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THE HIGH-DUCTILITY CLASS (DC3) DESIGN PROCEDURE IN THE NEW EUROCODE 8 FOR CLT BUILDINGS: A CASE STUDY

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ABSTRACT: This study aims to discuss the high ductility (DC3) design procedure for Cross-Laminated Timber (CLT) multi-storey buildings included in the draft document of the second generation of Eurocode 8. Through the analysis of the individual steps, possible issues in the interpretation and application of the rules of the DC3 design protocol are highlighted and resolved. Furthermore, a new method for medium ductility design (DC2+) is proposed; the goal is to define a simplified calculation method for segmented walls that allows the use of a higher behaviour factor than the one proposed for the DC2 and a lower computational burden than the DC3. The main steps of this phase have been the verification of the proposed behaviour factor through a pushover analysis on various single wall structural archetypes, and the impact of the new method on the connection design and on the computational demand.

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

1 – INTRODUCTION

According to the draft of the new Eurocode 8 [1], Cross-Laminated Timber (CLT or XLAM) structures made with segmented shear walls can be designed in high ductility class (DC3) when energy dissipation occurs primarily in the vertical joints between the panels. Analytical expressions are provided to ensure a coupled-panel kinematic mode of the shear walls establishing a hierarchy of yielding between the vertical joints and the hold-downs and angle brackets. Capacity design procedures are also introduced to protect non-dissipative components and ensure that yielding occurs first in the vertical joints.

This study aims to discuss the design process of CLT structures in DC3 according to the new Eurocode 8. Furthermore, a new design method is proposed for an intermediate ductility class between the medium (DC2) and high (DC3) ductility class, namely DC2+. The method applies to CLT structures with segmented shearwall and is based on a simplification of the capacity based design procedure.

The analysis is based on the design of a five-storey residential case-study building. This study has been conducted within the framework of the DPC-ReLUIS 2021–2023 research project.

2 – DESIGN PROCEDURES FOR CLT STRUCTURES IN DC3

According to the new Eurocode 8, in high ductility class (DC3) CLT multi-storey buildings, seismic energy is primarily dissipated in vertical joints between the panels of

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segmented walls through the yielding of dowel-type fasteners. Therefore, these connections must be the first to yield. However, due to the various factors influencing the behaviour of the wall and the presence of nonlinear connection behaviour, a rigorous application of capacity-based design in this context may be challenging.

In this regard, analytical expressions are provided in the new Eurocode 8 to ensure that the yielding of connections is achieved first in the vertical joints between wall panels, then in the hold-downs, and finally in the shear connections at the base of the walls.

Equation (1) from [3] defines the hierarchy of yielding between the hold-downs/tie-downs and the vertical joints.

If $K_{SLS,anc} \geq n_{vj} \cdot K_{SLS,con}$

$$F_{Rd,h} \geq 1{,}1 F_{Rd,c} \frac{K_{SLS,anc}}{K_{SLS,con}}$$

If $K_{SLS,anc} < n_{vj} \cdot K_{SLS,con}$

$$F_{Rd,h} \geq max\left[1{,}1 F_{Rd,c} \frac{K_{SLS,anc}}{K_{SLS,con}}; 1{,}1\ n_{vj}\ F_{Rd,c} - w\ b_{CL}\right] \tag{1}$$

where:

- $F_{Rd,h}$ is the design resistance of the hold-down/tie-down nail plate, in accordance with equation (13.1) of [1].
- $F_{Rd,c}$ is the design resistance of one fastener used in the vertical joint, as per equation (13.1) of [1].
- $K_{SLS,anc}$ is the elastic stiffness of the hold-down/tie-down nail plate.
- $K_{SLS,con}$ is the elastic stiffness of one fastener used in the vertical joint.
- n_{vj} is the number of fasteners used in the vertical joint.
- b_{CL} is the wall panel length.
- w is the gravity load per unit length applied on the segmented wall under the seismic load combination.

