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Numerical Analysis of Cross-Laminated Timber (CLT) Buildings: A Parametric Study on Steel Connectors When Subjected to Seismic Loading Under Eurocode 8 and NBR 15421

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Abstract: Cross-laminated timber (CLT) presents significant potential for sustainable construction but requires further investigation under seismic conditions. This study develops a numerical model to evaluate the seismic design requirements of CLT buildings according to European (Eurocode 8) and Brazilian (NBR 15421) standards. Experimental data from a full-scale CLT building were used to validate the model. The model was then applied to assess seismic design according to standard requirements across different geographic locations, and a parametric investigation was conducted to evaluate the impact of the connector design on structural performance. The results indicate that the tested CLT building was overdesigned for all evaluated regions, and a significant reduction in displacements—up to 33%—is achieved by adjusting the quantity of the connectors. Additionally, the analysis shows limitations in NBR 15421, as it resulted in higher average lateral displacements due to insufficient consideration of energy dissipation. These findings underscore the importance of optimising connector configurations to enhance the seismic performance of CLT buildings while reducing overdesign. Additionally, properly considering energy dissipation in design standards is crucial. In particular, the Brazilian standard would benefit from a comprehensive review to better address energy dissipation, ensuring safer and more efficient seismic designs.

Keywords: cross-laminated timber; seismic design; Eurocode 8; NBR 15421; parametric evaluation; numerical analysis

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1. Introduction

Construction is not only a short-term project within a specific time frame but also a crucial economic sector. Focusing solely on the on-site labour, one overlooks the broader production chain. Engineering, both for practitioners and researchers, is increasingly focusing on the entire process, including the impact of materials and techniques [1]. As a result, new materials and construction methods are emerging, intending to increase sustainability and productivity.

While housing remains a fundamental need, the construction sector significantly contributes to greenhouse gas emissions, solid waste production, and the depletion of natural resources [2]. In Brazil, for example, traditional concrete and masonry constructions

are known for high production costs, poor planning, low levels of worker qualification, and considerable material waste [3].

Cross-laminated timber (CLT) is a relatively new material gaining increasing recognition as sustainability becomes a priority. It is manufactured by laminating logs into panels with alternating grain directions, usually consisting of three to nine layers. This process strengthens the CLT panels, allowing them to withstand stresses in both in-plane and out-of-plane stresses, as fibres run along both the length and width. Figure 1 provides a visual representation of the panel construction process. CLT panels are generally manufactured from spruce sideboards (*Picea abies*), with an approximate density of 400 kg/m^3 [4]. Once the lamellae are produced (see Figure 1a), they are arranged to form panels, with the fibres oriented in alternating directions (see Figure 1b) and glued together (see Figure 1c). The adhesives used in this process typically exhibit quasi-rigid behaviour and include three main types: phenoplast- and aminoplast-based adhesives, 1-component polyurethane adhesives (1K-PUR), and emulsion-polymer-isocyanate adhesives (EPI) [5].

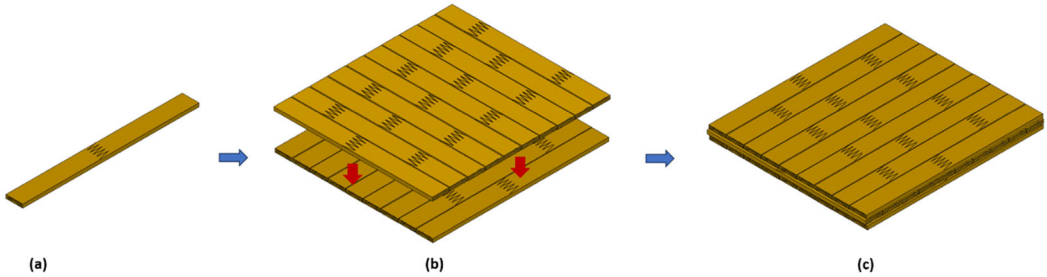


Figure 1. CLT panel conception: (**a**) finger-jointing; (**b**) lamellae crossing; and (**c**) panel glueing.

