World Conference on Timber Engineering Oslo 2023

SEISMIC DESIGN OF A CLT MULTI-STOREY BUILDING IN DIFFERENT DUCTILITY CLASSES

Giuseppe Cenci[1], Maurizio Follesa[2], Vincenzo Rinaldi[3], Daniele Casagrande[4], Massimo Fragiacomo[5]

ABSTRACT: This study aims to clarify fundamental concepts related to the design of Cross-Laminated Timber (CLT) multi-storey buildings in seismic-prone areas. An in-depth worked example of a CLT multi-storey building design is presented following four different cases of analysis: low and medium ductility according to the current Italian regulations (NTC 2018), and medium and high ductility (DC2 and DC3) according to the new draft of Eurocode 8. Finite Element (FE) models of the building have been implemented to analyse the linear dynamic behaviour of the structure. In high ductility class, a couple-panel behaviour of shear walls is achieved by following the analytical provisions reported in the new draft of the Eurocode 8. As a result, a clear design process is defined for each case analysed. Through the analysis of the individual steps, possible issues in the interpretation and application of the rules are highlighted and resolved. Finally, this study allows an easy and direct comparison between the various cases.

KEYWORDS: CLT buildings, Seismic analysis, Capacity-based design, Practice-oriented.

1 INTRODUCTION

The use of Cross-Laminated Timber (CLT) panels has become widespread for the construction of low-to-mid-rise multi-storey buildings, representing a valuable alternative to other traditional structural types also in seismic-prone areas [1,2]. However, the development of Standard provisions framing the design and calculation of this type of structure is still lacking in Europe, especially for seismic load situations.

1.1 DPC-ReLUIS 2019-2021

Through the DPC-ReLUIS 2019-2021 research project, the Italian Civil Protection Department and the ReLUIS research consortium have promoted the development of pre-normative documents related to structural types not yet considered in the current standards. They have also assisted in the actual drafting of the technical standards. Concerning CLT structures, the aim is to produce effective documentation to support normative review [3]. Therefore, an in-depth worked example of a CLT multi-storey building has been prepared by applying both the Italian Technical Regulations for Construction (NTC 2018) [4] and the new draft of the Eurocode 8 still under revision (prEN 1998-1-2: 2020) [5,6].

2 SCOPE

The primary purpose of the aforementioned document is to clarify fundamental concepts related to the design of CLT buildings in seismic areas, such as: (i) dissipative mechanisms, allowing the use of a specific value of the behaviour factor q; (ii) the minimum ductility values that the dissipative connections should exhibit; (iii) the design conditions for attaining those minimum ductility values; (iv) the over-strength factors used in the design of the non-dissipative components; (v) the partial safety factors to be used in the design while taking into account the strength reduction due to cyclic loading. Therefore, the objective of the present study is to clarify the practical application of the capacity-based design by defining unambiguous procedures.

3 ANALYSIS CASES

A 5-storey CLT residential building (Figures 1, 2 and 3) is designed following four different cases of analysis: low and medium ductility class in accordance with the current Italian regulations (NTC 2018), and medium and high ductility (DC2 and DC3) according to the new draft of Eurocode 8.

[1] Giuseppe Cenci, University of L'Aquila, Italy, cenci.giuseppe@outlook.it
[2] Maurizio Follesa, dedaLEGNO, Italy, follesa@dedalegno.com
[3] Vincenzo Rinaldi, University of L'Aquila, Italy, vincenzo.rinaldi@univaq.it
[4] Daniele Casagrande, National Research Council of Italy CNR-IBE, Italy, daniele.casagrande@ibe.cnr.it
[5] Massimo Fragiacomo, University of L'Aquila, Italy, massimo.fragiacomo@univaq.it

https://doi.org/10.52202/069179-0296

Figure 1: *Rendered view of the case-study building*

The table below summarises the different cases analysed and the corresponding adopted values of the behaviour factor q.

Table 1: *Analysis cases*

Case	Code	Ductility class DC	Behaviour factor q
1	NTC2018	Low	1.5
2.1	NTC2018	Medium	2.5
2.2	prEN 1998-1-2:2020	2	2.0
3	prEN 1998-1-2:2020	3	3.0

The walls with grey hatching in figure 3 are segmented in DC 3.