Equation (1) can be rewritten as expressed in Equation (2):

$$\frac{F_{Rd,h,conn}}{K_{SLS,anc\ conn}} \geq 1{,}1 \frac{F_{Rd,c}}{K_{SLS,con}} \dashrightarrow \delta_{y,h} \geq 1{,}1 \delta_{y,c} \tag{2}$$

where $\delta_{y,h}$ and $\delta_{y,c}$ represent the yielding displacement of the hold-down/tie-down and the vertical joint, respectively. In order to ensure the hierarchy of yielding, the hold-down/tie-down should yield at a value of displacement greater than or equal to 110% of the yield displacement of the vertical joint.

A similar approach should be followed for the angle brackets resisting shear forces at the base of the walls. Equation (3) ensures that this connection yields after the vertical joints and hold-down/tie-down have yielded.

$$F_{Rd,s} = 1{,}1 \frac{M_{Rd,rock}}{M_{Ed,E}} F_{Ed,E,s} \tag{3}$$

where:

- $F_{Rd,s}$ is the design capacity of the base wall shear connection.
- $F_{Rd,E,s}$ is the design shear force under the seismic load combination.
- $M_{Rd,rock}$ is the design rocking moment resistance, including the stabilizing effect of vertical loads.
- $M_{Ed,E}$ is the design rocking moment acting on the wall, caused by the seismic action.

The design action on non-dissipative components is calculated by means of Equation (4):

$$F_{Rd,nd} \geq \frac{\gamma_{Rd}}{k_{deg}} \Omega_d F_{Ed,nd} + F_{Ed,G} \tag{4}$$

where γ_{Rd} is the overstrength factor, $F_{Rd,nd}$ is the design resistance of the non-dissipative component, k_{deg} is the strength reduction factor, $F_{Ed,nd}$ and $F_{Ed,G}$ are the design forces on the dissipative components due to the seismic and non-seismic action, respectively. Ω_d is defined as the structure overstrength ratio calculated as:

$$\Omega_d = \min(\Omega_{d,i}) = \left(\frac{\sum_{j=1}^{N_i} |M_{Rd,rock,i,j}|}{\sum_{j=1}^{N_i} |M_{Ed,i,j}|}\right) \tag{5}$$

where $\Omega_{d,i}$ is the overstrength ratio at the i^{th} storey in the considered direction, and $M_{Rd,rock,i,j}$ and $M_{Ed,i,j}$ are respectively the design rocking moment resistance and the design rocking moment acting on the j^{th} shear wall at the i^{th} storey.

To ensure that the designed capacity is respected at a global level, allowing plasticization to be distributed throughout all storeys of the building, the maximum storey

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overstrength ratio $\max(\Omega_{d,i})$, and the minimum storey overstrength ratio should satisfy the following equation:

$$\Omega_d = \frac{\max(\Omega_{d,i})}{\min(\Omega_{d,i})} \leq 1.25 \qquad (6)$$

3 – A SIMPLIFIED DESIGN PROCEDURES FOR CLT STRUCTURES MADE WITH SEGMENTED WALLS: THE DC2+

A new design method for multi-storey buildings with segmented walls is presented in this section. The main purpose is to define an intermediate ductility class between DC2 and DC3 [1], namely DC2+.

This method involves the use of segmented walls where the joints between different panels in the shear wall are considered dissipative connections, similar to what is presented for the DC3 design. However, in this case, a rigorous application of the capacity-based design between the vertical joints and the hold-down/tie-downs is not considered. A behaviour factor *q* equal to 2.75 is proposed for the intermediate ductility class DC2+.

