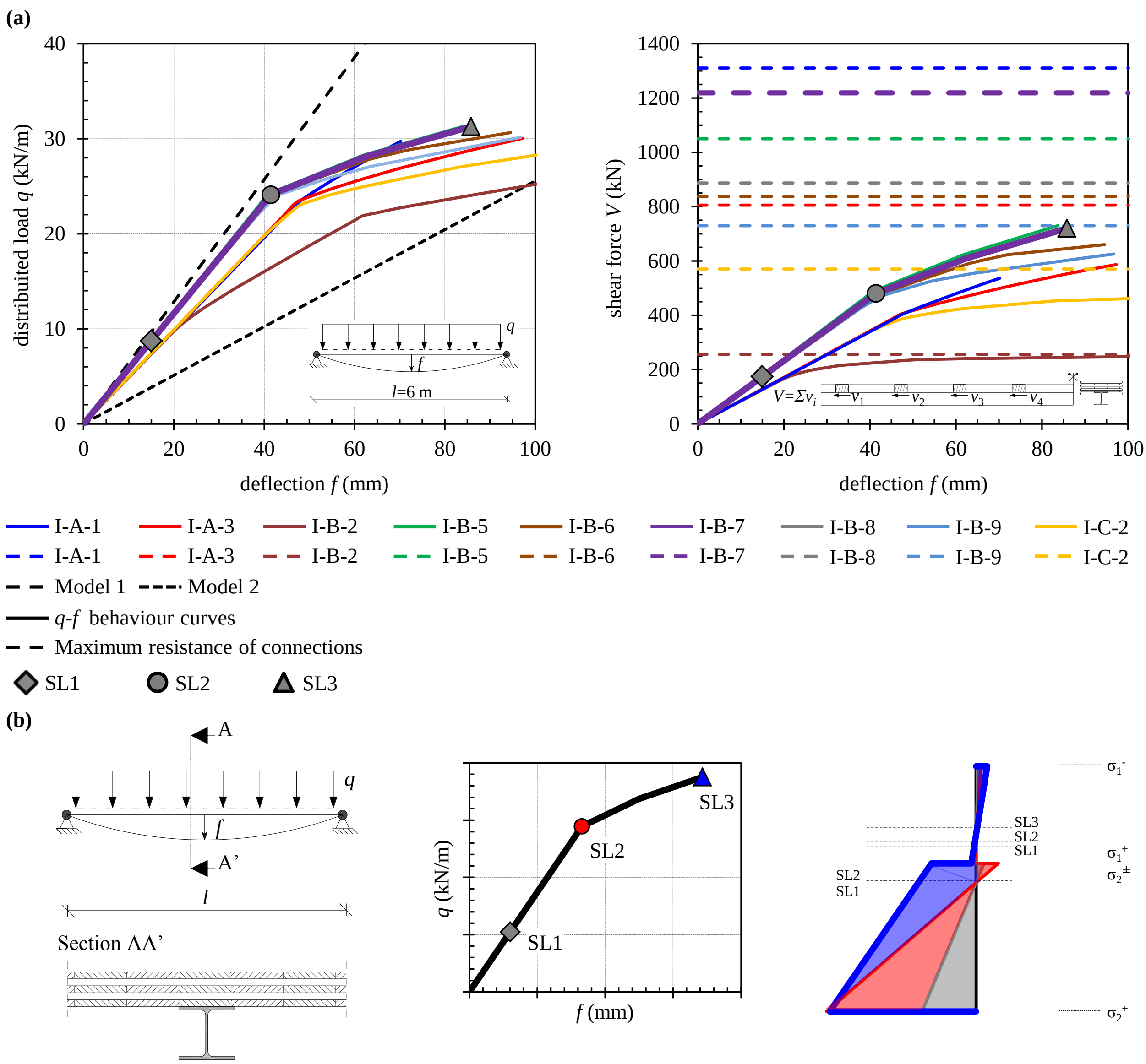


SAP2000®. FEM elements: *frame*, *shell*, *gap*, *special element* (user-defined)

Fig. 6. Finite Element Model (FEM) implementation of the proposed construction system.