In addition to CLT panels, steel connectors are essential to ensure the stiffness of the structure, especially under static and dynamic loads. Several studies indicate that structural failure under lateral loads primarily occurs in the connections [4,6,7]. Typically, two types of connectors are employed: angle brackets, to provide shear resistance, and hold-downs, to ensure uplift resistance—see Figure 2. As stated by Sikora et al. [5], the number of connectors and spacing must be specified so that their combined strength meets the required rotational and translational design load demands.



(**a**) (**b**)

Figure 2. Steel connectors used in the building: (**a**) hold down and (**b**) angle bracket.

Figure 3 presents a complete CLT structural system and its necessary components. The interaction between different components is crucial for the structure's overall performance

under loading. Connectors are the main responsible to guarantee continuity in the load distribution and ensure the resistance to uplift, sliding, and rocking movements. These elements are screwed into the foundations, which can be made of reinforced concrete or steel.

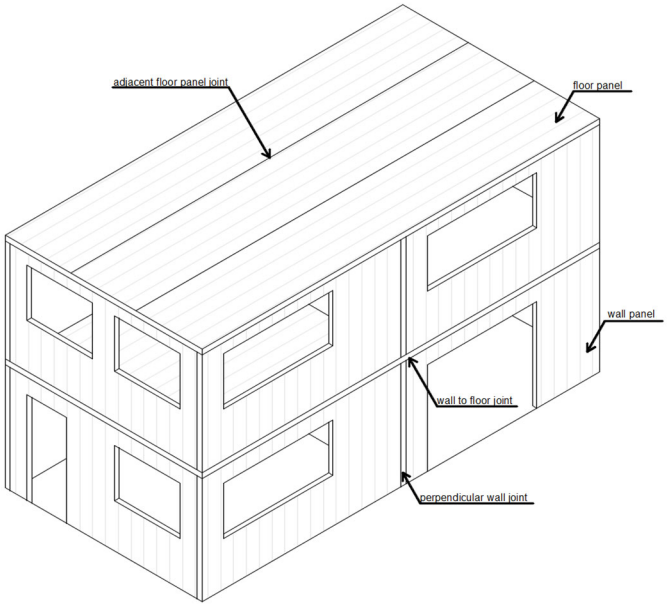


Figure 3. Complete CLT structural system: detailing of its components (nomenclature as in ref. [8]).

In recent years, research on the structural behaviour of CLT buildings and the adoption of this technology has grown significantly [9]. Many studies have focused on evaluating its performance under various conditions. Regarding lateral loads, such as seismic and wind, experimental [10,11] and numerical [12,13] studies have been conducted to assess design variations and the overall behaviour of the structural panels. Experimental campaigns typically involve monotonic and cyclic concentrated loads to simulate the lateral forces acting on the building. Considering the structural response of the buildings is governed by the steel connectors [7], the expected outcome of these experimental and numerical studies is the evaluation of energy dissipation and the hysteresis associated with cyclic loadings [14,15], with special attention to the behaviour under lateral loads [16].

With the increasing use of CLT in construction, design codes are being updated to address its specific requirements [16]. Notable examples include the proposed revision of Eurocode 8 (prEN 1998:2021) [8,17] and the Brazilian standards ABNT NBR 7190 [18] and NBR 15421 [19].

Concerning the Eurocode 8, the European Committee for Standardization (CEN) has initiated the revision process, and, for the first time, it will include provisions for CLT design, aiming to standardise its application in the building sector. The revision is considered crucial to address the issues with current regulations, which are seen as outdated and overly concise [16]. The second generation of Eurocode 8 was drafted in 2021 and addresses critical aspects such as soil characterisation, amplification factors, and ductility assessments of structures. This aligns with CEN's goals to incorporate new materials, such as CLT, into construction practices, as stated in its Mandate M/515 that paved the way for the second generation of Eurocodes [20–22].