The proposed method is essentially based on the main modifications compared to DC3 design listed in the following:

- Compliance with Equation (1) regarding the capacity-based design between wall joints and hold-down/tie-downs of a single segmented wall is not required
- Hold-down/tie-downs are designed for the tensile load obtained from the analysis multiplied by 1.1
- Angle brackets for shear forces are designed with the shear design action multiplied by 1.3
- A fixed value of the overstrength ratio Ω_d equal to 1.1 is used in Equation (4) for the design of non dissipative components
- The capacity design at the building level is not applied

Since the capacity based design is not rigorously applied like in DC3, for DC2+ the use of a lower value of the behaviour factor compared to the DC3 case is recommended.

Table 1 compares the design actions of various dissipative and non-dissipative connections for design in DC2 and DC3.

Table 1: Comparison of ductility classes

	DC2+	DC3
Behaviour factor q	2.75	3.2
Wall vertical joints	Design loads from numerical model.	Design loads from numerical model.
Hold-downs/ Tie-downs	Design loads from analysis increased by 10%.	Design loads from numerical model. Compliance with formulas (1)
Angle brackets for shear forces	Design loads from analysis increased by 30%.	Equation (3).
Non dissipative connections	Equation (4) with Ω_d = 1.1	Equations (4) and (5).
Capacity design at global level	n.a.	Equation (6)

The indicated formulas refer to [3].

4 – PRELIMINARY ANALYSIS FOR THE ASSESSMENT OF THE BEHAVIOUR FACTORS FOR DC2+

A preliminary validation of the behaviour factor proposed for the DC2+ is based on the numerical analysis of two structural archetypes [4], one with three stories and the other with five stories analysed with (W) and without (W/O) vertical loads. The seismic-resistant structure consists of CLT panels with five layers, with a total thickness of 120 mm.

The connections used are based on an experimental programme [5] in which WHT540 hold-downs with 12 cylindrical shank nails (4x60 mm) and BMF 90x116x48x3 angle brackets with 11 cylindrical shank nails (4x60 mm) were tested. The joints between adjacent panels are of the half-lap type, connected with 8x80 mm screws.

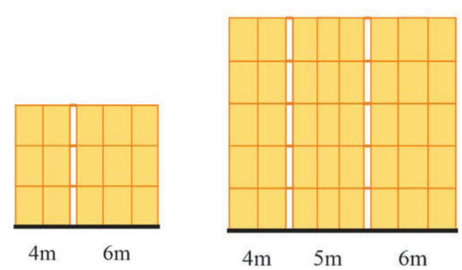

Figure 1. Structural archetypes analysed.

The archetypes are analysed using nonlinear static (push-over) analyses with force-controlled loading and an invariant load profile proportional to the first vibration mode.

The reference location is L'Aquila (Italy), where the Peak Ground Acceleration (PGA) is 0.26g (for a return period of 475 years), corresponding to a 10% probability of exceedance in 50 years for subsoil type A-T1.

Table 2: behaviour factors for DC2+ and DC3.

		DC2+			DC3 [4]		
	q_{design}	2,75			3,20		
		q_R	q_D	**q**	q_R	q_D	**q**
3-storey	W	1,4	1,33	**2,79**	1,5	1,54	**3,45**
	W/O	1,63	1,37	**3,33**	1,71	1,37	**3,52**
5-storey	W	1,43	1,21	**2,59**	1,44	1,53	**3,30**
	W/O	1,56	1,35	**3,17**	1,67	1,39	**3,47**

The difference between DC2+ and DC3 lies primarily in the plasticization mechanism of the connections at various limit states which is highlighted by a reduced global ductility. In DC2+, the sliding resistance system is more exploited than in DC3. Additionally, a higher number of connections remain in the elastic range.

The nonlinear static numerical analyses conducted on the two configurations revealed a behaviour factor ranging from 2.59 to 3.33, which aligns well with the value of 2.75 adopted for DC2+ in the design phase.