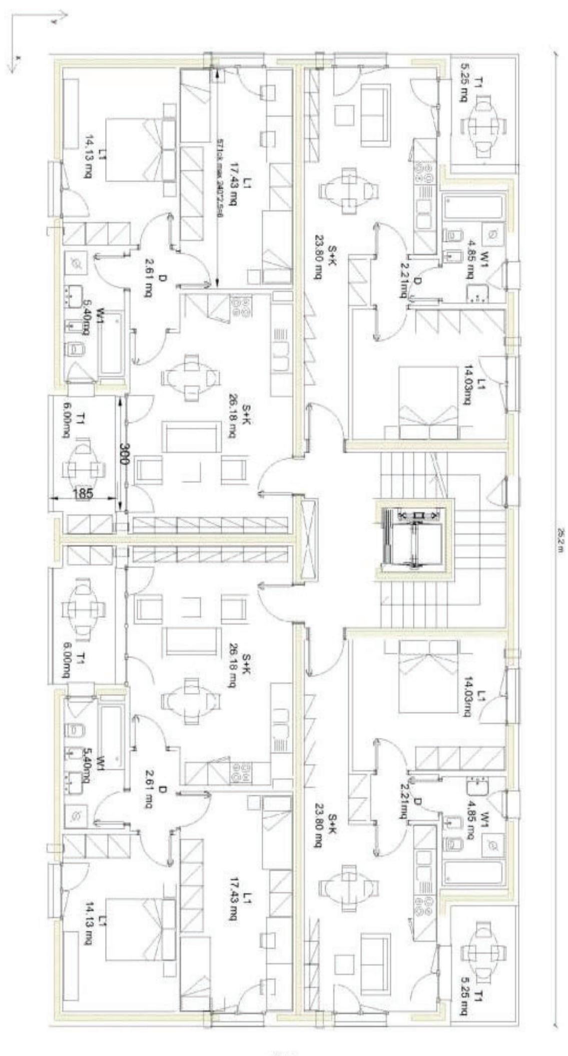

Figure 2: *Architectural plan of a typical floor*

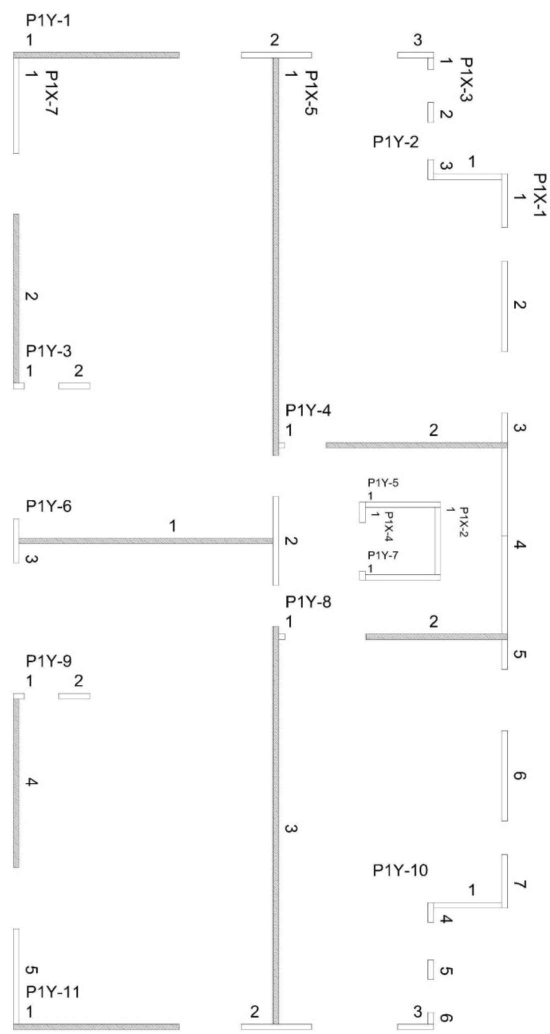

Figure 3: *CLT walls*

4 DESIGN DATA

4.1 THE BUILDING

The building is composed of five storeys, with an inter-storey height of 3.12 m; the total height of the buildings is 17.5 m. The structure is built using CLT panels (walls and slabs); the shear walls are designed to support both gravitational and lateral loads (wind and seismic action). The presence of many openings (windows and balconies) results in a segmentation of the shear walls that was addressed to define the Seismic Force Resisting System. The stairwell and the elevator shaft are also made of CLT panels; the connections with the diaphragm have been designed to ensure an adequate in-plane behaviour of floors. Neither the foundation system nor the soil-structure interaction are taken into account in the analysis. Three different types of panels are used, as shown in the table below.

