Innovative construction system for sustainable buildings

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Abstract

This paper deals with a contemporary integrated and sustainable construction technology for new residential buildings. Specifically, this research aims at developing innovative steel-timber hybrid structures which allow a rapid assembly of the individual prefabricated components, minimizing the construction times and limiting the costs of the work. The numerical analyses performed on a multi-storey building for social housing will be presented and discussed. The in-plane behaviour of the floors and shear walls will be analysed, considering in particular the types and arrangement of the different timber- and steel-timber joints. The connections to be used among the construction elements will be selected in order to develop a sufficient stiffness, ductility and bearing capacity according to the design criteria for seismic-resistant structures. These connections allow to enhance the on-site assembly operations, therefore working effectively also under harsh climatic conditions.

Keywords: Hybrid structures; Composite structures; Wood-based structures; Modular construction; Prefabrication; Cross laminated timber panels; Sustainability; Green design; Steel-timber connections;

1 Introduction

In Europe, in accordance with the 'Horizon 2020' Project -research and innovation programme for strategic development- the buildings have to drastically reduce the energy consumed during the whole life cycle, as well as their related emission of carbon dioxide (CO2) into the atmosphere [1]. As part of this process, the building industry is promoting construction technologies for buildings developed according to an energy-efficiency perspective.

In this paper we refer to a contemporary integrated and sustainable construction technology with a modular and prefabricated steel-timber structure. It is a flexible solution with regards to the changing housing needs over time, with the possibility to modify the internal and external distribution of spaces, and therefore it allows to respond to a wide range of housing needs. In the field of steel-timber hybrid construction systems some recent works can be cited: Tesfamariam and Stiemer [2], Asiz and Smith [3], Dickof et al. [4], Bhat et al. [5], He et al. [6] and Okutu et al. [7]. However, this research aims at developing innovative hybrid building systems which allow a rapid assembly of the individual prefabricated building elements, minimizing the construction times and limiting the costs of the work.

This document includes some of the results obtained from the numerical analyses performed on a multi-storey residential building. In particular, the aspects that affect the in-plane behaviour of the floors and shear walls are studied. Several arrangements of the connections will be analysed