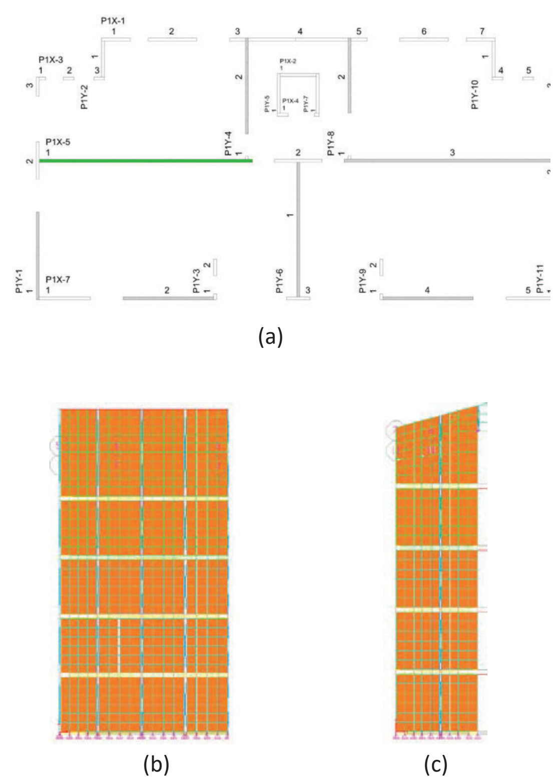

Figure 2. Plan view (a), lateral view of wall A along x direction (b), and lateral view of wall B along y direction (c).

5 – CASE STUDY

A comparison of the design procedure between DC2+ and DC3 ductility classes has been conducted as part of the design process for a five-storey residential case-study building using a response spectrum analysis [1]. Figure 2 shows the walls considered for comparing the results between different ductility classes. Wall P X-5-1, hereafter called Wall A, is a long wall located centrally within the building, consisting of five panels and subjected to significant gravitational loads. In contrast, wall P Y-11-1, hereafter called Wall B, is a short wall positioned externally in the building, consisting of two panels and subjected to low gravitational loads.

The results at different storeys for Wall A and B are reported for both DC2+ and DC3 in terms of design rocking moment (Figure 3 and 4), minimum number of screws required in both vertical joints and hold-downs (Table 3 and 4), design shear force (Figure 5 and 6), number of angle brackets (Table 6 and 7), design shear force for non-dissipative connections between slab and underlying wall (Figure 7 and 8), and number of angle brackets for non-dissipative connections (Table 9 and 10).

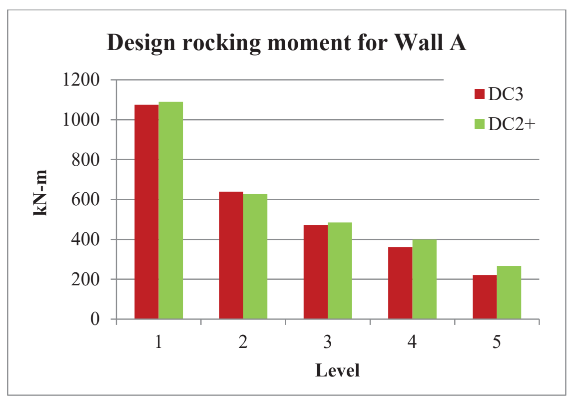

Figure 3. Design rocking moment for Wall A

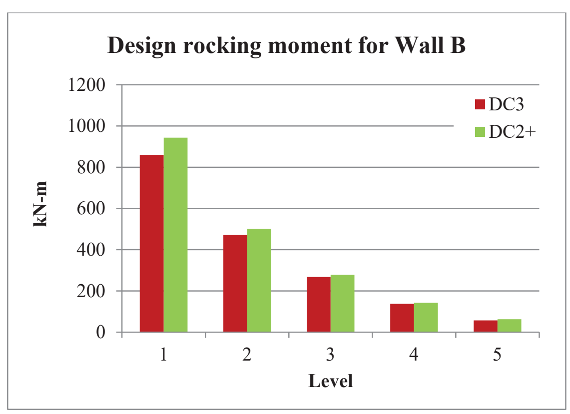

Figure 4. Design rocking moment for Wall B

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Table 3: Design loads derived from the numerical model and minimum number of screws required for Wall A.