In the Brazilian scenario, the recent code revisions have established CLT as a safe and viable material for use in construction. Unlike Eurocode 8, where seismic design is dependent on the material adopted, the Brazilian normative framework separates the definition of the seismic resultant loads according to NBR 15421 [19] and the timber

structure design according to NBR 7190 [18]. CLT was recently included in the revision of the latter, facilitating its introduction in the Brazilian construction industry.

In this context, the present work addresses the new regulation on seismic design, both European and Brazilian, and studies the structural response of a CLT case-study building when subjected to the loads proposed by each code. The building was experimentally tested at the University of Minho, and the results are available in De Matos et al. [6]. In this research, a numerical model for the building is used and validated with the available experimental data. In addition, a parametric analysis is conducted to assess the building's response when the number of steel connectors is reduced, comparing the designing hypothesis of NBR 15421 [19] and prEN 1998:2021 [8,17].

2. Theoretical Framework

This section aims to provide the theoretical background of the techniques used in this study. It will cover the entire modelling methodology, offering a detailed explanation of each technique employed. Furthermore, an overview of the relevant European and Brazilian codes will be provided, discussing their key principles and how they influence the approach taken in this research.

2.1. Response Spectrum Curves

The response spectrum method is a widely employed tool in seismic analysis, offering an efficient means to evaluate the dynamic response of structures subjected to earthquake-induced ground motion. This method simplifies complex seismic events into an envelope curve, representing the maximum structural response (e.g., acceleration or displacement) across a range of natural vibration periods [23]. By considering both site characterisation and seismic hazard, the response spectrum provides critical insights into the seismic demand imposed on structures.

This study utilises response spectrum curves generated in accordance with the requirements of Eurocode 8 [8,17] and NBR 15421 [19], ensuring that the influence of local soil conditions and regional seismic hazard levels is captured. Site effects have been normalised in the results to allow for consistent comparisons between the codes.

The analysis was conducted by selecting a site in the Brazilian state of Acre, located in the western Amazon region. While Brazil generally experiences low seismicity, the chosen location represents a scenario with elevated seismic hazard in the national context, aligning with a medium seismic activity classification under European standards. It is important to note that this site was arbitrarily selected for the purpose of this study, serving as a representative case. Additionally, comparable virtual regions were analysed using the National Determined Parameters of Eurocode 8, focusing on Portugal—where the experimental campaign of the case-study building used to validate the numerical model took place—and Italy, a country known for its moderate seismic hazard.

Both European and Brazilian methodologies were used to create the Response Spectrum Curves presented in Figure 4. The soil characterisation compatibility was carried out using N-SPT values, as this is the methodology used in both codes to determine site amplification factors. The value was arbitrarily chosen to be 25 blows, which resulted in similar soil classification.

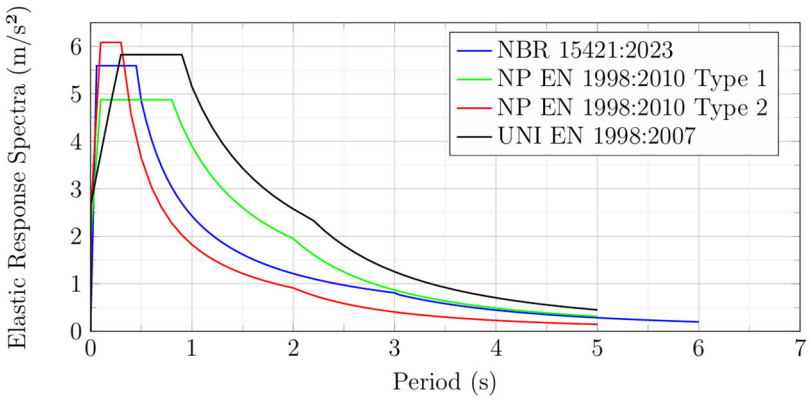


Figure 4. Elastic response spectra for the proposed region.

