Connections for steel-timber hybrid prefabricated buildings.
Part I: Experimental tests

by Cristiano Loss, Maurizio Piazza, Riccardo Zandonini

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Highlights

- Steel-timber hybrid construction systems foster a sustainable built environment.
- The structure is developed in order to use standardized hybrid components.
- Several tests are presented on different steel-timber and timber-timber connections.
- Designed connections permit the prefabrication of floor and shear wall components.
- Friction must be considered in the design of CLT panel-to-panel connections.

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Keywords: Steel-timber connections; Shear connections; Hybrid structures; Composite structures; Wood-based structures; Cross laminated timber; CLT; Sustainability; Green design.

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1. Introduction

Hybrid constructions have proved to be effective structural solutions for the fabrication of modern buildings, especially when the constructions are built by joining modular prefabricated elements on site. Hybrid structures can cover a wide range of constructions [1, 2]. However, for contemporary multi-storey buildings, the most common ones are timber-concrete, steel-timber or timber-timber construction systems. Canada, New Zealand, the USA, Japan and Europe are areas with widespread use of buildings with a hybrid-based structure. In Europe, in particular, there are three important cases of hybrid tall buildings, built in England (Banyan Wharf Place), in Austria (Lifecycle Tower, [3]) and in Norway (Treet, under construction, [4]).

Hybrid structures are generally construction systems assembled through the connection and mutual interaction of structural elements made of different materials. The combination of materials can be defined depending on whether it refers to the construction elements and components or to the subsystems of the structures. This paper refers specifically to hybrid structures which combine steel and timber at the component level (hybrid slabs and shear walls) [5].

Several studies have been carried out regarding the hybrid construction systems in which steel and timber are the main structural materials or are joined in order to enhance the load-bearing capacity. Some recent literature can be cited: Asiz and Smith [6], Dickof et al. [7], Bhat et al. [8], He et al. [9] and Okutu et al. [10]. Such studies deal primarily with construction systems designed to build mid-rise buildings and with a view to the implementation of high-rise buildings. The strong interest in these innovative technologies has pushed the research into cost-effective, reliable and, above all, efficient solutions which allow hybrid buildings to compete against the most traditional and widely used construction methods [11,12,13,14,15,16]. The research presented hereafter deals with innovative hybrid construction systems that allow 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 allows the building processes to take place even in areas exposed to harsh weather conditions.

Generally, the design of new residential buildings needs to meet important requirements and standards, which can include the use of natural and sustainable materials or ensure sufficient architectural flexibility regarding possible future changes in the use of internal spaces. Hybrid steel-timber frame-based structures allow buildings to be designed with an open architectural plan and a sustainability perspective [17,18,19], since this technology can cover long spans and limit the use of non-recyclable materials as much as possible. This research work concerns the implementation of hybrid construction systems with steel frame bearing elements, floor slabs and bracing systems produced by fastening Cross-Laminated Timber panels (CLT) to the steel beams or columns. The research evaluates different joining solutions suitable to develop an effective interaction between steel and timber elements. We discuss several solutions that provide adequate structural stability of the system and allow the development of a composite action between materials. Some of these solutions allow a quick assembly of the elements on site through the use of mechanical fasteners (dry solutions), while others require the use of epoxy-based resin. In both cases, each connection has been designed for an easy assembly of structural components, in particular considering the installation clearances (tolerances) related to the construction system and materials. Moreover, some joining solutions for CLT panel-to-panel connections will be analysed. Connections using self-tapping screws effectively transfer both in-plane and out-of-plane shear actions. This work investigates some arrangements of screws in order to maximize the floor stiffness or the inelastic deformation capacity of the bracing system.

The strength and stiffness properties of the connections, assessed via experimental tests, are reported together with the recorded load-slip curves. The experimental campaign is included within an industrial research project financed by a public fund for the utilization of local resources and for sustainable development.

The paper is organized as follows. The construction system is briefly described in Section 2. Section 3 discusses the evaluation of the floor behaviour, considering in particular the structural hierarchy and the role of the different connections. Section 4 describes the fastening configuration designed to join the steel elements with the timber CLT panels. The same Section defines some mechanical solutions for assembling CLT panels. Section 5 reports and describes the experimental tests, while the recorded data are shown in Section 6. Section 7 discusses the experimental outcomes, particularly pointing out the feasibility of making steel-timber composite floors.

From the single connector to prototypes of a floor element and a shear wall component, the companion paper (Part II, [20]) will present two innovative prefabricated components. Moreover, it will describe some aspects concerning the behaviour of shear walls, and more generally the lateral-load resisting system. With specific reference to the floors, the discussion will demonstrate the competitiveness of hybrid steel-timber structural systems compared to those using timber-timber or concrete-timber solutions.

2. Construction System

The reference building is aimed at the residential housing market. In addition to the modularity and easy assembly on site of the structural components, the structure has to provide architectural flexibility in the distribution of the internal and external spaces, to allow not only for flexibility in current use but also for possible modifications throughout the building life. The architectural plan permits different distribution layouts of the single housing units and, if required, volumes to be added to the building facades (Fig. 1).

[Fig. 1]

The reference hybrid 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 replace the degraded elements. In particular, it is possible to merge the assembly technologies typical of steel structures with those characteristic of timber constructions. In the construction system, the CLT panels replace the traditional plate and shell elements such as concrete floor slabs, shear walls or cores, with the aim of lightening the structure and improving the industrialization processes involved. The construction site organization, the quality control of the materials and the assembling process are significantly improved, leading to a considerable reduction in the construction time.

The construction system is regular and repeatable in space, with linear steel elements and flat timber components. Referring to Fig. 1, the main structure is built by placing steel frames along the two main directions of the building. The floors are assembled with steel beams forming a grid structure, which supports the steel-timber horizontal components, properly connected to ensure both the out-of-plane and in-plane diaphragm behaviour to withstand the vertical loads and horizontal forces, respectively. The lateral stability of the structure is provided by some walls braced using steel-timber vertical components joined following a well-defined configuration. Fig. 2 illustrates the construction system of a generic intermediate storey for the reference building, with a detail of the structural arrangement of the bidimensional hybrid steel-timber components for the floors and shear walls.

[Fig. 2]

Hybrid steel-timber components can be made of steel elements with different cross-section shapes, assembled with CLT panels using different connections and connection arrangements. Any of the following technologies can be used, as preferred: hot-rolled steel beams, asymmetric elements with welded flanges, cold-formed steel elements or even hollowed cold or hot formed sections (Fig. 2). Part II [20] of this work will describe in details innovative prefabricated steel-timber components for highly industrialized buildings. Specifically, floor and shear wall components are assembled joining timber and steel elements with standardized dimensions in order to increase the manufacturing efficiency. Connections are selected and designed to provide adequate transmission of shear actions among the resistant elements, or to develop the energy dissipation and deformation capacity of the structural system.

The next Section discusses the structural behaviour of the floors, considering both their in-plane diaphragm behaviour and out-of-plane bending behaviour.

3. In- and out-of-plane behaviour of floors

The floors can be built using modular highly prefabricated steel-timber prototype elements as depicted in Fig. 2. Fig. 3 shows two three-dimensional views of a portion of the floor, pointing out the difference between the in- and out-of-plane behaviours. Connections can be divided into two main groups: those referring primarily to the steel elements (group I) or those regarding the CLT elements (group II). With specific reference to group I, the subgroup Ia includes steel-timber connections between beams and CLT panels, while subgroup Ib relates to the mechanical connections between the steel beams and the main frame of the building, commonly steel bolted brackets. With reference to group

II, subgroup IIa includes timber-timber connections at the edges of the panels, while subgroup IIb relates to the steel-timber connections used to attach the CLT panels to the steel frame elements around the edge.

[Fig. 3]

In Fig.3, the vertical loads are transferred directly by contact between the steel beams and the CLT panels. Indeed, the gravity loads mainly cause a flexural stress state on the floor elements. The connections of the first group allow the shear stress induced by the composite action between the steel beams and the timber panels to be transferred to the main structure. The connections of the second group have a secondary and more general role, providing the floor with the required transverse capacity in the distribution of the loads. For out-of-plane loads, the floor can be analysed as an assembly of composite steel-timber prefabricated components, mainly in bending in the primary direction and collaborating in the related transverse direction.

With loads acting in the mean plane of floors (Fig. 3b), all the steel-timber, steel-steel and timber-timber connections may be involved (groups I and II). The operating principle of the floor is similar to an in-plane timber diaphragm used in wood-frame construction systems [21,22,23,24,25,26]. The resistant mechanism is activated as a result of deformations in the connectors of group II, placed at the edges of the floor grid to join the CLT panels to the steel elements, as well as at the end- and side-connections among the panels. However, here the in-plane resistance and stiffness are increased by the contribution of prefabricated components joined using connections of group I. In more detail, the beams pinned to the main frames are braced by the CLT panels, and therefore perform a stabilizing function of the diaphragm.

The real innovation of the proposed hybrid system is the concept of prefabricated steel-timber components and the related connections between the steel elements and the CLT panels. This work presents some novel connection technologies developed for the implementation of steel-timber hybrid structures. The connections are derived from the current practice applied to timber constructions built with CLT panels [27,28], and by considering the technologies of composite steel- and timber-concrete structures [29,30,31,32,33,34,35,36]. The connections have been engineered considering the mechanical behaviour of the materials and the installation tolerances, as well as the feasibility and costs of assembly on site.

4. Connections for steel-timber composite floors and shear walls

This Section describes two distinct types of connections (Fig. 4). The first set of connections (I) includes mechanical devices designed to join steel elements to the CLT panels, in order to develop a composite action between the different materials and in any case to transfer the shear stresses between the elements. The second set (II) covers connections to hold together the CLT panels.