4.2 Bending behaviour of the floors

The first study presented here concerns the evaluation of the bending response of composite steel-timber beams with CLT slabs. Figure 7a shows the load-deflection (q-f) curves obtained by the numerical FEM analyses for different composite systems, which differ in the number, arrangement and type of connectors used. In the charts, the dashed lines depict the behaviour of composite systems with fully composite action between the elements (Model 1), and no composite action (Model 2). The exchanged shear action between timber and steel at varying mid-span deflections is also illustrated in Figure 7b. The charts in Figure 7a highlight that all of the composite systems are quite ductile and the behaviour in the elastic range is close to that of the full composite system. The evaluated bending stiffness ranges from about 75% to 92% of that of the full rigid composite system. With the exception of cases with I-A-1 and I-B-6 connections, it is possible to build a composite section with good structural performance, particularly providing sufficient plasticity, overstrength and avoiding brittle failures. For all of the systems, the inelastic capacity is primarily activated in the steel beams and then in the connections. This mechanism of resistance reduces the instability of the beam-panel system, and avoids the brittle failure of the CLT panel, activating local plastic deformation in the cross-sections of the steel beam. Even though the behaviour of the systems is mainly influenced by the beam, connections play a fundamental role and must be carefully designed. From this preliminary comparison, the I-B-7 connections have been demonstrated to be more structurally efficient compared to the other connections. The load capacity of the I-B-7 composite beam is estimated at 188 kN, while flexural stiffness and displacement ductility are 3488 kN/m and 2.1, respectively. Figure 7b illustrates the stress state in the section for three different levels of deformation labelled SL1, SL2 and SL3. In particular, the distribution of stress in the composite beam complies with the material plastic models assumed in the FEM analysis.



Note

Model 1: model with fully composite action; Model 2: model without composite action; SL1: state corresponding to 15 mm of midspan deflection; SL2: state corresponding to yield strength of steel beam; SL3: state corresponding to the last condition.

Fig. 7. Behavioural curves of the steel-timber composite beams built utilizing different connections (a), shear-deflection trend of each composite system (b) and (c) stress-strain (σ-ε) state of the mid-span section for three different levels of deformation.

4.3 Diaphragm behaviour of the floors

The diaphragm plays a key role regarding the stability of the three-dimensional construction system, as well as in the distribution of the horizontal forces onto the individual vertical bracing elements. The second study presented here aims at finding practical effective solutions to connect the composite floor elements to their CLT slab sides. A parametric exclusive study was carried out in order to evaluate the diaphragm stiffness and bearing capacity of the floors loaded with a horizontal uniform force distribution. A total of 32 nonlinear static analyses were performed on floors that differ in the connectors used to fasten the CLT panels to the steel grid beams, panel-to-panel connections and their relative spacing. In this work, the contribution of the steel beams used to build the composite section and their connections were neglected. Figure 8 shows some of the findings obtained from the analyses, considering different matchings between beam-to-panel and panel-to-panel connections. The load-displacement curves in charts are derived only in weak directions.

The analyses demonstrate that the whole in-plane behaviour of the floors is very sensitive to the hierarchy between the beam-to-panel and panel-to-panel connections. In particular, the load-carrying capacity is mainly affected by the beam-to-panel connections, while the stiffness is considerably guided by the arrangement adopted for the panel-to-panel connections. As shown in Figure 8, the mean stiffness can range from about 22 kN/mm in the worst case scenario to 48 kN/mm in the best case scenario. Connections II-D-2, 3 and 4 demonstrate effective solutions when their pitch tends towards small. In general, for this type of construction system, the I-A-3 connections are recommended to join the CLT panels to the steel frame elements.