Level	DC3		DC2+	
	Vert. joints (Screws 5x120) (*)	***Hold-down (Screws 10x100)***	***Vert. joints (Screws 8x80) (*)***	***Hold-down (Screws 10x100)***
1	$F_{Rd,c,tot}$ = 37.0 kN 30 screws	$F_{Rd,hd}$ = 44.9 kN 6 screws	$F_{Rd,c,tot}$ = 30.8 kN 10 screws**	$F_{Rd,hd}$ = 15.0 kN 2 screws**
2	$F_{Rd,c,tot}$ = 12.3 kN 10 screws	$F_{Rd,hd}$ = 29.9 kN 4 screws	$F_{Rd,c,tot}$ = 30.8 kN 10 screws**	$F_{Rd,hd}$ = 15.0 kN 2 screws**
3	$F_{Rd,c,tot}$ = 12.3 kN 10 screws**	$F_{Rd,hd}$ = 15.0 kN 2 screws**	$F_{Rd,c,tot}$ = 30.8 kN 10 screws**	$F_{Rd,hd}$ = 15.0 kN 2 screws**
4	$F_{Rd,c,tot}$ = 12.3 kN 10 screws**	$F_{Rd,hd}$ = 15.0 kN 2 screws**	$F_{Rd,c,tot}$ = 30.8 kN 10 screws**	$F_{Rd,hd}$ = 15.0 kN 2 screws**
5	$F_{Rd,c,tot}$ = 12.3 kN 10 screws**	$F_{Rd,hd}$ = 15.0 kN 2 screws**	$F_{Rd,c,tot}$ = 30.8 kN 10 screws**	$F_{Rd,hd}$ = 15.0 kN 2 screws**

(*) design capacity of the vertical joint; (**) minimum number of screws for installation purpose

Table 4: Design loads derived from the numerical model and minimum number of screws required for Wall B.

Level	DC3		DC2+	
	Vert. joints (Screws 5x120) (*)	***Hold-down (Screws 10x100)***	***Vert. joints (Screws 8x80) (*)***	***Hold-down (Screws 10x100)***
1	$F_{Rd,c,tot}$ = 86.2 kN 70 screws	$F_{Rd,hd}$ = 314.2 kN 42 screws	$F_{Rd,c,tot}$ = 187.4 kN 60 screws	$F_{Rd,hd}$ = 187.0 kN 25 screws
2	$F_{Rd,c,tot}$ = 61.6 kN 50 screws	$F_{Rd,hd}$ = 157.1 kN 21 screws	$F_{Rd,c,tot}$ = 30.8 kN 10 screws**	$F_{Rd,hd}$ = 142.1 kN 19 screws***
3	$F_{Rd,c,tot}$ = 24.6 kN 20 screws	$F_{Rd,hd}$ = 89.8 kN 12 screws	$F_{Rd,c,tot}$ = 123.18 kN 40 screws	$F_{Rd,hd}$ = 82.3 kN 11 screws***
4	$F_{Rd,c,tot}$ = 12.3 kN 10 screws**	$F_{Rd,hd}$ = 37.4 kN 5 screws	$F_{Rd,c,tot}$ = 30.8 kN 10 screws**	$F_{Rd,hd}$ = 22.4 kN 3 screws***
5	$F_{Rd,c,tot}$ = 12.3 kN 10 screws**	$F_{Rd,hd}$ = 15.0 kN 2 screws**	$F_{Rd,c,tot}$ = 30.8 kN 10 screws**	$F_{Rd,hd}$ = 15.0 kN 2 screws

(*) design capacity of the vertical joint; (**) minimum number of screws, (***) uplift prevails over rocking