For the first typology (I), there are “all-dry” mechanical devices (A-type), connections that use an epoxy-based resin (B-type) and mixed mechanical devices, which combine the resistant properties of the resin and of the steel connectors (C-type). B-type connections are suitable to create an effective composite steel-timber section; A-type connections are more general and can also be used to join CLT panels to the steel beams in order to ensure an out-of-plane collaboration. Finally, C-type connections can be used indifferently to transfer shear stresses between elements due to in- and out-of-plane applied loads. With reference to Fig. 4a, A-type includes 9 different connection configurations:

-I-A-1, L-shaped thin profiles welded on both sides of the steel beam and connected to the CLT panel using self-tapping screws inserted perpendicularly to the grain;

-I-A-2/3, steel elements welded on both sides of the steel beam and connected to the CLT panel using full-threaded self-tapping inclined screws; I-A-2, cold-formed thin elements; I-A-3, steel thick elements;

-I-A-4/7, steel thick discs with flared holes, joined to the CLT panel using full-threaded self-tapping screws and tightened to a threaded bar welded to the steel beam; I-A-4, flexible bar; I-A-7, rigid bar;

-I-A-5, steel plate screwed to the CLT panel using straight full-threaded self-tapping screws, tightened to a threaded bar welded to the steel beam;

-I-A-6, steel plate screwed to the CLT panel using full-threaded inclined self-tapping screws, tightened to a threaded bar welded to the steel beam;

-I-A-8/9, shear plate type connector (Geka) fastened at the base using a threaded bar welded to the steel beam; I-A-8, single shear plane; I-A-9, double shear plane.

The B-type connection category includes 9 different solutions depending on the use of epoxy-based resin. The transmission of the stress between the steel beam and the CLT panel is ensured by the contact between the element surfaces. From Fig. 4a:

-I-B-1/2, bar welded to the steel beam and glued to the CLT panel using an epoxy-based resin; I-B-1, half-threaded bar; I-B-2, full-threaded bar;

-I-B-3, hollow pipe welded to the steel beam and glued to the CLT panel using an epoxy-based resin;

-I-B-4, bar welded to the steel beam and joined to the CLT panel using an epoxy-based resin; the connection is strengthened at the base with a steel perforated plate placed horizontally;

-I-B-5/6/7, steel drilled plate with smooth surface welded to the steel beam and connected to the CLT panel using epoxy-based resin; I-B-5, partially drilled plate; I-B-6, symmetrically drilled plate; I-B-7, partially drilled plate and epoxy-based resin with a different aggregate dosage;

-I-B-7/8, steel plate with rough surface welded to the steel beam and connected to the CLT panel using an epoxy-based resin; I-B-7, striated surface; I-B-8, plate with thin steel elements welded on it.

The C-type connection category encompasses mixed connection devices. In particular, the configurations I-C-1 and I-C-2 differ from the related I-A-6 and I-A-7 configurations, respectively, only in the application of epoxy-based resin in the space between the holes and the bars.

In the design of this first type of beam-to-panel connections, calculation models from the literature [28,37,38,39,40,41] were adopted considering every possible failure mechanism, involving steel elements and parts, fasteners or timber panels. The failure mode has been selected in order to avoid brittle failure of timber elements. In particular, breakages are expected to occur in the fasteners, and in other cases on the components welded to the steel beams.

The second set of connections (II) depicted in Fig. 4b concerns the CLT panel-to-panel edge connections. More specifically, they consist of crossed full-threaded screws, always installed in pairs, placed following different geometric configurations and arranged at a constant pitch. Each configuration is designed to provide bearing capacity for the in- and out-of plane loads of the floors. From Fig. 4b, all connections use a pair of screws and differ from each other in the angles of insertion of screws α and β, in relation to the panel plane (P1, in dashed line) and the perpendicular panel plane (P2, in thick line), respectively.

[**Fig. 4.]**

From a cost-effectiveness perspective, each solution requires a different installation procedure, varying not only in the equipment required but also in the processing time at the factory and at the construction site. Even though it is difficult to evaluate the most convenient solution, based only on economic factors, all the above mentioned connections have been designed taking into account the common working operations for timber and steel elements. Furthermore, the epoxy grout resin used to fill the volume in holes between timber and steel is a very common and cheap product. When required, other high-priced structural adhesives are available with superior mechanical properties.

5. Experimental tests

The nonlinear behaviour of connections was investigated via experimental tests. More specifically, push-out (push and pull) tests were performed, in displacement and load control mode, on specimens which differ in the number, types and geometry of the connections. Fig. 5 depicts the geometry of all the specimens used to test the first (Fig. 5a) and second (Fig. 5b) set of connections, using a double shear plane configuration. All specimens were assembled with CLT panels which have 5 layers of C24-strength class timber boards in accordance with EN 338 [42]. CLT panels have a nominal thickness of 100 mm with a lamellar structure of 20 *l* / 20 *t* / 20 *l* / 20 *t* / 20 *l*, three layers in the longitudinal direction (*l*) and the other two in the transverse direction (*t*). In the outer layers, the grain direction is oriented towards the beam main axis direction. Density and moisture content were measured in all the CLT specimens, giving a mean value of ρ_m=453 kg/m^3 and an average Moisture Content (MC) of 11.7 %.

[**Fig. 5.]**

Specimens for set of connections I were obtained by assembling steel elements (HE-shaped section) with CLT panels (Fig. 5a). The hot-rolled steel beams are made of steel S275 in accordance with EN 10025 [43]. The steel beams were strengthened on one end, by welding on them a perforated flange and other steel parts such as transversal stiffeners and ribs, cut from a plate of the same beam material. For set of connections II, specimens were built by joining the CLT panels at their edges (Fig. 5b). Set of connections II includes specimens with two different boundary configurations. In the configuration called T1 the panels have natural contact surfaces at edges, while for the configuration T2 the panels have smooth contact surfaces at edges. The latter configuration is obtained by inserting teflon strips between the edges of the timber elements.

With reference to Fig. 4, Table 1 shows the dimensions and mechanical characteristics of connectors for each connection configuration as used in the tests. 52 different specimens were tested, 40 specifically for the beam-to-panel connections (set I) and the other 12 for the CLT panel-to-panel connections (set II). The assembly method of the specimens was studied in detail in order to both respect the test principles and reproduce the real situation in the worksite. Then a work-bench was used in the assembling process to avoid offsets (Fig. 6a), misalignments and clearance between the elements. Particular care was taken in mixing and pouring the epoxy-based resin (Fig. 6b), as well as in applying a controlled tightening torque to the bolts (Fig. 6c) and in inserting screws in the CLT panels (Fig. 6d).

[**Table 1]**

[**Fig. 6.]**

5.1 Setup and specimen instrumentation

In the tests, two different setups (Fig. 7) were used in addition to a reaction frame tied at the base and a hydraulic electro-assisted actuator with push and pull features respectively equal to +1000 kN and -600 kN (Fig. 7a). Each setup was designed to carry out push and pull tests in a two-shear plane configuration with imposed monotonic and cyclic loading paths. With specific reference to the beam-to-panel connections (set I), the test configuration generates shear forces on the connectors mainly along the slip plane direction. The out-of-plane forces are self-equilibrated and in any case the specimens are restrained at the base by the setup. However, the lateral displacement among the elements was monitored, since it could occur due to non-expected transversal forces or to a possible separation of the components. The setup and the specimens were restrained using high-resistance bolts, pre-loaded to minimize the effects induced by the hole-bolt clearance. The instruments installed on the specimens for the tests were 9 LVDT-50 and -100 strain gauge linear displacement transducers, respectively with a 50 mm and 100 mm stroke (Fig. 7b).

[**Fig. 7.]**

The particular arrangement of the instruments allows both the total and the relative displacements between the elements to be measured. In particular, the relative slip between the different materials can be evaluated as an average of the different instruments. Similarly, the lateral displacement is determined as the average of the different instruments placed on the upper and lower side of the specimen. In addition, it is possible to evaluate every possible rotation between the CLT panels and the steel elements for both the main directions, as well as to define the instant of disjunction of the elements.

In the case of tests on the CLT panel-to-panel connections, similarly, the setup configuration was designed to impose shear stress only in the plane direction of the elements (Fig. 7a). Every timber element was restrained in order to prevent displacements in the out-of-plane direction. The displacements were measured by placing 11 transducers, as depicted in Fig. 7c. Two Quantum X -HBM®- power units, equipped with eight acquisition channels each, were used for the data acquisition. Two acquisition channels were connected to the jack to allow the recording of both the absolute displacement and the reached/imposed load level. A measurement device was assigned to each of the other channels. A sample rate of 5 Hz was used during the data acquisition.

5.2 Testing Method

Experimental tests with monotonic increasing loads were carried out in displacement control mode, imposing a deformation speed of 0.05 mm/s (Fig. 8a). The test procedure complied with the test principles of EN 26891 [44]. The tests were performed on 28 different specimens, 20 (set I) for the beam-to-panel connections and 8 (set II) for the panel-to-panel connections. A second test campaign, for only the beam-to-panel connections, was carried out in force control mode on 20 other specimens, applying incremental load cycles. The maximum load (F_M) was increased in the range from 5% to 40% of F_M, where F_M was determined in the previous displacement controlled tests. The loading history was constituted by five loading steps, each composed of five load cycles (Fig. 8b). This cyclic loading protocol was intended to simulate the elastic behaviour of a floor in operating state.

[Fig. 8.]

Twenty quasi-static cyclic tests were performed on these same steel-timber specimens in accordance with EN 12512 [45]. Similarly, four other quasi-static cyclic tests were carried out for the panel-to-panel connections on new 4 specimens in accordance with EN 12512 [45]. The measured cyclic force-slip curves and the data of these tests will be reported in detail in the companion paper [20].

6. Behaviour of the tested connections

The force-slip (F-δ) curves and the main recorded data from the tests are shown in Figs. 9, 10 and 11 and are reported in Tables 2 and 3 for sets I and II of the connections, respectively. In Tables 2 and 3, the yield points were derived according to EN 12512. Therefore, stiffness, yield force and yield slip, listed in Tables 2 and 3, comply with EN 12512 procedure. The other mechanical properties of the connections are derived in accordance with EN 26891 [44]. The trends in Figs. 9 and 10, together with the data of Tables 2 and 3, show the possibility of creating steel-timber collaborating systems for in- and out-of-plane composite actions.

[**Fig. 9.]**

[**Fig. 10.]**

[**Fig. 11.]**

[**Tab. 2.]**

[**Tab. 3.]**

6.1 Set of connections I

Considering connection set I, first of all, it is possible to observe that no connections were interested by a brittle failure before achieving a relative slip of about 10 mm. Within this level of deformation, the behaviour is basically ductile even though there is a high loss of load bearing capacity in connection I-B-4. The failures observed in the A- and C-type connections, as well as in cases I-B-1 and I-B-2, are mainly associated with the shear-tensile failure of screws or threaded bars. Specifically, the observed local ruptures of connectors are the tearing-off of the screw head or net shear-tension failure at the base of the bars. In the other B-type cases, the connections offer a consistent bearing capacity, about 50% of the maximum recorded load, also for large deformations, regardless of the failure detected in the contact surface between the resin and the steel element. Some representative failure modes observed during the tests are illustrated in Fig. 12.