2.2. Ductility

Among the codes studied herein, only the European standard provides a framework for evaluating ductility. The Brazilian code does not include any factors to modify the seismic load after the creation of the Response Spectrum Curve (RSC). In contrast, prEN 1998:2021 [17] introduces the use of a behaviour factor, which reduces the seismic impact on the structure. This reduction is directly influenced by the level of detailing in the structural model, with higher ductile classes requiring greater attention to simplification assumptions.

The behaviour factor is determined based on the ductile class, reflecting the ductility assumed during modelling. The code defines three ductile classes, each representing distinct levels of seismic performance: Ductile Class 1 (DC1), which considers only the overstrength capacity of structural elements; Ductile Class 2 (DC2), which incorporates deformation and energy dissipation; and Ductile Class 3 (DC3), which also includes a global plastic mechanism in addition to the considerations of DC2. In this study, DC1 and DC2 are considered separately and will be further explained.

3. Numerical Modelling Approach

The computational model developed aims to replicate the structural behaviour of cross-laminated timber (CLT) buildings subjected to seismic loading. The numerical model was implemented in the Dlubal RFEM software (v5.31), incorporating advanced modelling techniques to simulate the interaction between structural components, connectors, and boundary conditions.

The numerical modelling approach builds upon the model proposed by De Matos et al. [6] and consists of modelling the CLT panels with 2D shell elements, where mixed interpolation of tensorial components was used with quadrilaterals (4 nodes) and triangular (3 nodes) elements—MITC4 and MITC3, respectively. In addition, 1D rigid beam elements were used to model the connections between the floor-to-wall CLT panels. The stiffness of these elements was defined by the penalty method. The modelling strategy is chosen specifically for representing CLT structures. Floors were modelled separately from wall panels to better depict the behaviour of steel connectors under seismic loading. The separation between the floors and walls introduces a gap of 0.1 m, ensuring a realistic simulation of connection dynamics, as illustrated in Figure 5.

The steel connectors were modelled using nonlinear nodal supports to define their mechanical properties. These elements were used to replicate the practical placement of hold-downs and angle brackets. Their behaviour was defined using load-displacement envelope curves derived from experimental cyclic loading protocols. Specifically, the 1st Load Envelope Curve (1st LEC) was utilised to simulate the connectors' response, capturing elastic, yielding, and post-yield behaviours. This approach ensures that the

structural interaction and force transfer between elements are realistically represented. The mechanical behaviour based on the 1st LEC of each connector is presented in Figure 6. Regarding maximum values under tension loads (Figure 6a), AE116 achieved 12.9 kN and 5.5 mm of ultimate load and displacement, respectively, when fixed into CLT support. In the other hand, under the same conditions, HTT22 achieved 53.8 kN and 18.9 mm. Under lateral loads, AE116 achieved 21 kN and 18.1 mm of ultimate load and displacement, respectively.