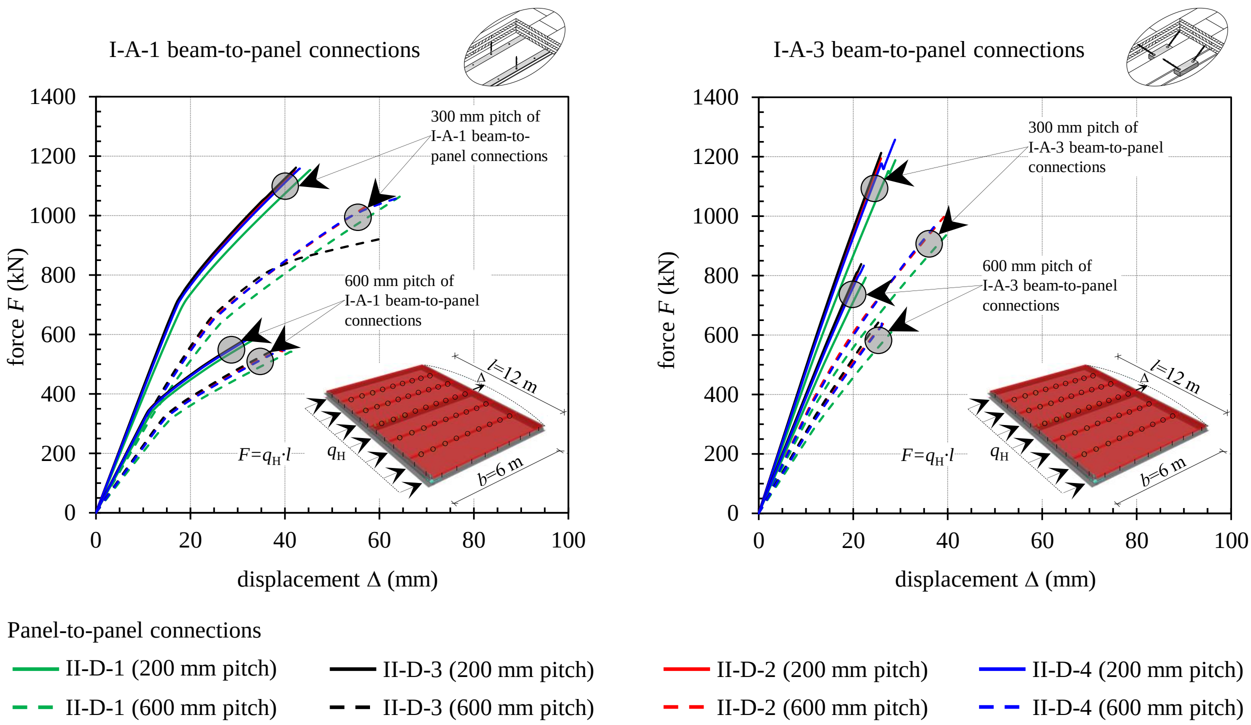


Fig. 8. Results from numerical study of different floors loaded in-plane.

4.4 Ductile behaviour of the shear walls

The construction system presented here is designed to achieve a dissipative seismic-resistant behaviour. The ductility and energy dissipation capacity are provided by the bracing walls, and involve the plastic deformation of the connectors.

Another parametric study was conducted on different bracing walls, which differs in the types of connections, both column-to-panel and panel-to-panel connections, their spacing, arrangement, as well as for the dimensions of CLT panels and number of the panel edges fastened to the steel frames. A total of 38 nonlinear static analyses were performed in order to evaluate the non-linear response of a single-storey wall. The charts in Figure 9 show the results of two case studies with I-A-1 and I-A-3 connections distributed along the column length and without fasteners between the beam and the panels. The results highlight that the response is mainly affected by the type and number of column-to-panel connections employed. In particular, wall configurations with I-A-3 connections are not recommended solutions, in view of their low plastic deformation and overstrength capacities. A more detailed analysis regarding the inelastic deformation mechanism highlights that sliding is concentrated mainly at the corners. The yield occurs in the connectors starting from those installed at the corners and moving towards the central part of the panel sides. Connections between CLT panels do not provide additional ductility to the system before most loaded column-to-panel connectors reach their failure. Therefore, solutions with I-A-1 connections prove to be a better choice in terms of seismic performance than I-A-3 solutions, also taking into consideration the remarkable reduction in the effective available bearing capacity.