[Fig. 12]

From the tested specimens, it is simple to show that the join configurations between I-B-5 and I-B-9 are stiffer than other cases. The related maximum recorded loads regarding the mentioned configurations vary in the range between about 170 and 250 kN. Such connections work with a shear mechanism which develops between the surfaces of steel and timber parts and the epoxy-base resin. In the other cases of B-type connections, instead, the resistant mechanism is provided by compression between the steel and timber parts and the epoxy-resin. These tests show better mechanical properties of B-type connections in shear deformation mode. With specific attention to the connections made with steel bars, in join configuration I-B-1, I-B-2 and in I-C-1, I-C-2, the load-bearing capacity is limited by the steel bar cross-section resistance, regardless of the resulting resisting mechanism. Furthermore, observing connections I-A-6 and 7, with their analogous solutions I-C-1 and 2, which respectively differ in the use of epoxy resin, there is a consistent increase of stiffness and the estimated yield load is about twice. Paying attention to the A-type mechanical solutions, the use of screws appears more attractive than the use of steel bars, especially if screws are inserted in the timber panels with an inclination angle between the screw axis and the grain direction of CLT panels. Indeed, I-A-3 exhibits the highest stiffness value of connections covered in the A-type subgroup. An interesting case is represented by join configurations I-A-2 and I-A-8, which have very similar mechanical properties. However, connections I-A-8 require an accurate installation process and the minimum pitch between these devices is higher than in the case with screws.

In the out-of-plane behaviour perspective, independently of the configuration type, B-type connections appear more attractive than other solutions. Considering Table 2 and the trends of Fig. 9, the best solution to build composite steel-timber floor elements is the I-B-7. Such connection ensures a higher bearing capacity compared to that of the other solutions, in addition to a high slip modulus (rise approximately up to 400 kN/mm) and a good ductility capacity. The latter provides the composite system with a load redistribution capacity within connections when yielding occurs. It is therefore possible to develop steel-timber composite systems that behave in a ductile fashion ([46,47]).

Table 2 shows the estimated main mechanical properties and the values of minimum connector spacing, considering the geometrical dimensions of the mechanical connectors and devices. The connector pitch complies with the requirements of Eurocode 5 [37] and DIN-1052 [48]. The mechanical properties are defined as recommended in EN 12512 [45], but assuming a slip limit of 15 mm instead of 30 mm. The composite efficiency defined in Eq. 1 was evaluated assuming a composite structure made of 100 mm thick CLT panels, steel beams with HE150-shaped section in accordance with the European wide flange sections, a distance of 1.2 m between the beams and connectors arranged using the minimum allowed spacing of Table 2. The materials have the mechanical characteristics as reported in Section 5.

$$\eta = \frac{EJ_{eff} - EJ_0}{EJ_\infty - EJ_0} \qquad \text{Eq. (1)}$$

In Eq. 1, η is the composite efficiency, EJ_{eff} is the effective stiffness of the composite system with semi-rigid connections, EJ_∞ is the stiffness in the case of a perfectly rigid composite system, while EJ_0 is the stiffness in the case of a system without connections between the elements. The effective stiffness was evaluated using the design formulas in Eurocode 5 [37].

In the case of "full-dry" A-type connections it is not generally advantageous to create a composite section, except in the particular cases of connection types I-A-1, I-A-2 and I-A-3. Indeed, structural efficiencies are less than 0.5 except for connections I-A-1, I-A-2 and I-A-3 in which the efficiency can rise up to 0.68. In the design phase, it is also possible to use an inclination of screws different from that used in tests, in order to maximise the mechanical performance of the connectors [41,49,50].

In the evaluation of the bending behaviour of the composite systems, it is also important to consider the residual slip (δ_r) due to the variation in the applied loads in service conditions. The cyclic tests performed by loading and unloading the specimens simulated such conditions (Fig. 10). Independently of the load level, the B- and C-type connections appear to be more suitable to develop composite sections, because they allow the total deformation to be recovered when the load is removed (δ_r<0.1 mm). The recorded residual slip of connection I-A-2 is also acceptable, while the residual slips of I-A-1 and I-A-3 connections significantly exceed the conventional limit value of 0.1 mm. Therefore, when designing the connections with screws, it is recommended to avoid as much as possible the embedment and tearing out of wood fibres of I-A-1 and I-A-3 connections, respectively, and rather force the breakage in the screws as happens for the I-A-2 configuration.

Generally, when the flexural collaboration between the elements is not the main issue, the mechanical devices I-A-1, I-A-2 and I-A-3 are a better choice to transfer shear forces between the elements. In particular, in connections made using screws, CLT panels are not arranged by creating holes or openings. The final assembled floor component is made by joining a rough CLT panel with a wrought steel beam. In all the other cases, the CLT panels were properly arranged, creating holes or openings following a specific pattern.

Part II [20], reported in the companion paper, will explain other aspects related to the cyclic and hysteretic behaviour of the connections. Particular attention will be paid to the possibility of development of a diaphragm behaviour of floors and of hybrid bracing walls with dissipative capacity.

6.1 Set of connections II

With reference to connection Set II, Fig. 11 shows eight different force-slip (F-δ) response curves, grouped on the basis of the main stress on the connectors. Table 3 summarises the main data recorded from the tests. The curves in Fig. 11a refer to laterally-loaded screws, and Fig. 11b to the case of screws subject to composite shear-tensile stresses. In the figures, the solid lines relate to the behaviour of the connections where the friction occurs at the edge surfaces of the CLT panels (T1), while the dotted lines are used for the cases without friction (T2). The latter situation was created by inserting a teflon layer between the timber surfaces at the edges of the panels. The tests mainly regard an in-plane load configuration, which is typical of slabs that offer bracing capacity.

The trends of load-slip curves highlight that the load-carrying capacity is affected by friction. The friction effect is a function of the force applied perpendicularly to the slip plane. This force is strictly related to the insertion angle of screws and can increase the load capacity by about 40% as per case II-D-1. Indeed, the non-balanced force acting between the panel edges and perpendicular to the slip plane increases as the axial stress component in screws rises. The stiffness is less sensitive to the friction effect and the maximum evaluated difference is approximately 25% (II-D-4).

In any case, the stiffness values for panel-to-panel connections are compatible with those of beam-to-panel connections with screws (I-A-1, I-A-2 and I-A-3), despite the different joint configuration. This is not true for the other B- and C-type connections. Based on this fact, the contribution of beam-to-panel and panel-to-panel connections together should be considered in the design of composite systems, such as diaphragms or shear walls. Considering the in-plane stiffness of floors as the main issue, the configuration II-D-4 appears to be the best solution. It is possible to define a floor configuration to maximise the external load distribution capacity. Otherwise, a suitable deformation capacity can be reached for the case of shear walls by placing screws according to the configuration II-D-1 and II-D-2 (Table 3). In Part II, reported in the companion paper [20], there will be a more detailed description of each aspect of the cyclic/oligo-cyclic response of the panel-to-panel connections with crossed screws.

7. Hybrid steel-timber prefabricated solution: a promising prospect

Hybrid structures made of steel elements and CLT wood-based panels allow the construction of lightweight and highly industrialized buildings. Such a construction technology permits the design of open space structures in which the construction elements are quickly and accurately joined using mechanical “full-dry” devices, therefore working effectively also in harsh climatic conditions. Furthermore, components or structural parts can be easily replaced, restored or recycled during the building lifetime.

With reference to the stability against lateral loads, hybrid systems with steel frames and CLT wood-based panels are typically as light as timber-timber mixed construction systems. However, they can take advantage both of the intrinsic deformation capacity of the steel elements and the use of mechanical fastening devices. Furthermore, since steel-timber hybrid structures are joined using special dry devices, they do not require either casting concrete on site or on site completion of precast concrete elements, as commonly required by heavy hybrid timber-concrete construction systems.

This work, in Parts I and II, reports on a study carried out on different joining solutions for the implementation of hybrid steel-timber prefabricated systems. Several tests on different connections were presented in this first Part, both for timber-to-timber and steel-to-timber configurations. Some connections are specific to composite sections, while others are used to fasten the plain CLT panels all together and to then connect these to the main steel frame. Based on these preliminary tests, connections I-A-3, I-B-7 and II-D-4 have been selected for the implementation and prototyping of the floors. A different structural role is considered for each connection type: I-A-3 connectors used to join CLT panels with the steel frames, I-B-7 connectors used to build steel-timber composite systems such as prefabricated floor elements, while II-D-4 connectors are used to join together the prefabricated elements at the panel edges. In order to make the dissipative bracing walls sufficiently stiff and strong in their main plane, the recommended connections are I-A-1, II-D-1, and II-D-2. Such connections allow a quick joining of components on site and provide a good inelastic deformation capacity. Part II will report in more depth on the response of the connections, with particular reference to the static and seismic behaviour of hybrid steel-timber structures.

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Figures

Fig. 1. (a) 3D axonometric section of the building; (b) Architectural (left) and structural (right) plan of the reference building.

Fig. 2. Construction system with mounting sequence of the hybrid steel-timber prefabricated components.

Fig. 3. Behaviour of hybrid steel-timber floors; (a) out-of-plane and (b) in-plane response.

Fig. 4. Connections for hybrid steel-CLT floors and shear walls; (a) beam-to-panel connections (b) panel-to-panel edge connections.

Fig. 5. Drawing of specimens for (a) beam-to-panel connections and (b) panel-to-panel connections.

Fig. 6. Assembling process of some specimens. (a) Specimens restrained to the work-bench, (b) pouring of epoxy-based resin into the hole, (c) application of a controlled tightening torque to a bolt, (d) self-tapping screws inserted transversally into CLT elements.

Fig. 7. Test setup and reaction frame (a); instrumentation layout for beam-to-panel connections (b) and panel-to-panel connections (c).

Fig. 8. Load protocols for (a) monotonic tests and (b) cyclic tests.

Fig. 9. Force-slip curves of the beam-to-panel connections.

Fig. 10. Force-slip cyclic curves of the beam-to-panel connections with estimated residual slip (δ_r).

Fig. 11. Force-slip monotonic curves of the panel-to-panel connections.

Fig. 12. Observed characteristic failure modes for the tested beam-to-panel connections.

Tables

Table 1

Dimensions of the connectors with related mechanical characteristics, as used in the tests.