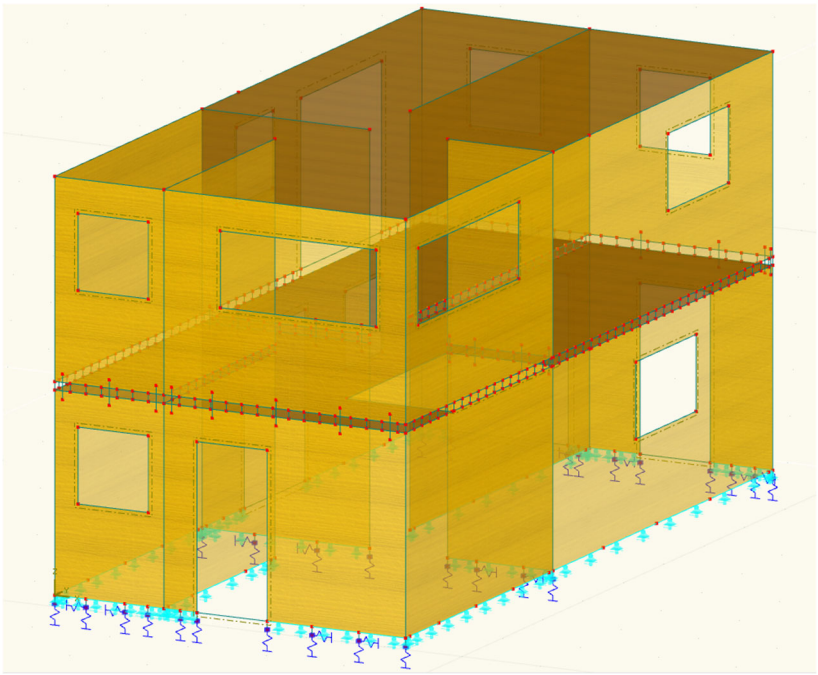


Figure 5. Computational model developed in the present study.

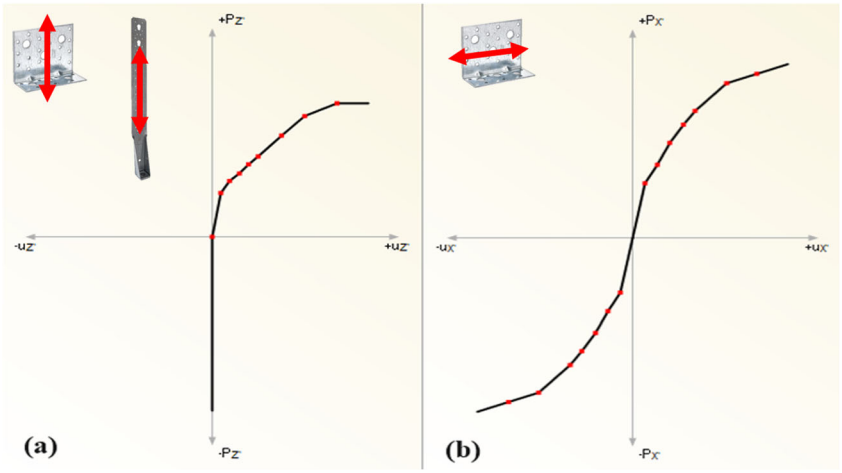


Figure 6. First LEC force (P) versus displacement (u). (**a**) AE116 and HTT22 under tension and (**b**) AE116 under shear.

Connections between wall panels were considered rigid in the numerical model, as specified in De Matos et al. [6] during DC1 analysis. To use DC2 in the analysis, an improvement of the proposed model is required to meet the prEN 1998:2021 [8] requirements. As a result, a minor modification was made to the structural model to account for energy dissipation in the connections. Given that the model proposed by De Matos et al. [6] was already characterised by a high degree of detail, the only modification required concerned the connections between orthogonal walls. The literature reveals a variety of connections

of this nature, such as nailed [24], screwed [25,26], dowelled [27], or even glued [28]. In this study, screwed connections between CLT wall and floor panels were adopted, following the experimental campaign of De Matos et al. [6]. This type of connection shows higher capacity when compared to other connection types, with good efficiency when submitted to lateral or axial loads [29]—despite the worse behaviour in energy dissipation [30].

The numerical modelling of these connections was performed using the line hinge tool in Dlubal RFEM (v5.31). This tool allows using a spring constant in surface boundary lines to represent the mechanical behaviour of the screwed connections. The spring constant was defined as the secant stiffness at peak based on the experimental results available [25], resulting in 40,057 kN/m. Since there is no energy dissipation in these elements, only the secant stiffness was used to represent the connector behaviour. This approach was chosen because it provides a conservative estimate of the connection stiffness. The load-displacement curve is presented in Figure 7, and the detailing of the connection is presented in Figure 8. This connection is constituted of 8 fully threaded (FT) self-tapping screws (STS) with 8 mm of diameter, made of hardened steel with strength up to 1000 MPa [25], and it is referred to as the 16X-400 STS connection.