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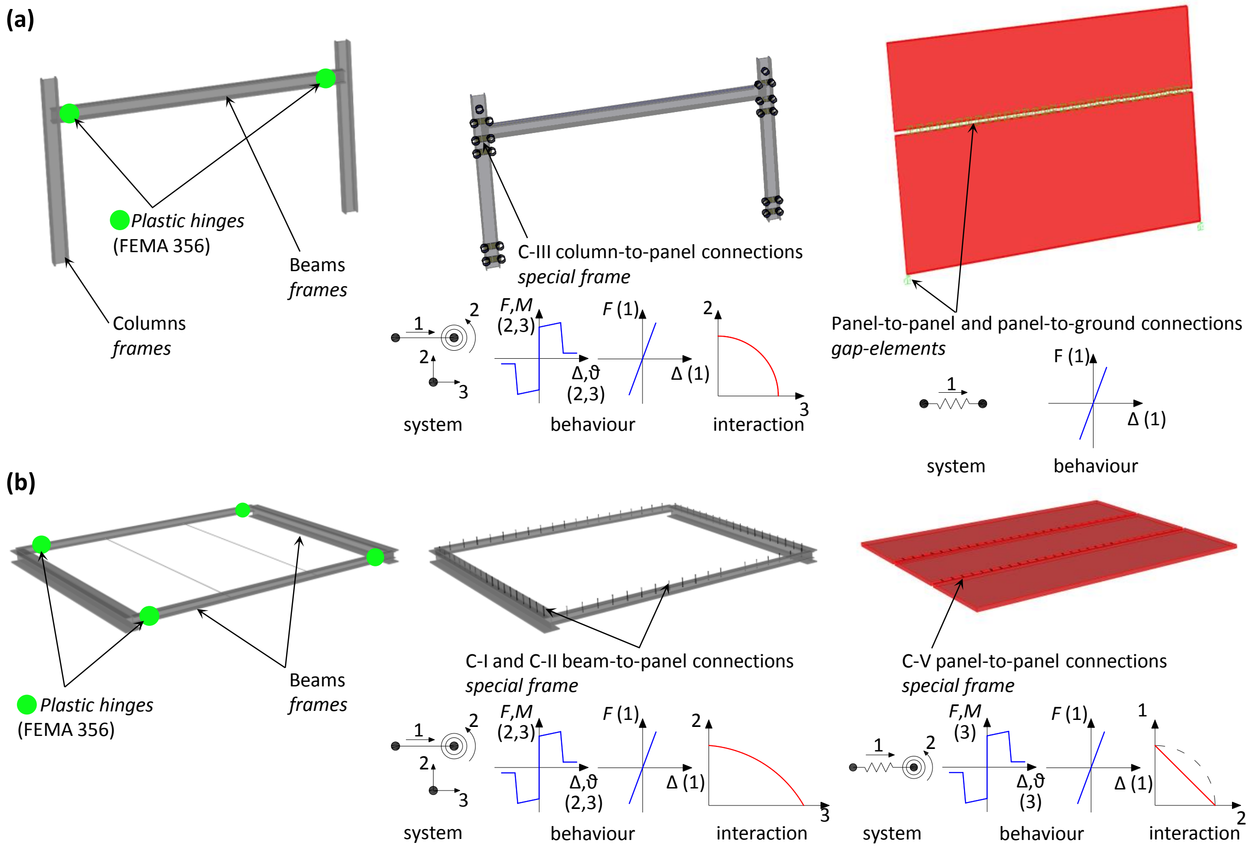

Figure 2. Finite Element Model (FEM) implementation for shear walls (a) and diaphragms (b)

The model was developed taking into account the exact geometry of elements and connections, as well as considering the structural components with lumped plasticity. With reference to Figure 2, the bare steel frames and grid were modelled using *frame* elements and considering two-dimensional *shell* elements for the CLT panels. The connections of the construction system were reproduced with a set of *special frames* defined in order to represent their actual behaviour. The load-slip curves measured in the experimental tests were replicated using equivalent mechanical properties and geometry of the *special frames*.

The model was constructed considering plastic hinges located at the ends of the steel beams and columns. For steel elements, plastic hinges were defined in accordance with FEMA 356:2000 (*Prestandard and Commentary for the Seismic Rehabilitation of Buildings*). The inelastic response of the timber-to-timber and steel-to-timber joints was reproduced with plastic hinges distributed at the top and bottom of the *special frames*. In order to replicate the shear mechanism of the joints in all directions, the plastic hinges were defined considering several surfaces of interaction based on engineering practice (Figure 2). In the FEM model other special *link* elements were used to account for the interaction of CLT panels at their edge surfaces or at the foundation level.

In the model, CLT panels were considered as built with 5 layers of C24 (EN 338: 2009) timber boards (lamella structure mm: 20/20/20/20/20), while the hot-rolled steel elements were made of mild steel S275 (EN 10025: 2004). The stress-strain curve of steel complies with the Italian Building Code [9]. The mechanical properties of wood were modelled considering an orthotropic elastic behaviour and an equivalent thickness of elements which takes into consideration the actual elasticity and shear modulus [10]. Table 1 shows the mechanical properties of materials used in the analyses, while the element dimensions are illustrated in Figure 3.

In the FEM model the contribution of secondary beams and intermediate vertical supports was neglected.

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4.1 In-plane behaviour of shear walls

A parametric study was conducted varying types of connections, spacing, arrangement of both the joints and the CLT panels, dimensions of panels and edges of the panels fastened to the steel frames. Around 300 nonlinear static analyses were performed in order to obtain the best solution in terms of resistance and ductility capacity.