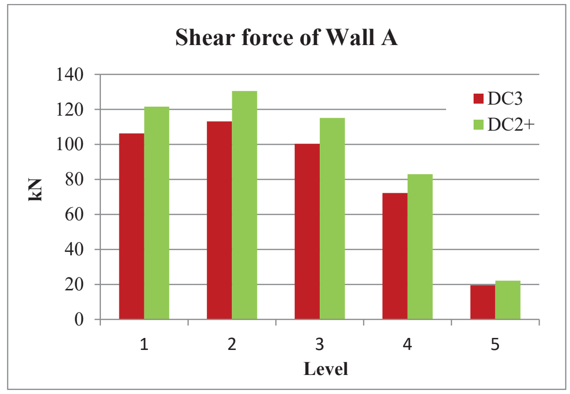

Figure 5. Design shear forces for dissipative connections for Wall A

Table 5: Design capacity of angle brackets used in dissipative connections

Angle bracket	Product	Design capacity
TYPE A*	TCN240 with 36-Ø5x70mm partially threaded screws + 2- M16 x 138mm anchor	33.0 kN
TYPE B	TCN240 with 2 x 36-Ø5x70mm partially threaded screws	33.0 kN

(*) Wall-to-foundation connection

Table 6: number of angle brackets for Wall A in dissipative connections.

Level	DC3	DC2+
1	4 TYPE A	4 TYPE A
2	4 TYPE B	5 TYPE B
3	4 TYPE B	4 TYPE B
4	3 TYPE B	3 TYPE B
5	2 TYPE B *	2 TYPE B

(*) Reduced number of fasteners

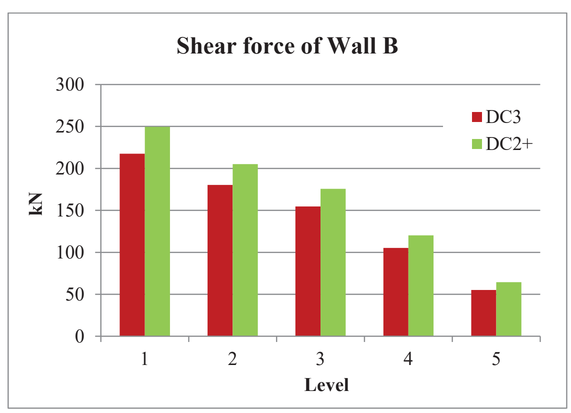

Figure 6. Design shear forces for dissipative connections for Wall B

Table 7: number of angle brackets for Wall B in dissipative connection.

Level	DC3	DC2+
1	7 TYPE A	8 TYPE A
2	6 TYPE B	7 TYPE B
3	5 TYPE B	6 TYPE B
4	4 TYPE B	4 TYPE B
5	5 TYPE B *	6 TYPE B *

(*) Reduced number of fasteners

Table 8: Design capacity of angle brackets used in non-dissipative connections (slab-to-underlying wall connection)

Angle bracket	Product	Design capacity
TYPE C	TTV240 w/ 36 + 30-Ø5x70mm partially threaded screws + 2- Ø11 x 200mm fully threaded screws	65.7 kN
TYPE D	TTF200 w/ 2 x 30-Ø5x70mm partially threaded screws	46.8 kN

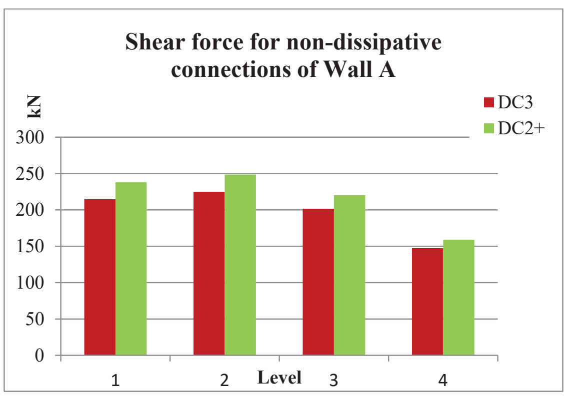

Figure 7. Design shear for non-dissipative connections between slab and underlying wall for Wall A