The mean value of the ductility of walls with 100 pitch I-A-1 connections is about 3, while it is about 2.2 in the most favourable case of I-A-3 connections. In these cases, the initial stiffness is compatible, at about 19.42 kN/mm and 21.17 kN/mm in mean value for I-A-1@100 mm and I-A-3@300 mm, respectively. While all of the panel-to-panel connections can be used indifferently, solution II-D-4 appears to be more convenient for this case. We point out here that in this preliminary study the contribution of intermediate vertical steel supports has been neglected. Further detailed analyses will help to clarify the seismic behaviour of the shear walls within the construction system, considering in particular their energy dissipation capacity.

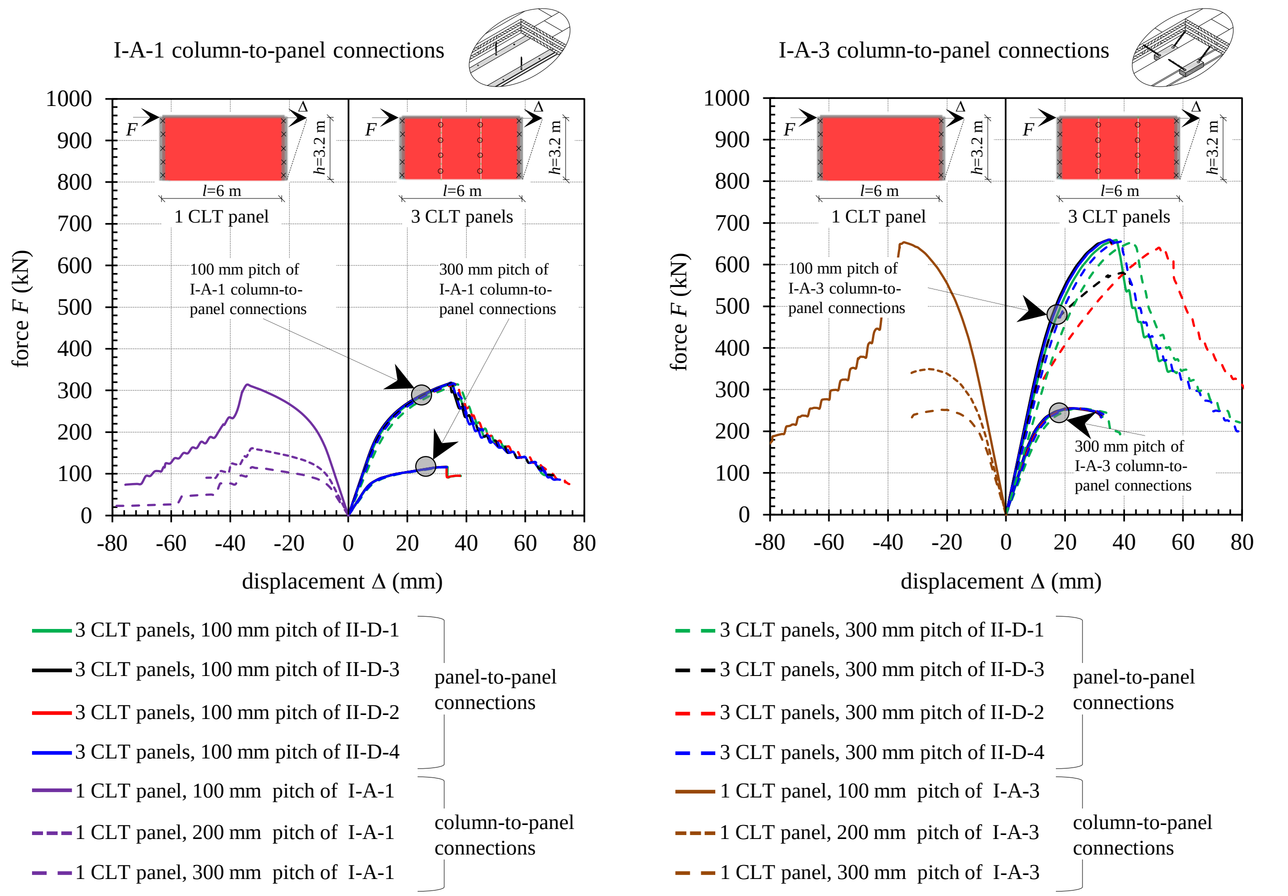


Fig. 9. Outcomes from numerical study of different single-storey bracing walls loaded at the top by a horizontal force.