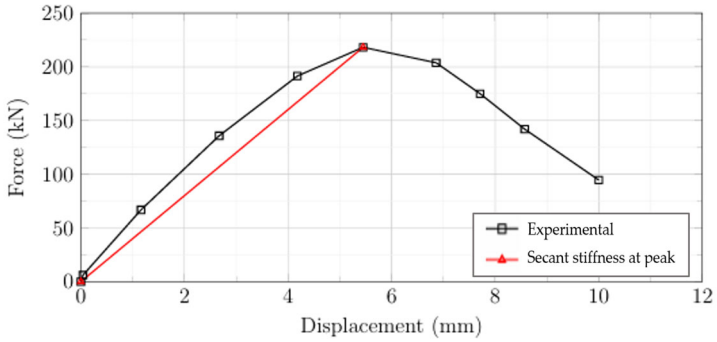


Figure 7. Load–slip curve under shear for 16X-400 configuration.

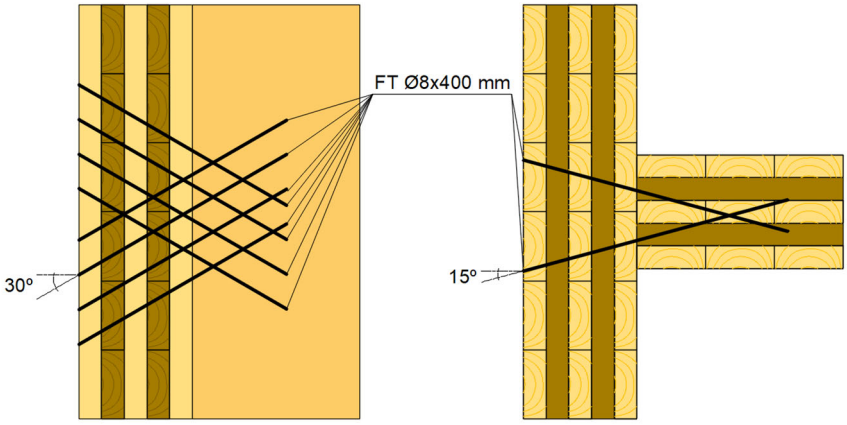


Figure 8. Detailing of 16X-400 STS. Elevation (**a**) and plan view (**b**)—inspired from from ref. [25].

Mathematical Modelling

To solve the equations governing the structural system, two computational methods were employed. For linear equations, the Cholesky factorisation was utilised, a direct method well-suited for symmetric positive-definite matrices, which are typical in structural mechanics. This technique decomposes the global stiffness matrix into the product of a lower triangular matrix and its conjugate transpose, ensuring computational stability and

efficiency. Once factorisation is complete, the equations are resolved through forward and backward substitution [31].

Nonlinearities in the model were handled using the Newton–Raphson method, an iterative approach that linearises the system at each step based on the tangent stiffness matrix. This method progressively refines the solution by iterating until convergence is achieved, allowing us to accurately capturing the nonlinear behaviour. The specific nonlinearities considered in this study are discussed in detail in the following section.

Modal analysis was conducted to determine the dynamic response of the structure, employing the square root of the sum of squares (SRSS) method to combine modal contributions.

The SRSS approach is expressed as follows:

$$R_{SRSS} = \sqrt{R_1^2 + R_2^2 + \cdots + R_p^2}, \quad (1)$$

where R_p represents the response contribution of the p-th vibration mode [32]. This method enables the aggregation of modal effects while respecting their individual contributions to the overall dynamic behaviour.

To ensure an accurate representation of the structural response, the analysis considered enough modes to satisfy the effective modal mass criterion, defined as follows:

$$\sum_{i=1}^{n} m_{eff,i} \geq 0.9\ M_{total}\ , \quad (2)$$

where M_{total} is the total mass of the structure, $m_{eff,i}$ is the effective mass associated with mode i, and n is the number of modes included. This criterion guarantees that at least 90% of the total structural mass is represented in the analysis, consistent with both Eurocode 8 and NBR 15421 requirements.

The elastic response spectrum results were adjusted using a behaviour factor (q) for energy dissipation, as recommended by Eurocode 8. For low ductility class (DC1) structures, a reduction factor of 1.5 was applied, reflecting the reduced seismic demands on the elastic spectrum.