The results highlight that the response is better when connections are located at the corners of the CLT panels than when they are distributed along the panel sides (Figure 5). Based on the previously defined set of connections, solution C-III proves to be a better choice in terms of seismic performance than other C-I and C-II solutions. More in detail, to develop a global ductile behaviour of the mixed steel-timber shear walls, the connections between the columns and the panels shall be placed according to the configuration of Figure 6 (labelled SW-1).

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 the elements installed at the corners and moving towards the central part of the panel sides. C-I and C-II solutions do not have enough ductility capacity to activate all the plastic deformation before the most loaded connectors reach their failure. Therefore, the solutions with connections distributed along edges, C-I and C-II, need to be re-designed in order to provide a sufficient plastic capacity.

4.2 In-plane response of the floors

In this paper particular attention was paid to the evaluation of the in-plane behaviour of the floors. A parametric study was carried out in order to evaluate the diaphragm stiffness and bearing capacity of floors loaded with a horizontal uniform force distribution. 70 nonlinear static analyses were performed on floors which feature different joints used to fasten the CLT panels to the steel beams, panel-to-panel connections, spacing, as well as number and arrangement of shear walls within the building.

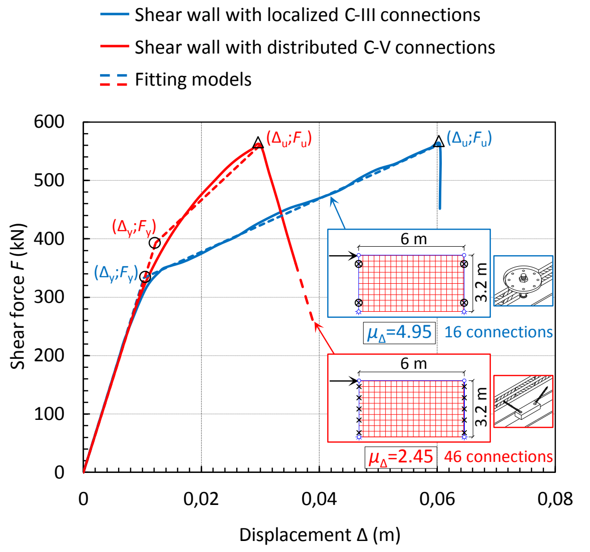

Figure 5. Nonlinear behaviour of shear walls built using two different joint configurations

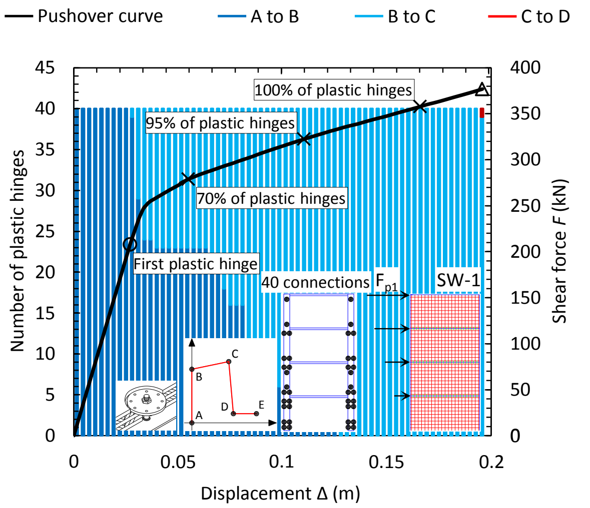

Figure 6. Ductile hybrid steel-timber shear wall

Figure 7 shows the load-displacement curves of two different floors extracted from the analyses for the two main directions. Specifically, Figure 7 depicts the FL-1 configuration of the floor adopted in the successive analyses.