Table 2: *CLT layup and thickness*

Element	Layup	Thickness
Slab	7 layers	220 mm
Walls level 1 to 3	5 layers	140 mm
Walls level 4 to 5	5 layers	120 mm

4.2 GRAVITY LOADS AND SEISMIC ACTION

Table 3 lists the values of the gravity loads acting on the building while Table 4 reports the parameters of the pseudo-elastic spectrum for the two limit states considered: Significant Damage SD (SLV according to NTC18) and Damage Limitation DL (SLD according to NTC18). Values of the seismic actions are selected with reference to common real cases in Italy. The peak ground accelerations for SD and DL have a probability of exceedance of 10% and 63% in 50 years, respectively. The site is mainly flat (topographic amplification factor S_t = 1) with a soil class C (180 m/s < $v_{s,30}$ < 360 m/s).

Table 3: *Gravitational loads*

Element	G_2	Q_{LIVE}
Slab	2.5 / 3.0 kN/m^2	2.0 / 4.0 kN/m^2
Roof	0.5 kN/m^2	-
Walls	0.5 kN/m^2	-

Table 4: *Seismic action parameters*

Limit state	a_g/g	F_0	T_C*
SD (SLV)	0.266 g	2.312	0.351
DL (SLD)	0.101 g	2.336	0.284

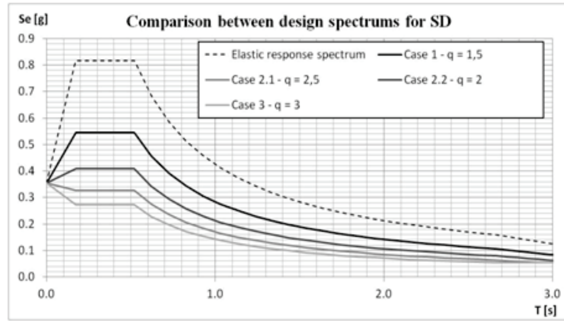

Figure 4: *Comparison between design spectrums for SD*

Given the interest on the seismic behaviour, only the seismic combination is considered, including all the possible variable loads.

$$E + G_1 + G_2 + P + \psi_{21} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \psi_{33} \cdot Q_{k3} \ldots$$

5 DESIGN PRINCIPLES AND FEM MODEL

The seismic design principles used in this example are based on the results of several research projects and of some full-scale tests on earthquake shaking tables (e.g. SOFIE project, IVALSA-CNR institute) [7]. These tests also enabled some modelling methods to be validated, like the one mentioned in the next paragraph.

5.1 FINITE ELEMENT MODEL

A 3D Finite Element (FE) model of the building is defined for each analysis case following the method proposed in [8]. Each model simulates the behaviour of the real structure in the context of linear dynamics.

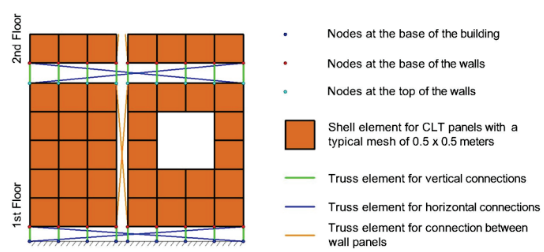

Figure 5: *Typical wall schematisation*

https://doi.org/10.52202/069179-0296

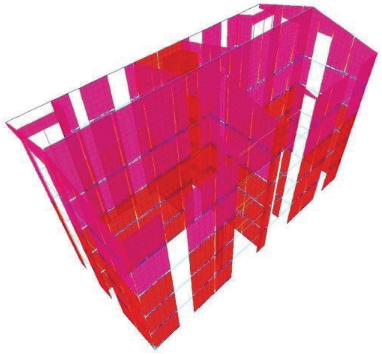

Figure 6: Finite Element model

Through an iterative process, it is possible to check the force level of the connections and update the corresponding deformability values which strongly affect the structure's vibration modes.