Numerical modelling of the cross-laminated timber (CLT) panels followed the classical laminate theory, treating each layer of the laminate as thin and orthotropic [33]. This approach accounts for the anisotropic mechanical properties in each principal direction, enabling the accurate simulation of the material's behaviour. Mechanical properties specific to the layers are detailed in Section 3. Additionally, the influence of shear deformation, which is particularly significant in bending, was incorporated using the Reissner-Mindlin plate theory, allowing for a more comprehensive representation of the panel's behaviour, as it accounts for shear deformation along the thickness of the plate. In this theory, the orthogonality of the straight fibres to the middle surface is not imposed after the deformation [32]. The response of the plates was assumed to remain linear throughout the study.

The stiffness matrix of the structure was obtained with the aid of RF-Laminate Dlubal's add-on to the CLT panels and using the elements previously mentioned to the other elements. The module applies the theory previously explained to create the local stiffness matrix, and then all matrices are transformed into one global stiffness matrix.

4. Model Validation

In this section, the validation of the numerical modelling approach is presented. Experimental results from a full-scale CLT building test in the literature were used for comparison.

4.1. Overview of the Experimental Results

In De Matos et al. [6], a series of experiments were conducted to develop a numerical model of a two-storey CLT building undergoing a pushover analysis. The study began with an experimental campaign focused on standard steel connectors that link CLT panels to one another and to the foundation. Two types of connectors were tested: HTT22 hold-downs produced out of Z275 pre-galvanised mild steel [33] and AE116 angle brackets manufactured with S250GD [34], both from Simpson Strong-Tie (Clever Reinforcement Ibérica Lda, Amora, Portugal) (see Figure 2). These connectors were tested according to EN 12512:2001 [35] to obtain the load-displacement curves for future numerical modelling. Subsequently, a full-scale CLT building, presented in Figure 9, was subjected to quasi-static loading to evaluate its three-dimensional behaviour under lateral loads. While specific details of the connector testing are not given here for conciseness, the reader is referred to De Matos et al. [6] for additional information.

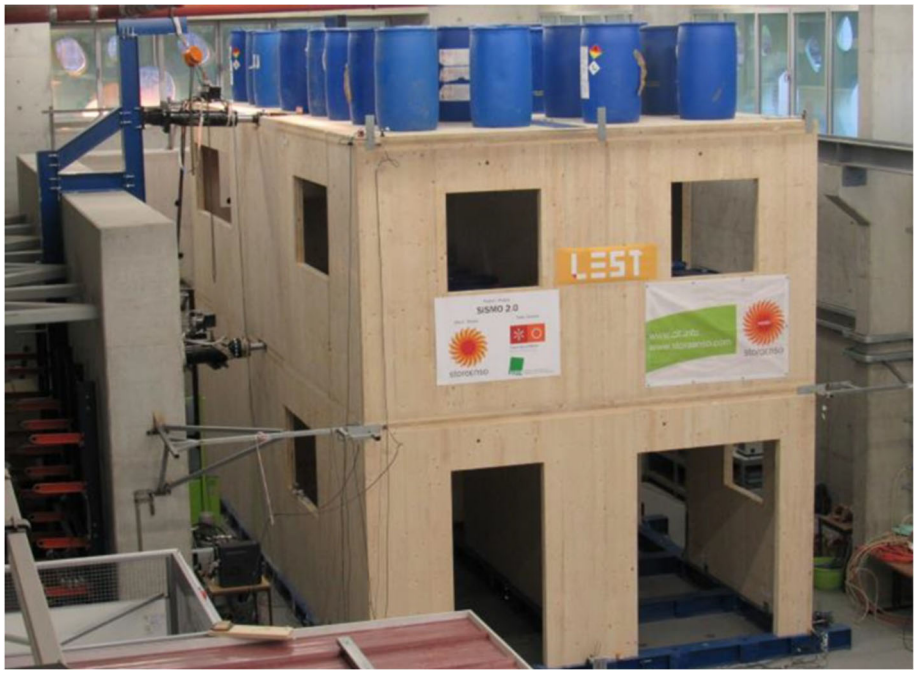


Figure 9. Experimental CLT building built and tested at the University of Minho.