5. Hybrid solutions and prototyping of new prefabricated modular components

This section concerns the final implementation of the construction system. Figure 10a depicts the standardized modular components assembled by combining cold-formed steel elements with CLT panels. The mounting method of the structure is then shown in Figure 10b. With specific reference to the floor elements, the thin steel beams have perforated parts at the upper side, while the CLT panel is equipped with pocket-holes. The composite system is made by the application of epoxy-resin in the cavities between the timber panels and steel beams. For the shear walls, modular elements are formed in dry conditions, using self-tapping screws in steel-to-timber shear configurations. The modular components are produced in the factory and then mounted on-site operating with screws and bolts inserted for both the floors and the bracing walls. Figure 10b also illustrates the selected construction connections based on this preliminary study, strongly supported by an exclusive experimental campaign and by numerical simulations that include nonlinear static analyses.

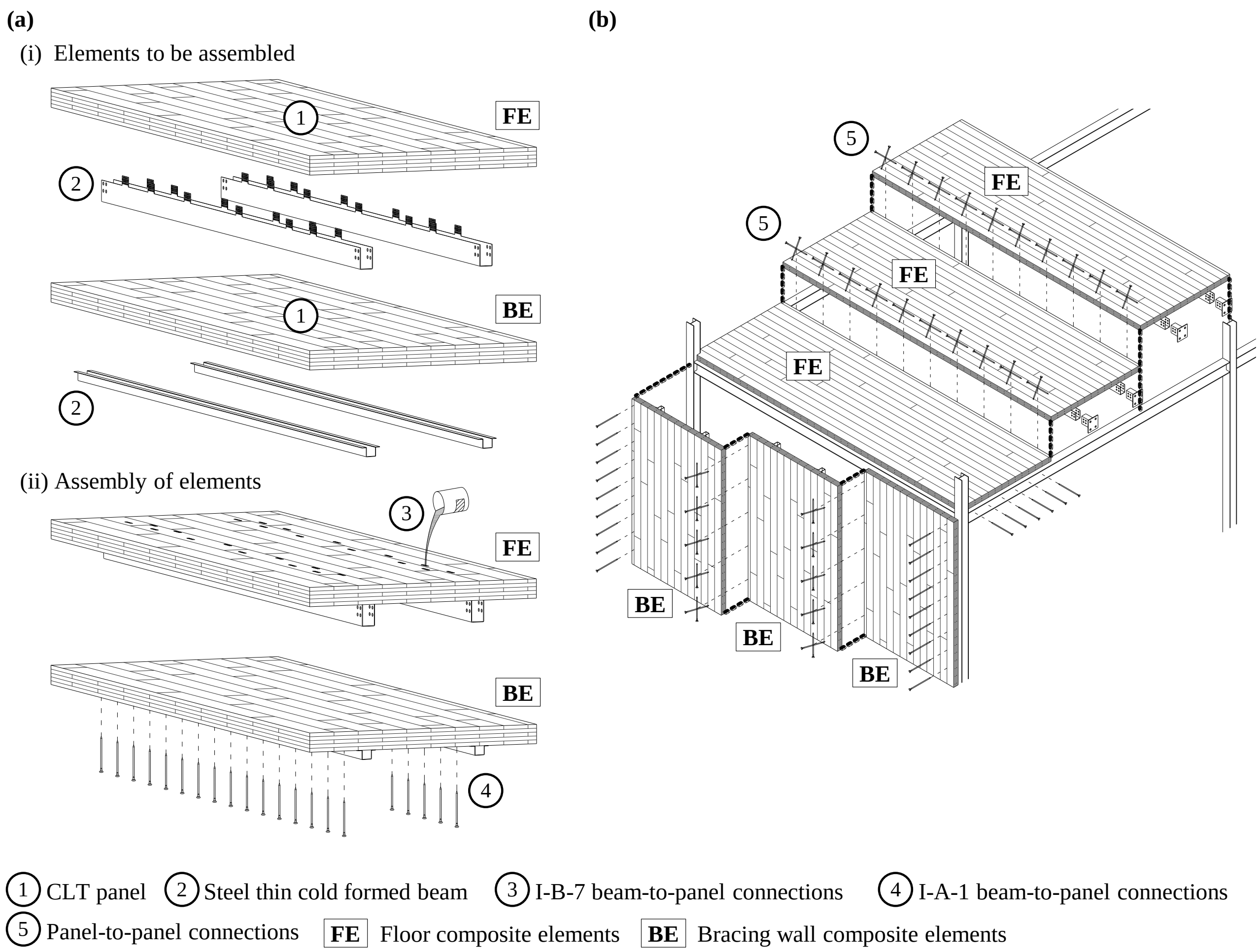


Fig. 10. Innovative hybrid steel-timber solution for high industrialized modular buildings. Building concept and structural components.

These hybrid solutions provide benefits in terms of lightness, repair, restoration and reuse of the structural components, as well as in reducing the time and costs of the construction work. The cold-formed steel elements were customized by incorporating parts of connections (I-B-7 and I-A-1) and engineered in order to reduce the self-weight of the construction components. Therefore, the amount of materials and other resources used are reduced, providing environmentally and economically sustainable construction solutions. As an example, the cross section of a 6 m span residential floor was designed in accordance with Eurocodes 2, 3, 4 and 5 ([22-25]). The floor is composed of a cold-formed steel element 180 mm in height and a 5-ply CLT panel 85 mm thick. The total weight of the composite floor is about 0.52 kN per square meter, while design loads are about 8.9 kN per square meters. Considering a unit area of floor, a total amount of 8.3×10^{-2} (97.4% of total), 2.0×10^{-3} (2.3% of total) and 2.3×10^{-4} (0.3% of total) cubic meters of timber, steel and epoxy-resin, respectively are required. In other words, the proposed composite floor system promotes the use of natural and renewable materials and reduces as much as possible the use of non-recyclable materials, such as epoxy-resin.

6. Conclusions