6 CAPACITY-BASED DESIGN

The design of dissipative seismic-resistant structures requires that a portion of the energy transferred by the earthquake to the structure is dissipated through the plasticisation of some connections. In this case, the seismic actions can be reduced provided that the Lateral Load Resisting System (LLRS) is able to provide an extra post-elastic resistance through well-localised plastic deformations. This means that the plasticisation phenomena must not make the structure unstable and must not compromise its overall behaviour. To achieve this goal, all the elements that must not exceed the elastic phase are designed with an over-strength factor compared to those with ductile behaviour, so that the dissipative phenomena will concentrate on the latter one [9,10].

6.1 OVER-STRENGTH LEVELS

As described below, the over-strength condition affects a structure at different levels:

- Single connector: in connection with Johansen's formulas, the brittle failure modes must have adequate over-strength with respect to ductile failure mode, represented by the plasticisation of the fasteners through one or more plastic hinges.
- Connections: the brittle failure mode of an entire connection must have adequate over-strength with respect to ductile failure mode. Typical brittle failure modes are shear failure in the carpentry nodes, shear or tension failure in wood sections, tensile or shear failure of metal plates and shear or punching shear failure of bolts and screws. Ductile failure modes are typically shear failures of dowel-type fasteners characterised by at least a plastic hinge formation;
- Structural elements and global behaviour: collapse of structural elements with non-dissipative behaviour and the formation of soft storey mechanisms are avoided, enhancing the overall behaviour of the entire structure.

6.1 HIGH DUCTILITY CASES (DC3)

In the new draft of Eurocode 8, a high ductility class (DC3) is proposed for CLT buildings consisting of LLRS made with multi-panel shear walls. In these walls, the panels are connected together with vertical joints that can dissipate energy primarily through the yielding of fasteners, leading to a coupled-panel (CP) behaviour where all the panels remain in contact with the ground or the floor below and a significant slip of vertical joints is attained. In order to ensure that CP behaviour is also attained in the plastic region, vertical joints have to be designed to yield before the hold-downs whereas angle brackets have to be slightly over-designed so that the sliding failure mechanism of the wall is prevented. Non-dissipative elements, typically characterised by a brittle failure mode, are designed to remain elastic when the vertical joints attain the desired level of inelastic displacement according to the capacity-based design approach.
The analytical expressions proposed by Casagrande et al. [9] are used in the draft of the new Eurocode 8 to ensure that CLT shear walls of buildings designed in DC3 attain the desired kinematic mode.

7 DISSIPATIVE CONNECTIONS

Dissipative connections must ensure proper energy dissipation; it is therefore important to analyse the different failure mechanisms in order to provide sufficient over-strength to those that exhibit brittle behaviour.

7.1 ANGLE BRACKETS FOR SHEAR LOAD

Angle brackets for shear load connect the wall panels to the floor panels or to the concrete foundation. To ensure the desired behaviour, the following failure mechanisms have been analysed:

- Shear failure of the steel plates;
- Failure of the plate-to-wall dowel-type connection;
- Failure of the plate-to-slab dowel-type connection.

Four types of angle brackets manufactured by Rothoblaas have been analysed in combination with two different types of fasteners (LBA 4 x 60 mm high bond nails and LBS 5 x 70 mm screws):

- TCN240 for timber-to-concrete connections;
- TTN240 for timber-to-timber connections;
- TTV240 for timber-to-timber connections with reinforcing screws that absorbs the eccentricity in the vertical plate of the bracket;
- TTF200 for timber-to-timber connections for lower loads.

In this case, the energy dissipation is exclusively located in the plate-to-wall screwed connection. The NTC2018 and the draft of the Eurocode 8 used for this work provide different over-strength factors, 1.3 and 1.6, respectively.

https://doi.org/10.52202/069179-0296

These values considerably affect the design resistance of the connection, as shown in the table below.