Table 2

Main test results for the different tested beam-to-panel connections.

Table 3

Main results of the tested panel-to-panel connections.

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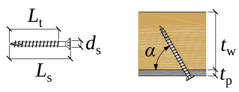
Dimensions of the connectors with related mechanical characteristics, as used in the tests.

I-A-1 | I-A-2 | I-A-3

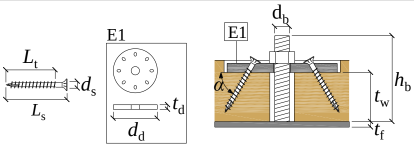


in all specimens

$f_{y,s}$=900 N/mm^2 , t_w=100 mm

	I-A-1	I-A-2	I-A-3
d_s (mm)	8	11	11
L_s (mm)	100	150	150
L_t (mm)	52	115	115
t_p (mm)	5	5	30
α (°)	90	45	45
n_s	2	4	4

I-A-4 | I-A-7 | I-C-2

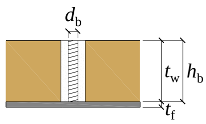


in all specimens

d_s=11 mm, $f_{y,s}$=900 N/mm^2, t_d=15 mm, t_f=9 mm, $f_{y,b}$=355 N/mm^2, d_d=180 mm, α=45°, h_b=140 mm

	I-A-4	I-A-7	I-C-2
L_s (mm)	150	100	100
L_t (mm)	115	65	65
t_w (mm)	100	80	80
d_b (mm)	16	24	24
			resin
n_s	8	8	8

I-B-1 | I-B-2

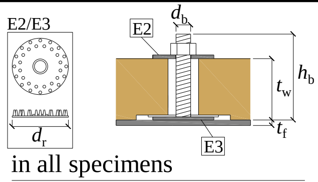


in all specimens

t_w=100 mm, d_b=16 mm, $f_{y,b}$=355 N/mm^2, t_f=9 mm, h_b=100 mm, epoxy-based resin

Note
I-B-1: half-threaded bar
I-B-2: full-threaded bar

I-A-8 | I-A-9



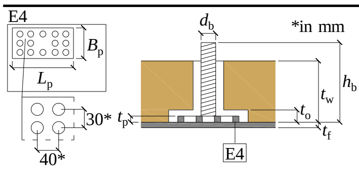
in all specimens

d_r=115 mm, d_b=24 mm, h_b=140 mm, $f_{y,b}$=355 N/mm^2, t_f=9 mm

	I-A-8	I-A-9
t_w (mm)	100	60

Note
I-A-8: single Geka (E2)
I-A-9: double Geka (E2-3)

I-B-4

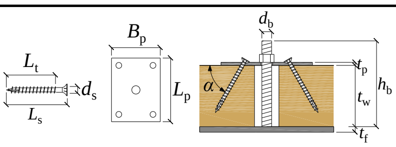


in all specimens

t_w=100 mm, d_b=24 mm, h_b=120 mm, L_p=200 mm, B_p=100 mm, t_p=5 mm, t_o=20 mm, $f_{y,b}$=355 N/mm^2, t_f=9 mm, epoxy-based resin

Note
E4 in horizontal disposition

I-A-5 | I-A-6 | I-C-1

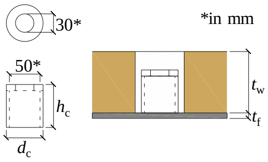


in all specimens

$f_{y,s}$=900 N/mm^2, t_w=100 mm, d_b=16 mm, h_b=140 mm, t_f=9 mm, $f_{y,b}$=355 N/mm^2

	I-A-5	I-A-6	I-C-1
d_s (mm)	8	11	11
L_s (mm)	100	150	150
L_t (mm)	52	115	115
t_p (mm)	10	5	5
α (°)	90	45	45
B_p (mm)	150	100	100
L_p (mm)	250	100	100
			resin
n_s	4	4	4

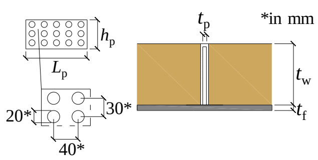
I-B-3



in all specimens

d_c=60 mm, h_c=70 mm, t_f=9 mm, t_w=100 mm, epoxy-based resin

I-B-5 | I-B-6 | I-B-7 | I-B-8 | I-B-9



in all specimens

L_p=200 mm, h_p =95 mm, t_w=100 mm, t_f=9 mm, epoxy-based resin

	I-B-5	I-B-6	I-B-7
t_p (mm)	6	6	6
holes	10	15	15

	I-B-8	I-B-9
t_p (mm)	6	12

Note
I-B-8: plate with rough surface
I-B-9: plate with welded elements

Note

$f_{y,s}$ screw yield strength, t_w wood thickness, t_f flange beam thickness, d_s screw diameter, L_s screw length, L_t threaded length of screws, t_p plate thickness, α angles of insertion of screws, d_d external diameter of disk, t_d disk thickness, d_b bar diameter, h_b bar length, $f_{y,b}$ bar yield strength, d_r Geka external diameter, L_p plate length, B_p plate width, h_p plate height, d_c external pipe diameter, h_c pipe length, t_o thickness of opening, n_s number of installed screws

Table 2
Main test results for the different tested beam-to-panel connections.

ID	K_i (kN/mm)	F (kN)			δ (mm)				μ	η	s (mm)
		F_y	F_M	F_u	δ_y	δ_M	δ_u	$\delta_{L,u}$			
I-A-1	8.2	16.8	21.84	21.45	2.1	12.47	15.00	0.24	7.3	0.53	40
I-A-2	28.1	47.4	57.97	54.86	1.7	8.44	11.33	0.23	6.7	0.66	80
I-A-3	31.0	43.7	50.32	41.06	1.4	3.24	15.00	0.28	10.6	0.68	80
I-A-4	1.8	**	27.39	27.39	**	15.00	15.00	0.14	**	0.04	275
I-A-5	4.8	7.6	18.53	18.53	1.6	15.00	15.00	0.00	9.4	0.16	140
I-A-6	6.0	17.9	24.26	23.91	3.0	13.56	15.00	0.03	5.1	0.11	265
I-A-7	10.0	26.0	37.42	37.42	2.6	15.00	15.00	0.34	5.8	0.19	240
I-A-8	24.6	49.7	59.47	59.13	2.0	9.42	15.00	0.19	7.4	0.28	345
I-A-9	18.0	43.5	62.33	62.33	2.4	15.00	15.00	0.01	6.2	0.23	345
I-B-1	43.4	30.3	36.84	35.30	0.7	13.62	15.00	0.26	21.4	0.75	80
I-B-2	64.4	32.5	36.55	35.87	0.5	4.36	10.93	0.16	21.9	0.82	80
I-B-3	126.6	93.6	109.92	94.70	0.7	2.60	15.00	1.81	20.3	0.70	300
I-B-4	515.3	108.4	171.36	137.09	0.2	1.80	5.16	1.21	24.6	0.83	600
I-B-5	480.7	190.1	209.92	204.26	0.4	9.29	15.00	0.26	37.5	0.92	230
I-B-6	309.5	139.3	167.36	136.06	0.5	8.11	15.00	0.52	33.3	0.88	230
I-B-7	372.4	207.2	243.76	195.01	0.6	6.43	14.61	0.51	17.7	0.90	230
I-B-8	402.2	165.3	177.38	163.62	0.4	4.38	15.00	1.18	36.6	0.91	230
I-B-9	308.7	117.3	145.88	122.78	0.4	2.98	15.00	1.14	39.5	0.87	260
I-C-1	52.4	36.1	38.50	38.50	0.7	10.24	10.24	0.03	14.8	0.52	265
I-C-2	60.7	60.7	71.26	71.26	1.0	15.00	15.00	0.35	15.0	0.59	240

ID, connector identifier
K_i, initial slip modulus according to EN 12512
F_y, yield load according to EN 12512
F_M, maximum recorded load within a slip of 15 mm
F_u, ultimate load, the load at the ultimate slip
δ_y, yield slip according to EN 12512
δ_M, slip at maximum load
**Case with no yielding point within 15 mm

δ_u, ultimate slip defined as the slip matching the minimum between the failure load and the 80% of maximum load, anyway not greater than 15 mm
$\delta_{L,u}$, average lateral opening between the steel and timber elements
μ, slip ductility of the connection evaluated as the ratio between the slip at the ultimate and yield load
η, structural efficiency of the connection assuming the minimum allowed spacing
s, minimum spacing for connections according to Eurocode 5 and DIN 1052

Table 3
Main results of the tested panel-to-panel connections.

ID	K_i (kN/mm)	*F* (kN)			δ (mm)			μ	*s* (mm)	*k* (kN/mm)/m
		F_y	F_M	F_u	δ_y	δ_M	δ_u			
II-D-1-T1	3.0	26.2	26.94	26.77	7.9	24.75	30.00	3.8	65.00	45.4
II-D-1-T2	2.6	12.3	19.22	19.22	4.2	30.00	30.00	7.2	65.00	39.7
II-D-2-T1	4.0	12.7	18.58	18.58	2.8	30.00	30.00	10.7	65.00	59.6
II-D-2-T2	3.9	11.8	16.52	16.52	2.8	30.00	30.00	10.9	65.00	58.5
II-D-3-T1	8.9	14.3	20.59	19.05	1.5	8.61	15.00	10.3	205.00	35.7
II-D-3-T2	7.9	13.4	19.28	18.66	1.7	10.41	15.00	8.7	205.00	31.6
II-D-4-T1	6.1	23.3	27.16	26.04	3.5	10.18	15.00	4.2	260.00	18.4
II-D-4-T2	8.3	17.8	22.60	22.39	2.0	13.00	15.00	7.4	260.00	25.0

ID, connector identifier
K_i, initial slip modulus according to EN 12512
F_y, yield load according to EN 12512
F_M, maximum load within 30 mm for configurations II-D-1 and 2, within 15 mm for configurations II-D-3 and 4
F_u, ultimate load within 30 mm for configurations II-D-1 and 2, within 15 mm for configurations II-D-3 and 4
δ_y, yield slip according to EN 12512
δ_M, slip at maximum load
δ_u, slip at ultimate load
μ, slip ductility of the connection evaluated as the ratio between the slip at the ultimate and yield load
s, minimum spacing for screws according to Eurocode 5
k, joint stiffness per meter placing screws with the minimum allowed spacing