This paper has shown a new hybrid steel-timber construction system for modular residential buildings. The system developed has heavily industrialized modular prefabricated components made of timber and steel, and is equipped with smart connection solutions which reduce the on-site time and cost of erection of the building as much as possible. Furthermore, the unit construction modules for the floors and walls are manufactured starting from CLT panels with the same dimensions and utilizing standardized cold-formed steel elements with free-form shapes. The supply of raw materials is thus economical and can also cut the relative costs of production. In this research, the connections to be used to build composite elements and for the on-site construction of the building have been developed and extensively investigated. The proposed joining solutions allow the quick replacement of the bearing elements and components during the building lifetime. This preliminary study has focused on the connections, considering different fastener configurations, mechanical devices, connector materials, methods of assembly and other technical aspects which relate to the tolerances, mounting steps and costs. The work has demonstrated that it is simple to obtain composite floor elements which provide excellent flexural behaviour under vertical loads. In particular, the prototype of the modular floor component provided in this paper enhances the structural performance of the system, leading to a reduction of the self-weight and balancing the use of steel and timber. The intrinsic lightness of the floors also reduces the effects of earthquakes on the buildings. The lateral-load resisting system consists of a collection of steel frames and steel-timber hybrid components that transmit loads and forces from each storey to the foundation. The paper has illustrated how to create seismic-resistant buildings with a dissipative structural behaviour. In this context, the selection of the type and arrangement of connections needs to be carefully considered, at both the local and global level. Several connection solutions are provided for ductile shear walls and stiff diaphragms, while others are suited to building composite beams. The part of the research that focuses on the numerical analyses helps to understand the role of each connection in the structural behaviour of the building components. The connection solutions adopted in the definition of a shear wall prototype have been borrowed from the outcomes of a comparative study. New analyses are in progress at system level, in order to assess the effective flexural and diaphragm load-deformation response of the construction components, with experimental tests on full-scale campaigns. This paper demonstrates the potential of steel-timber hybrid construction systems for the easy construction of flexible sustainable buildings. The findings published within Part I and here in this document (Part II) are strongly supported by an exclusive laboratory experimental research programme in partnership with industry.

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