Table 5: *Angle brackets resistance in each case*

Angle bracket	Non-dissipative joints (from ETA)		Dissipative joints	
	Case 1	Cases 2.1, 2.2, 3	Case 2.1	Cases 2.2, 3
TCN240 + LBA	23.8 kN	33.3 kN	26.8 kN	26.8 kN
TCN240 + LBS	28.5 kN	39.9 kN	33.0 kN	33.0 kN
TTV240 + LBA	46.9 kN	65.7 kN	39.0 kN	31.7 kN
TTV240 + LBS	46.9 kN	65.7 kN	39.1 kN	31.8 kN
TTN240 + LBA	29.8 kN	41.7 kN	26.8 kN	21.8 kN
TTN240 + LBS	36.7 kN	51.4 kN	33.0 kN	26.9 kN
TTF200 + LBA	27.9 kN	39.1 kN	23.4 kN	19.0 kN
TTF200 + LBS	33.4 kN	46.8 kN	29.0 kN	23.6 kN

7.2 HOLD-DOWNS AND TIE-DOWNS

The anchoring connections against uplift (hold-downs, tie-downs) have been similarly analysed. Tie-down anchors through the floor panels have been used in combination with screwed connections between the steel plate and the CLT wall panel. The following failure mechanisms have been analysed:

- Tensile failure of the steel plate;
- Failure in shear of screwed connections between metal plate and CLT panel.

7.3 VERTICAL JOINTS BETWEEN WALL PANELS

In case 3 (high ductility case DC3), the increased demand for energy dissipation is ensured by the panel-to-panel connections; such joints are activated by the coupled-panel (CP) behaviour previously mentioned. The capacity-based design for a segmented wall requires that the vertical joints must yield before the anchoring connections against uplift [11,12]; this condition strongly affects the design of vertical joint between the wall panels. The half-lap joint replaces the step joints made with a cross-layer stripe inserted in grooves on one side of the CLT panels. The step joints are used only for low and medium ductility cases.
The shear force acting on the vertical joints has been calculated by isolating each individual wall (continuous wall from the foundation to the roof) and analysing it as disconnected from the rest of the structure. The actions (lateral load and gravitational loads for seismic combination) are taken from the global model. Once the loads are defined, it is possible to design the joints between adjacent wall panels by using the analytical expressions proposed by Casagrande et al. [9]

8 LINEAR DYNAMIC ANALYSIS

The stiffness of timber structures, particularly the CLT platform-frame type, is markedly affected by the deformability of the connections. However, this deformability can only be determined after the design of the connection itself.
The first step involves the use of a linear static analysis; this analysis allows for the "calibration" of the connections' resistance through a preliminary design.
Both the NTC2018 and the new draft of the Eurocode 8 propose Equation (1) as a method to identify the building's fundamental period of vibration.

$$T_1 = 0.05 \cdot h^{\frac{3}{4}} \tag{1}$$

where h is the height of the building, in [m], measured from the foundation or from the top of a rigid basement.
Once the connections are preliminarily designed, it is possible to calculate the related deformations that will be used to complete the numerical model. The dynamic analysis that, through an iterative process, leads to the convergence of the solution can then be run.

9 ANALYSIS OF RESULTS

The results obtained make it possible to compare the cases analysed, evaluating the differences in relation to various aspects.

9.1 MODAL ANALYSIS

A comparison of the results of the modal analysis demonstrates that the differences between the vibration periods of cases 1, 2.1 and 2.2 are negligible (in the order of a hundredth of a second). On the contrary, for case 3 there is an increase of about one-tenth of a second in the first vibration period; this result reflects the larger deformability of the structure due to the wall segmentation. The following table summarises the values for the first three modes of vibration.

Table 6: *Vibration periods*

Mode	Vibration periods	
	Cases 1, 2.1, 2.2	Case 3
1	0.384 s	0.486 s
2	0.363 s	0.479 s
3	0.294 s	0.398 s

The mode shapes do not present significant variations; the first three mode shapes are translational along the two horizontal directions and rotational along the vertical axis.

https://doi.org/10.52202/069179-0296

9.2 CONNECTIONS

The following subparagraphs compare the loads obtained for the different types of connections. Two shear walls have been chosen for the sake of synthesis: the first P1X-5-1 is a long wall on the first level, with significant gravitational loads; the second P4Y-11-1 is a short external wall on the fourth level, only slightly loaded in terms of vertical loads.

9.2.1 Angle brackets

The following table shows the number of angle brackets for the two walls analysed. It is interesting to compare the values relating to cases 2.1 and 2.2: the number of angle brackets in case 2.2, already increased due to the lower behaviour factor, is further amplified by the lower performance of the angle brackets themselves, caused by the greater over-strength factor.