Connections for steel-timber hybrid prefabricated buildings.
Part I: Experimental tests

by Cristiano Loss, Maurizio Piazza, Riccardo Zandonini

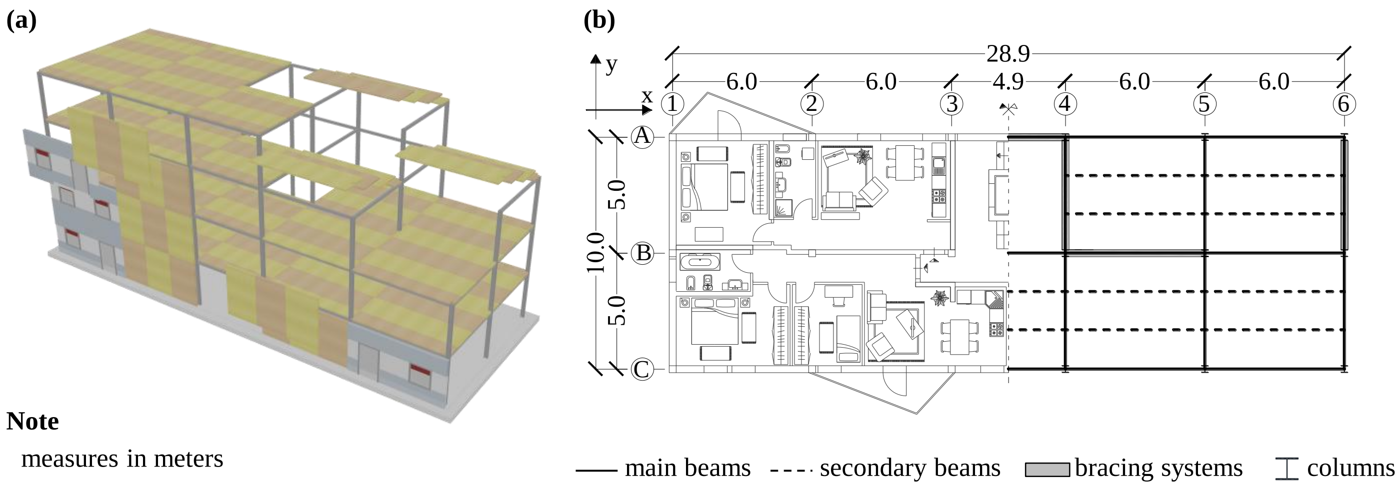


Fig. 1. (a) 3D axonometric section of the building; (b) Architectural (left) and structural (right) plan of the reference building.

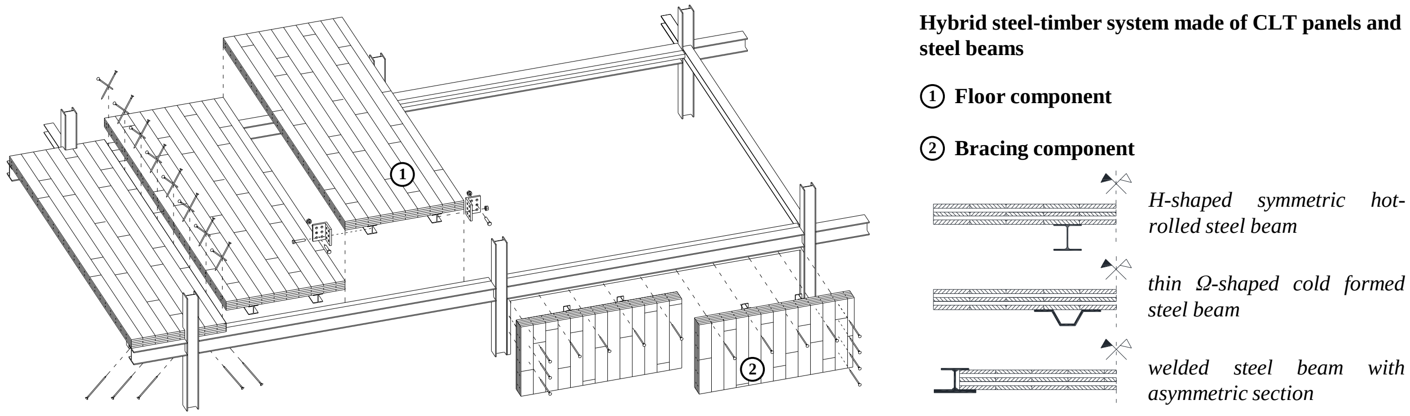


Fig. 2. Construction system with mounting sequence of the hybrid steel-timber prefabricated components.

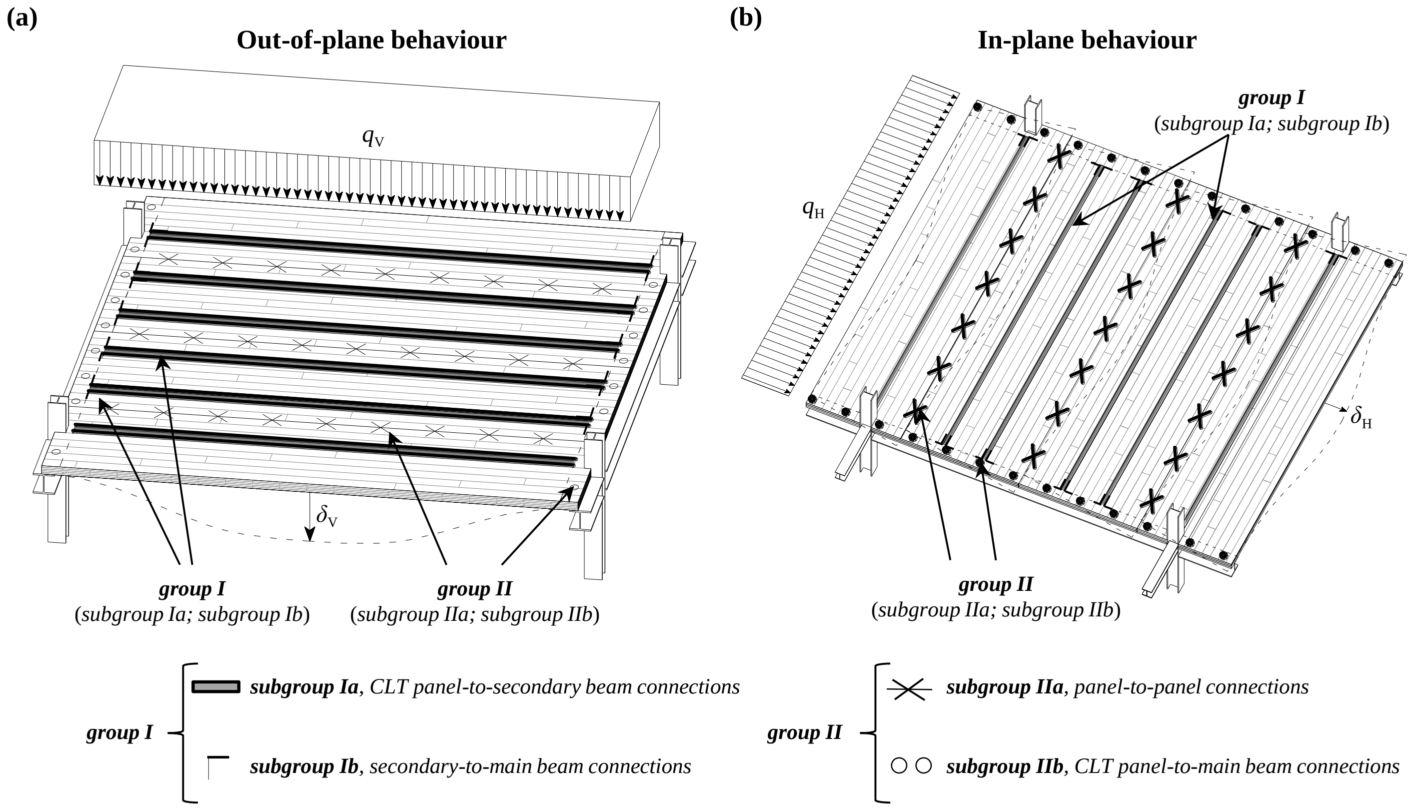
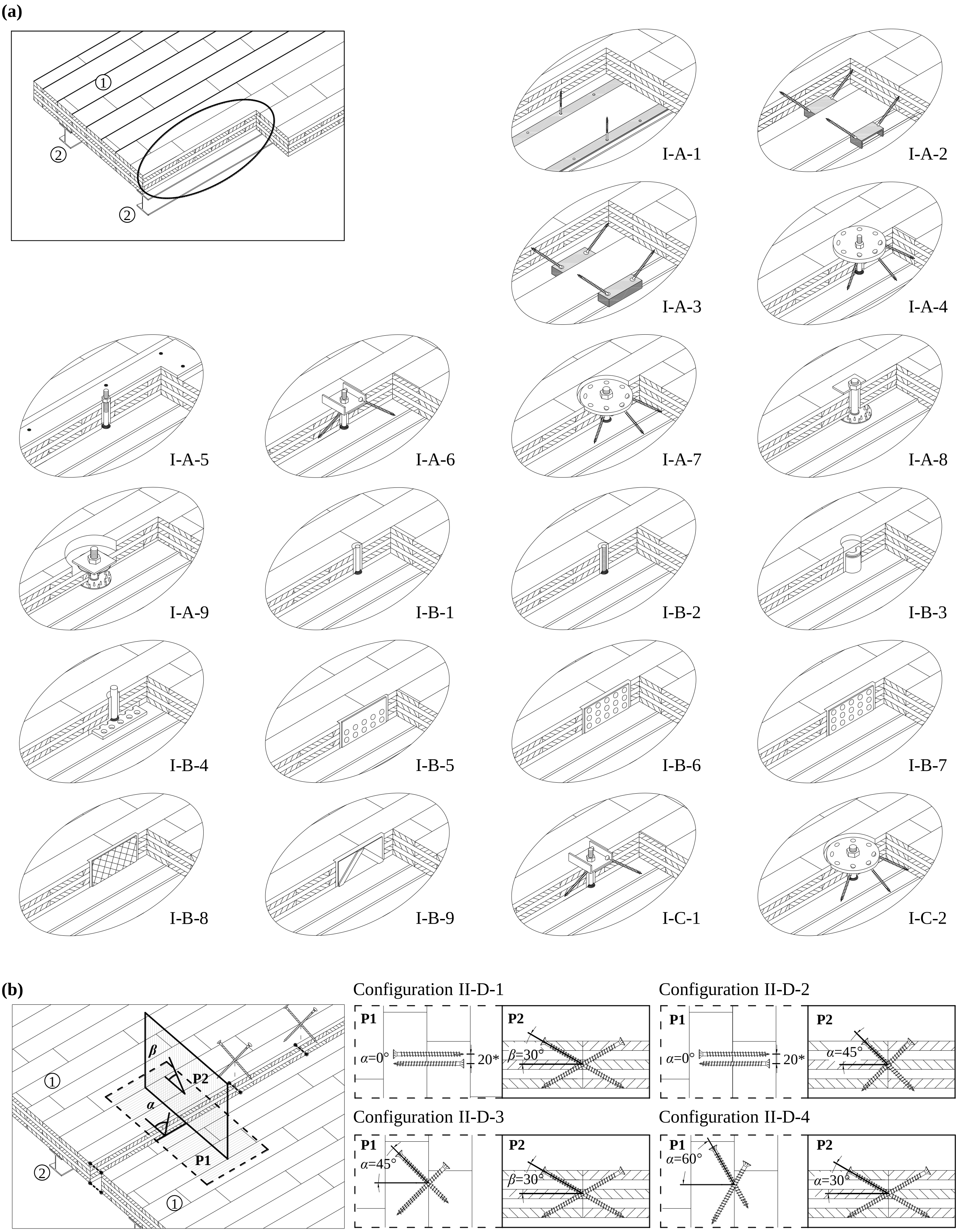