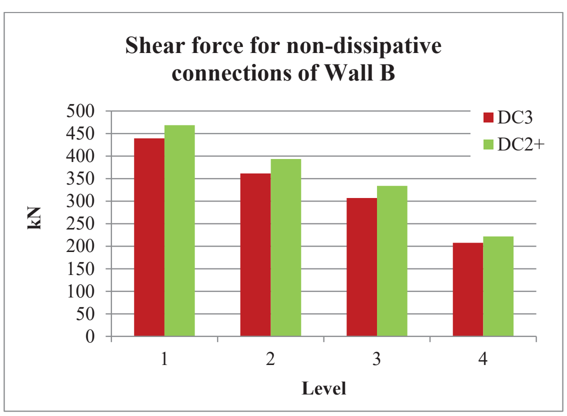

Figure 8. Design shear for non-dissipative connections between slab and underlying wall for Wall B

Table 9: number of angle brackets for non-dissipative conn. on Wall A.

Level	DC3	DC2+
1	4 TYPE C	9 TYPE C
2	5 TYPE D	9 TYPE D
3	5 TYPE D	9 TYPE D
4	4 TYPE D	9 TYPE D

Table 10: number of angle brackets for non-dissipative conn. on Wall B.

Level	DC3	DC2+
1	7 TYPE C	9 TYPE C
2	9 TYPE D	9 TYPE D
3	8 TYPE D	8 TYPE D
4	5 TYPE D	6 TYPE D

The main results from the analyses are reported hereinafter:

- In DC2+, the rigorous capacity-based design between the hold-down/tie-down and vertical joints is not applied. As a result, it is possible to encounter situations where hold-down/tie-down connections yield before the vertical joints.
- In DC2+, there is a loss of control over the actual overstrength of non-dissipative connections. In DC3 design, the coefficient Ω_d is calculated based on the actual resisting rocking moment, taking into account any over-dimensioning.
- The use of a fixed value of the coefficient Ω_d leads to a reduction in the actual overstrength of the non-dissipative connections whenever the dissipative connections against rocking are over-dimensioned.

In practice, this condition can frequently occur since it is difficult to define a different number of connectors for each load condition.

A brief analysis revealed that small homogenizations of the dissipative connections lead to significant reductions in actual overstrength, which, in extreme cases, can completely vanish.

As can be observed from the results and as expected, the lower value of the q-factor in DC2+ leads to higher seismic demand in dissipative and non-dissipative connections with respect to the DC3 case, with a maximum increase of 18% for shear dissipative connection and an average increase of 10% lower than the decrease of the q-factor (2.75 vs 3.2 with a 14% reduction) and with the increase lowering to almost zero as the number of storey increases.

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Therefore, this analysis highlights that the DC2+ design, even if with a lower energy dissipation of the entire structure, proved to be applicable for medium-rise CLT buildings even with commercial connectors.

6 –CONCLUSIONS

This study clarifies the main steps to be followed in DC3 design of CLT multi-storey buildings. The expressions included in the new Eurocode 8, developed to ensure a rigorous application of capacity-based design, are presented and applied to assess a case study.

The condition expressed by Equation (1) require specific attention in the selection of the vertical joint connections, which should be characterized by yielding values lower than those of the hold-downs. Half-lap joints or butt joints seem to be more appropriate than spline joints, as they exhibit higher stiffness values.

As a simplification of the rigorous DC3 approach, the DC2+ method can be applied, with a considerable simplification of the computational burden balanced on the other hand by the use of a slightly lower behaviour factor (q) value with respect to the one proposed in [3] for DC3.

7 – ACKNOWLEDGEMENTS

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