***Table 7:** Number of angle brackets per wall*

Wall (angle bracket)	Number of angle brackets (load)			
	Case 1	Cases 2.1	Case 2.2	Cases 3
P1X-5-1 (TCN240)	14 (535 kN)	9 (324 kN)	13 (380 kN)	4 (118 kN)
P4Y-11-1 (TTN240)	5 (182 kN)	3 (110 kN)	5 (139 kN)	5 (110 kN)

9.2.2 Hold-down/Tie-down

In the case of anchoring connections against uplift, the design loads are shown instead of the connection setup. This is because in case 3, the connection setup was changed to accommodate the needs of the capacity-based design in high ductility.

***Table 8:** Uplift forces in hold-down / tie-down*

Wall	Uplift forces			
	Case 1	Cases 2.1	Case 2.2	Cases 3
P1X-5-1	26 kN	0 kN	0 kN	4 kN
P4Y-11-1	69 kN	32 kN	46 kN	42 kN

9.2.3 Non-dissipative connections

For medium and high ductility, the non-dissipative connections must ensure sufficient over-strength in order to remain elastic when the dissipative connections attain the desired level of inelastic displacement. With Equation (2) it is possible to take into account the effective resistance of the dissipative connections before applying the desired over-resistance factor to it.

$$F_{Rd,b} \geq \frac{\gamma_{Rd}}{k_{deg}} \cdot \Omega_d \cdot F_{Ed,E} + F_{Ed,G} \tag{2}$$

where F_{Rd} = design resistance of the non-dissipative joints, γ_{Rd} = over-resistance factor, k_{deg} = strength reduction factor, $F_{Ed,E}$ = action effect in the non-dissipative joint, or member due to the design seismic action, $F_{Ed,G}$ = action effect in the non-dissipative joint or member due to the non-seismic actions in the design seismic situation, and Ω_d = minimum value of all the over-strength ratios $\Omega_{d,i}$, where i is the storey (see CEN/TC250/SC8 prEN 1998-1-2:2020 for further information).

Table 9 shows the results obtained for the joints between orthogonal walls; due to the heavy loads, these connections are made with angle brackets for shear forces; two connections were chosen in locations where the greatest loads occur.

It is important to point out that these connections are schematised with rigid links in the numerical model.

***Table 9:** Number of angle brackets for joints between orthogonal walls*

Wall (angle bracket)	Number of angle brackets (load)			
	Case 1	Cases 2.1	Case 2.2	Cases 3
P1X-5-1 (TTF200)	6 (169 kN)	4 (173 kN)	5 (231 kN)	4 (177 kN)
P4Y-11-1 (TTF200)	2 (66 kN)	2 (68 kN)	3 (96 kN)	2 (85 kN)

9.3 DESIGN COMPLEXITY

Finally, the design complexity of the different cases is analysed. After having calibrated the model through the linear static analysis, few iterations in the dynamic analysis are enough to make the solution converge for low and medium ductility. The angle brackets for shear loads represent the only deformable connections in the numerical model. With the results extracted from the model, it is possible to design all the joints.

In the case of high ductility, one more step must be added: once the global numerical model of the structure is completed, the loads acting on the different connections of the segmented walls are calculated through free body diagrams on each single wall. Horizontal and vertical loads acting on the wall can be extracted from the numerical model. These conditions must be validated through the equations proposed by Casagrande et al. [9] in order to attain the desired kinematic mode.

10 CONCLUSIONS

This study presents the design process of a multi-storey CLT building according to the methods and design provisions included in the new draft of the Eurocode 8. A detailed worked example of design is presented for four different cases of analysis, including also a comparison with the current Italian regulations (NTC 2018). The design process starts from the calibration of the global models through an equivalent static analysis up to the characterization of the main connections that this construction system requires.

Through the detailed analysis of each individual step, possible issues in the interpretation and application of the rules are highlighted. Some solutions complying with the requirements related to the various cases and ensuring ease of use in the application are then proposed. Such simplified solutions are indispensable for a practical design.