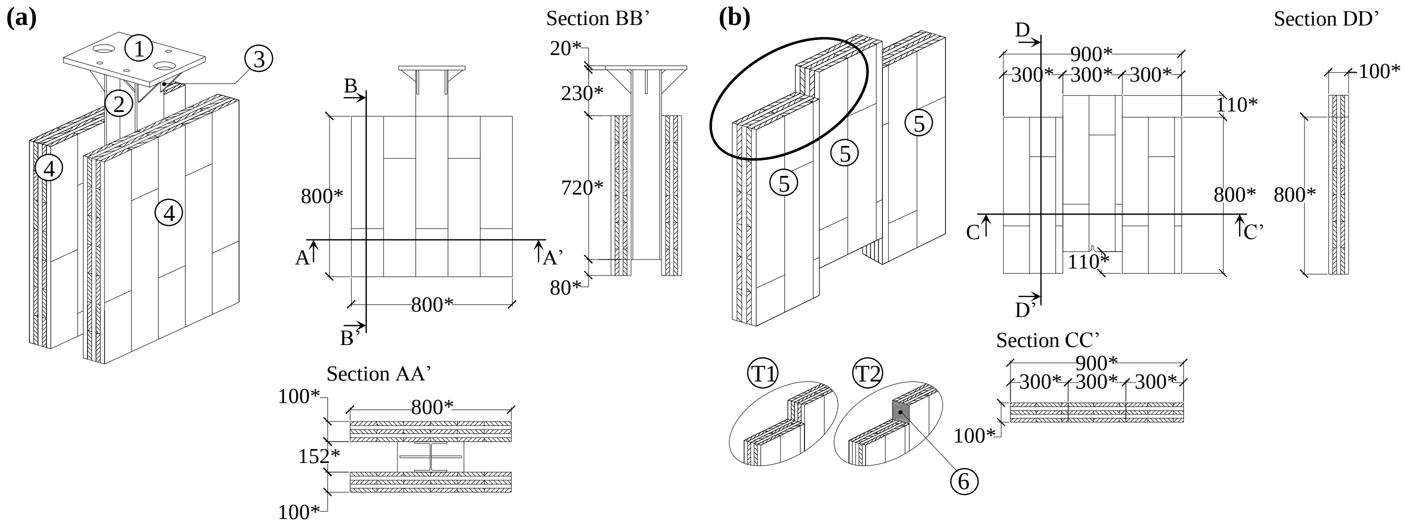
Fig. 3. Behaviour of hybrid steel-timber floors; (a) out-of-plane and (b) in-plane response.



Note

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

Fig. 4. Connections for hybrid steel-CLT floors and shear walls; (a) beam-to-panel connections (b) panel-to-panel edge connections.



Note

* measures in mm
① welded top plate
② steel beam
③ steel transversal ribs
④ CLT panels 800x800x100
⑤ CLT panels 300x800x100
⑥ teflon layer
T1 configuration without any layer between panel surfaces
T2 configuration with a teflon layer between panel surfaces

Fig. 5. Drawing of specimens for (a) beam-to-panel connections and (b) panel-to-panel connections.

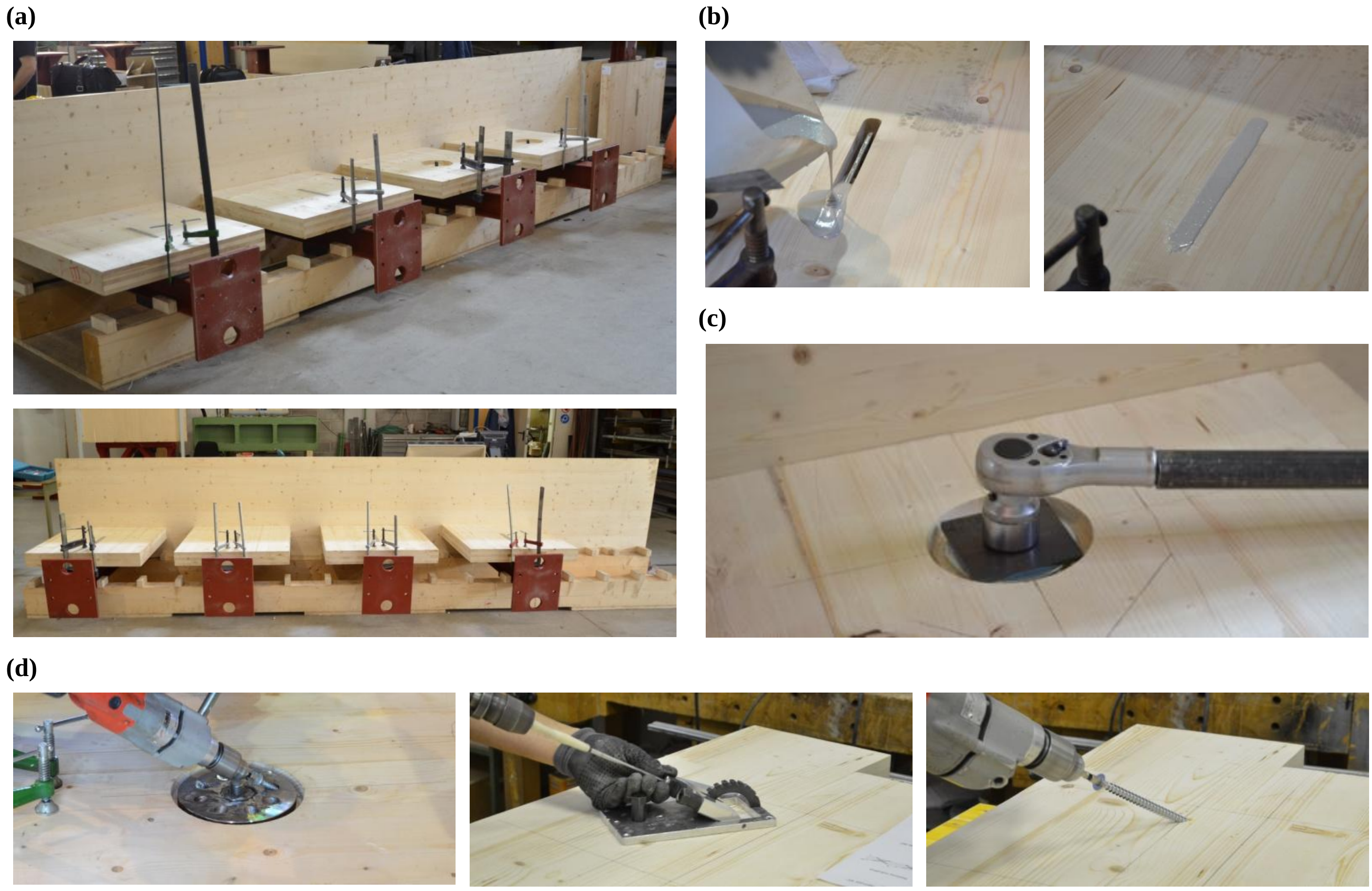
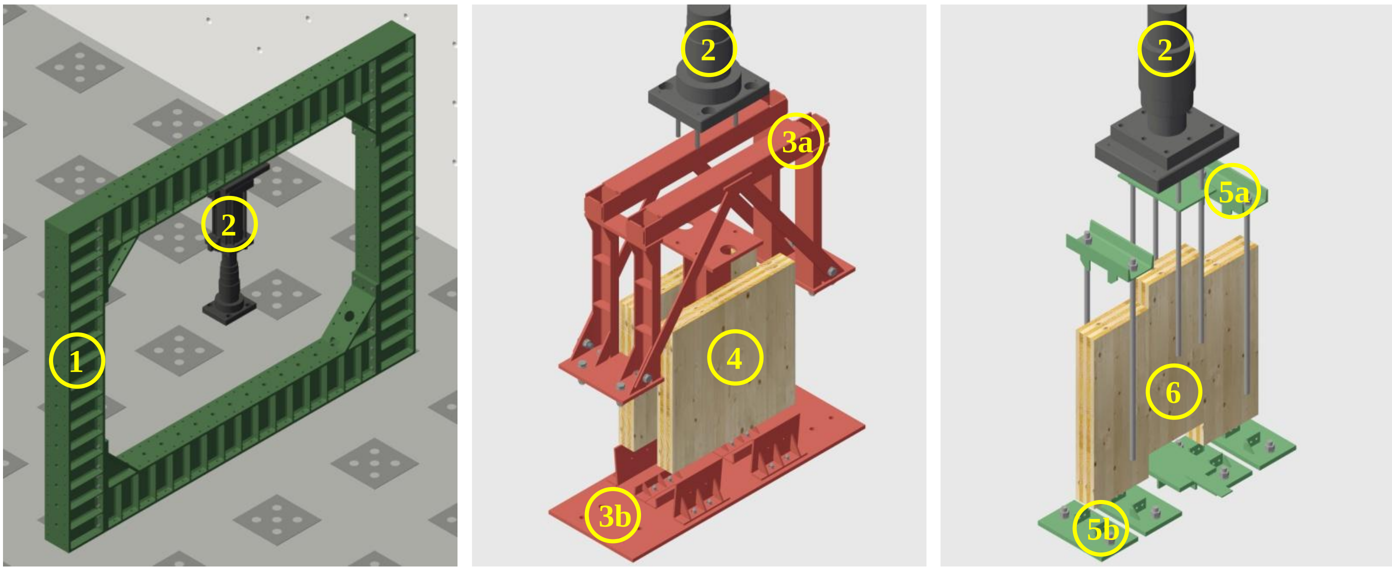


Fig. 6. Assembling process of some specimens. (a) Specimens restrained to the work-bench, (b) pouring of epoxy-based resin into the hole, (c) application of a controlled tightening torque to a bolt, (d) self-tapping screws inserted transversally into CLT elements.