The FL-1 floor arrangement was constructed joining CLT panels together at their edges with C-V connectors installed at a constant pitch of 200 mm, while the connections between steel elements and CLT panels were made using C-I connectors placed at a constant spacing of 300 mm. The floor was fixed to the ground in eight positions according to the design shear walls scheme of Figure 7

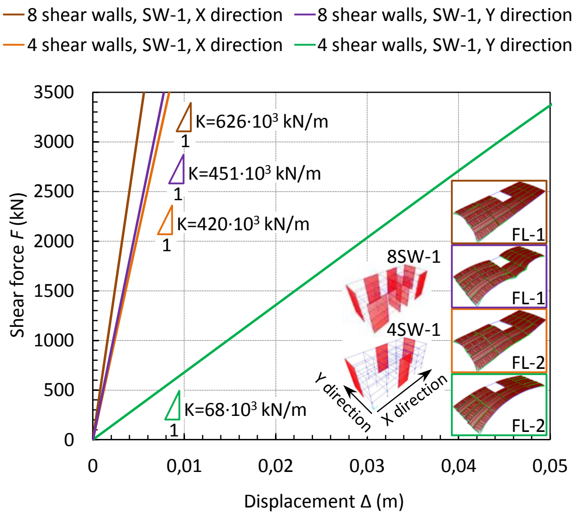

Figure 7. Stiffness and load-carrying capacity of two designed floors under in-plane loads

4.3 Three-dimensional response of the construction system

The diaphragm plays a key role regarding the in-plane forces distribution on the individual vertical bracing elements. More specifically, rigid diaphragms allow the distribution of the horizontal forces according to the bracing stiffness; also the masses and the moments of inertia of each floor can be lumped at their centre of gravity. Therefore, the complexly of the seismic structural analysis is reduced.

The last study presented here concerns the evaluation of the global behaviour of the building. Several nonlinear analyses were performed considering a floor with an “ideal” rigid behaviour and with its actual in-plane flexibility. Figure 8 shows the load-displacement curves extracted from the analyses for each main direction of the building. The storey shear force profile of the most loaded shear wall is reported in Figure 9. In addition, Table 2 depicts some structural parameters evaluated for two different displacement levels: DL1 and DL2 corresponding to the first and second damage state, respectively. For displacements within this range, from DL1 to DL2, the structure exhibits a consistent deformation capacity and energy dissipation (statically-determined), and the displacement ductility can rise up to about 10.

With specific reference to the monotonic load-displacement curves and the related shear force profiles of the building, Figures 8 and 9, together with the data of Table 2, show that the in-plane behaviour of diaphragms is close to rigid. As a matter of fact, the building complies with the rigid diaphragm requirement of Eurocode 8 [8] for the X direction. Otherwise, in the Y direction the difference of the load-displacement curves between the model with in-plane flexible and rigid diaphragms exceeds the recommended value of 10%. In the design perspective (Figure 9), the storey forces measured in the most loaded bracing element for the DL2 level can be underestimated of about 8% in average. However, the difference of the shear forces can locally rise up to about 40%, as per Y-direction wall depicted in Figure 9.

In this work the FEM model of the building was developed leaving out some elements such as secondary beams, which can increase the in-plane stiffness of floors. On the basis of this sensitivity study, new analyses are ongoing using a more refined FEM model.

Table 2. Three-dimensional response of building with flexible and rigid diaphragms (Δ, displacement; F, force)

DL1 (fixed force, displacement comparison)				
		Δ_{actual} (mm)	**Δ_{rigid} (mm)**	**difference (%)**
Pattern 1	X direction	27.76	26.25	5.74
	Y direction	28.28	23.12	22.30
Pattern 2	X direction	23.73	22.50	5.48
	Y direction	26.84	22.50	19.29
DL2 (fixed displacement, force comparison)				
		F_{actual} (kN)	**F_{rigid} (kN)**	**difference (%)**
Pattern 1	X direction	1792.03	1768.14	1.35
	Y direction	1506.61	1677.58	10.19
Pattern 2	X direction	2145.05	2096.60	2.31
	Y direction	1996.69	2218.54	10.00

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