Finally, the results of the various cases analysed are compared. When it comes to capacity-based design, the effects of the over-strength factors strongly characterise the design of each connection as well as the choice of connection types. This over-strength requirement makes the implementation of the connections more difficult in some cases. Furthermore, these coefficients cancel out almost completely the advantage of using higher behaviour factors for all the non-dissipative joints and elements.

Therefore, some serious considerations should be made about the values of the over-strength factors to be used in the seismic design of highly redundant structures like multi-storey CLT buildings. The fairly high values currently proposed in the new Eurocode 8 draft, in fact, may lead to a limited use of these structures in seismic prone areas. On the other hand the full-scale tests conducted so far demonstrated that such structures are capable to withstand severe seismic actions with limited damage and high dissipative behaviour.

Several issues related to the modelling process are also analysed in order to verify the underlying hypotheses and their correspondence with the experimental results.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the financial contribution of the DPC-ReLUIS 2019–21 and 2022-24 Research Projects, WP13 “Timber Structures – Regulatory Contributions”.

REFERENCES

[1] Izzi, M., Casagrande, D., Bezzi, S., Pasca, D., Follesa, M., Tomasi, R.Seismic behaviour of Cross-Laminated Timber structures: A state-of-the-art review (Open Access) (2018) Engineering Structures, 170, pp. 42-52.

[2] Pei, S., Van De Lindt, J.W., Popovski, M., Berman, J.W., Dolan, J.D., Ricles, J., Sause, R., (...), Rammer, D.R. Cross-Laminated Timber for Seismic Regions: Progress and Challenges for Research and Implementation (2016) Journal of Structural Engineering (United States), 142 (4), art. no. E2514001.

[3] Cenci, G., Follesa, M., Rinaldi, V., Casagrande, D., Fragiacomo, M.: Edifici in legno a pannelli massicci: Esempio di calcolo di un edificio a pannelli massicci in zona sismica. Report ReLUIS WP13-Task3, 2022.

[4] C.S.LL.PP. [2018] “Aggiornamento delle norme tecniche per le costruzioni” Supplemento ordinario alla “Gazzetta Ufficiale” n. 8 del 20 febbraio 2018 - Serie generale – n. 42 (In Italian.)

[5] CEN/TC250/SC8. prEN 1998-1-2:2020: Eurocode 8: Design of structures for earthquake resistance. Part 1-2: Rules for new buildings. Draft document. European Committee for Standardization; 2021.

[6] Follesa, M., Fragiacomo, M., Casagrande, D., Tomasi, R., Piazza, M., Vassallo, D., Canetti, D., Rossi, S. The new provisions for the seismic design of timber buildings in Europe (Open Access) (2018) Engineering Structures, 168, pp. 736-747. doi: 10.1016/j.engstruct.2018.04.090

[7] Ceccotti, A., Sandhaas, C., Okabe, M., Yasumura, M., Minowa, C., Kawai, N.SOFIE project - 3D shaking table test on a seven-storey full-scale cross-laminated timber building (2013) Earthquake Engineering and Structural Dynamics, 42 (13), pp. 2003-2021. doi: 10.1002/eqe.2309

[8] Follesa M., Christovasilis I.P., Vassallo D., Fragiacomo M., Ceccotti A.: Seismic design of multi-storey cross laminated timber buildings according to Eurocode 8. Ingegneria Sismica, vol. 30, 2013.

[9] Casagrande, D., Doudak, G., Polastri, A. A proposal for the capacity-design at wall- and building-level in light-frame and cross-laminated timber buildings, (2019) Bulletin of Earthquake Engineering, 17 (6), pp. 3139-3167.

[10] Jorissen, A., Fragiacomo, M. General notes on ductility in timber structures (2011) Engineering Structures, 33 (11), pp. 2987-2997. doi: 10.1016/j.engstruct.2011.07.024

[11] Masroor, M., Doudak, G., Casagrande, D.The effect of bi-axial behaviour of mechanical anchors on the lateral response of multi-panel CLT shearwalls (2020) Engineering Structures, 224, art. no. 111202.

[12] Masroor, M., Doudak, G., Casagrande, D. Design of Multipanel CLT Shear Walls with Bidirectional Mechanical Anchors following Capacity-Based Design Principle, Journal of Performance of Constructed Facilities Volume 36, Issue 11 February 2022 Article number 04021113

https://doi.org/10.52202/069179-0296