(a)



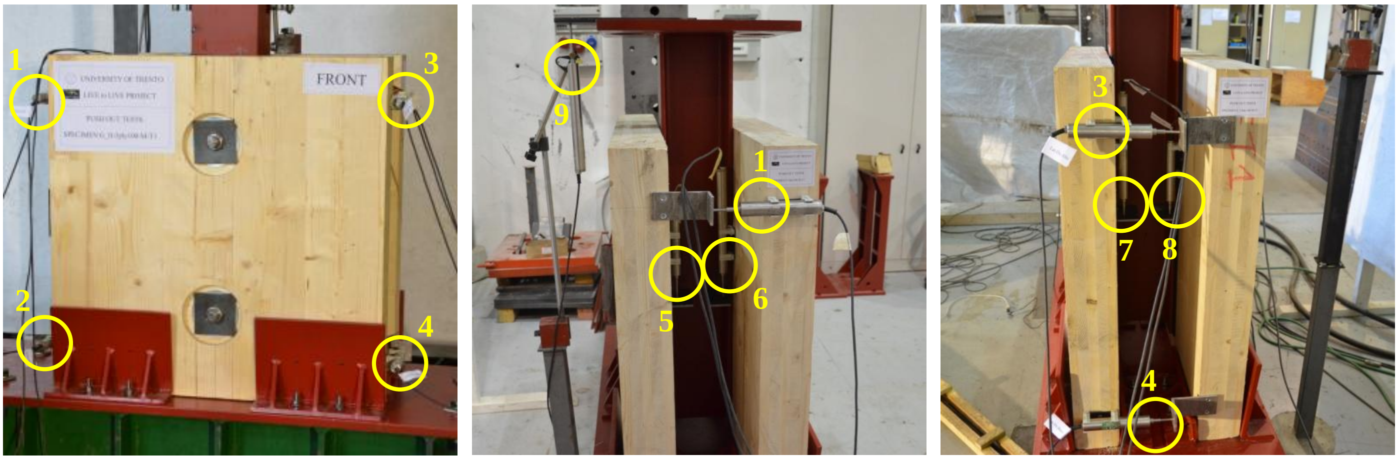
(1) frame reaction system (2) hydraulic jack (3a) upper part of test setup for beam-to-panel connections

(3b) lower part of test setup for beam-to-panel connections (4) steel-timber specimens

(5a) upper part of test setup for panel-to-panel connections (5b) lower part of test setup for panel-to-panel connections

(6) panel-to-panel specimens

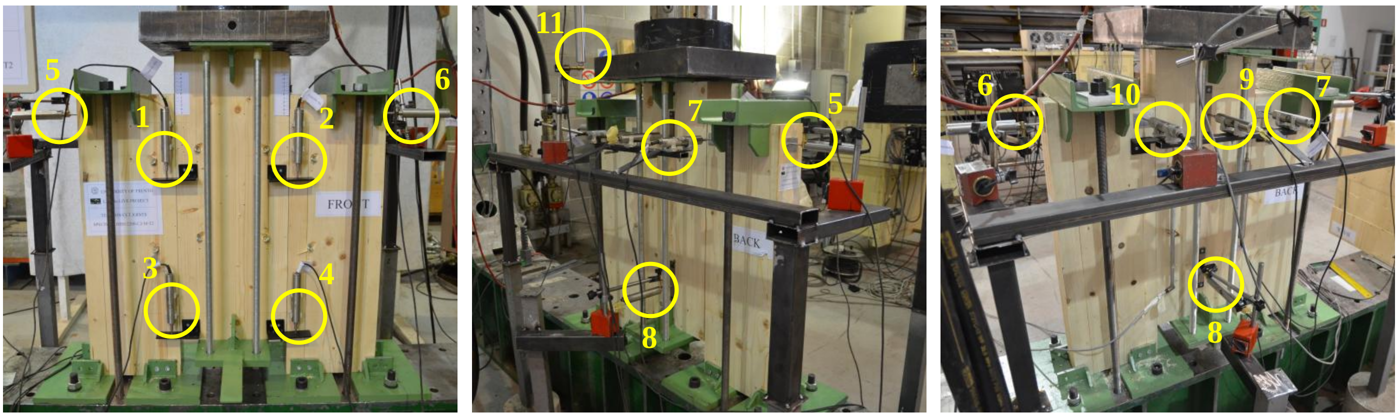
(b)



Instrumentation locations: 1, 2, 3, 4, 5, 8 **LVDT 50** and 6, 7, 9 **LVDT 100**

1, 2, 3, 4 horizontal position and 5, 6, 7, 8, 9 vertical position

(c)



Instrumentation locations: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10 **LVDT 50** and 11 **LVDT 100**

5, 6, 7, 8, 9, 10 horizontal position and 1, 2, 3, 4, 11 vertical position

Fig. 7. Test setup and reaction frame (a); instrumentation layout for beam-to-panel connections (b) and panel-to-panel connections (c).

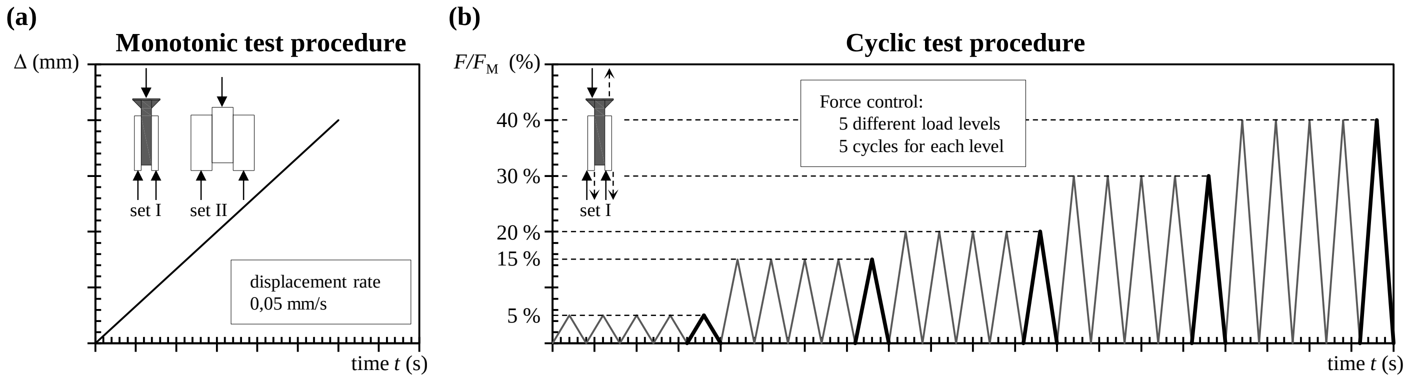


Fig. 8. Load protocols for (a) monotonic tests and (b) cyclic tests.

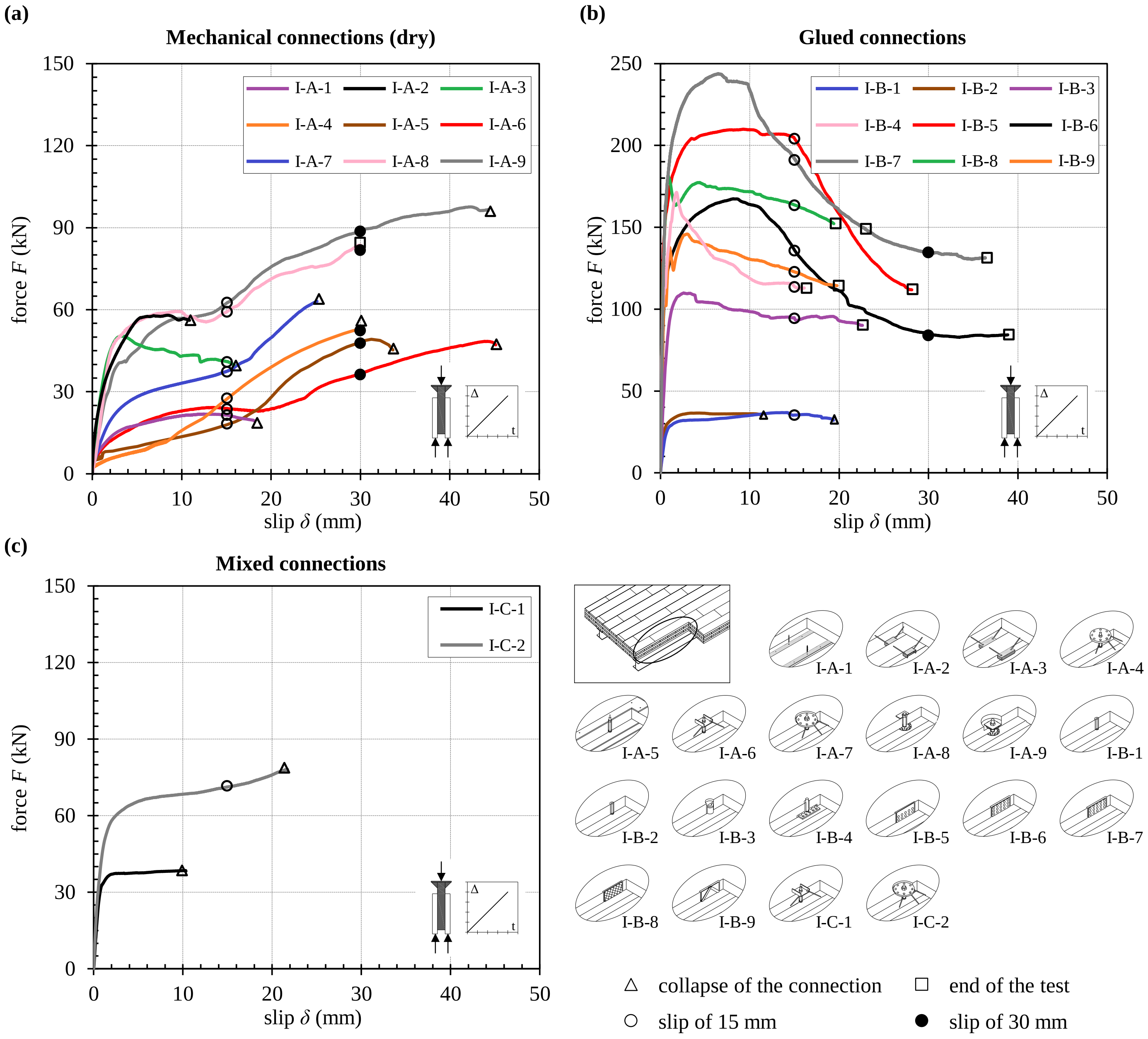


Fig. 9. Force-slip curves of the beam-to-panel connections.

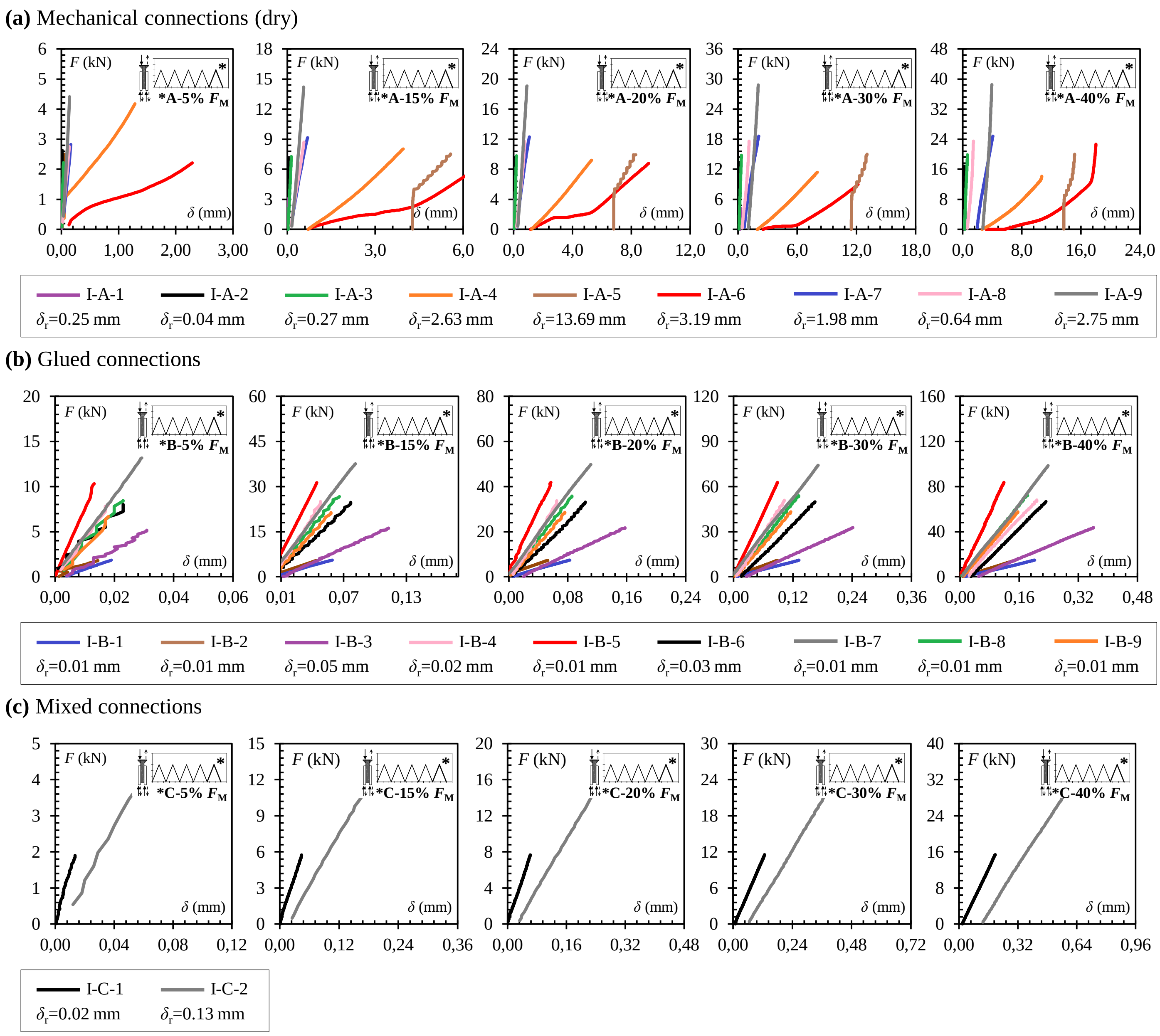


Fig. 10. Force-slip cyclic curves of the beam-to-panel connections with estimated residual slip (δ_r).

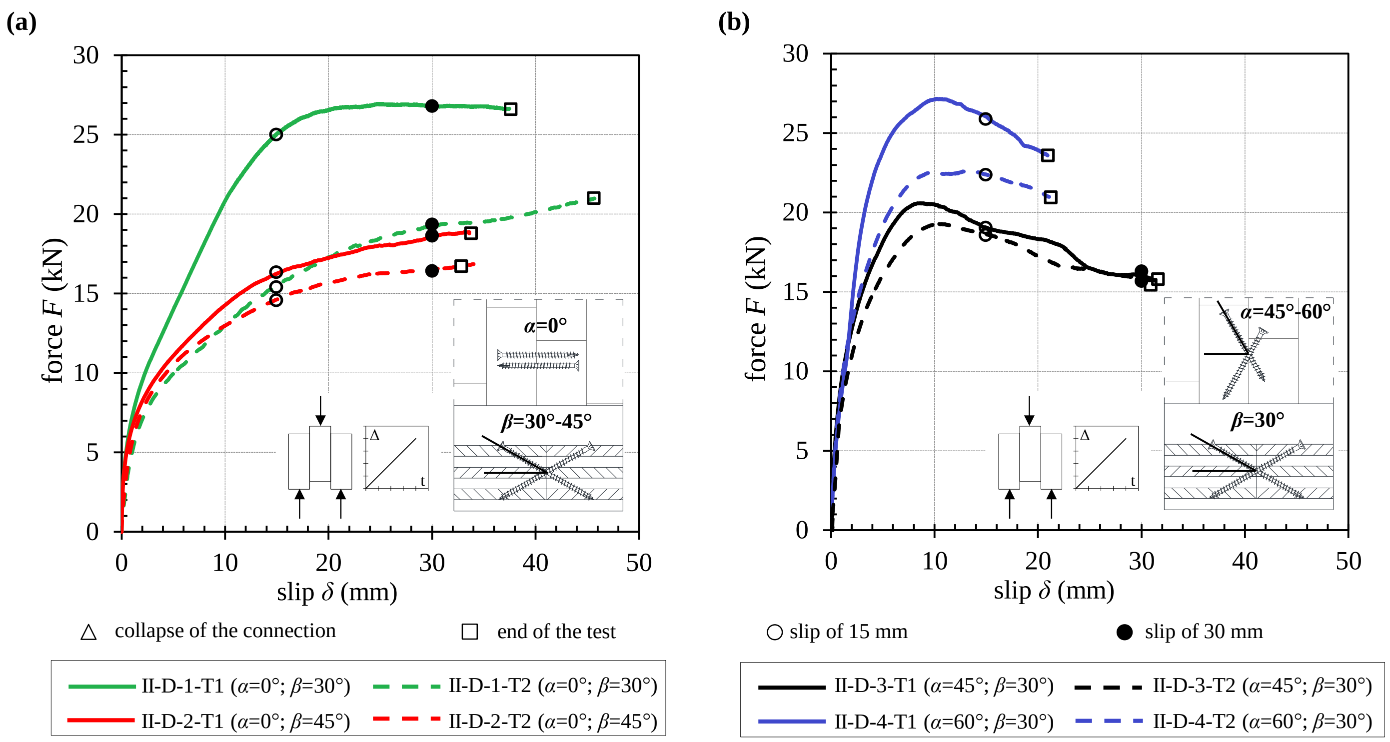


Fig. 11. Force-slip monotonic curves of the panel-to-panel connections.

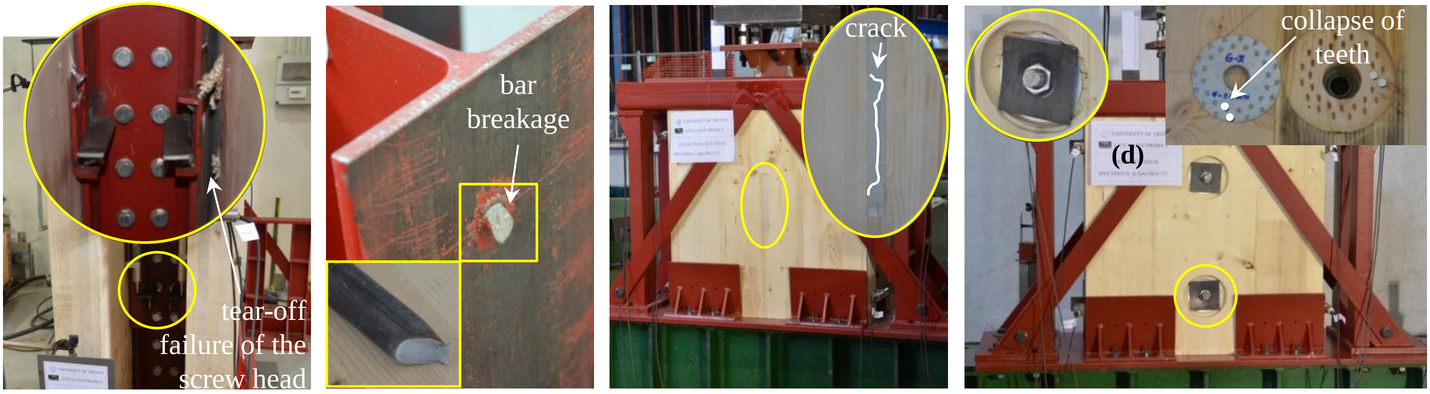


Fig. 12. Observed characteristic failure modes for the tested beam-to-panel connections.