The dimensions of the tested building were 4.5 m in width, 9.1 m in length, and a total height of 5.04 m, as shown in Figure 10. The building was designed to represent a typical structure, featuring wall and floor openings typical of real buildings. The design included an asymmetric layout to induce torsional effects under seismic loads. The CLT wall panels were made of spruce and consisted of five layers, each 20 mm thick, resulting in a total thickness of 100 mm. Meanwhile, the floor panels were composed of three layers, each 40 mm thick, resulting in a total thickness of 120 mm. CLT elements were produced by Stora Enso (Stora Enso Oyj, Helsinki, Finland) in accordance with ETA 14/0349 [36], and the elastic properties of the C24 boards used in the panel assembly are presented in Table 1.

Table 1. C24 boards elastic properties (MPa).

Modulus of Elasticity		
Parallel to grain direction	$E_{0,mean}$	12,500
Shear Modulus		
Parallel to grain direction	G_{mean}	690
Perpendicular to grain direction	$G_{r,mean}$	50

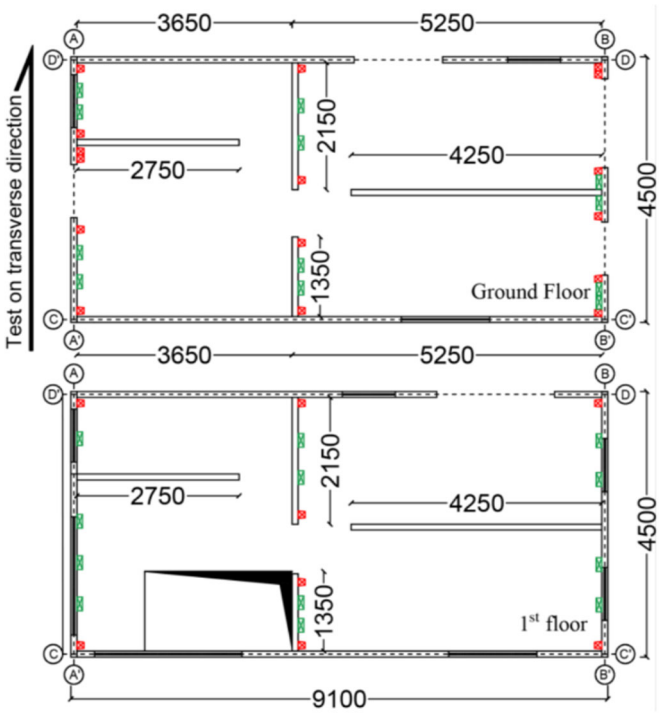


Figure 10. CLT building plans and connectors location on the ground and first floors for the transverse loading direction. Dimensions are in millimetres.

The building was subjected to monotonic loading via two hydraulic jacks, one positioned on each floor. The jacks applied displacement-controlled loading, incrementally increasing until the structure experienced significant damage or collapse.

To prevent overlapping effects, the connectors were exclusively applied to the shear walls in each loading direction [6]. As an example, Figure 10 also shows the connector distribution used during the transversal loading direction.

To determine the natural vibration modes of the building, it was essential to account for the additional mass introduced during the experimental campaign by De Matos et al. [6]. Water barrels were employed to simulate live and permanent loads exceeding the structure's self-weight, with additional loads of 2 kN/m^2 and 1.7 kN/m^2 applied to the first and second floors, respectively. Those loads were defined in accordance with EN 1998:2002 [37]. The total mass, combining the structure's self-weight and the added loads, was defined as a mass case, and the mass matrix was calculated consistently [38], fully representing the translational and rotational behaviours of the structural elements—this mass case was used in both European and Brazilian scenarios. The eigenvalue problem was solved using the root of the characteristic polynomial method, employing a direct solution approach. In selecting modal shapes, both standards utilised in this study require a minimum effective modal mass factor of 90%, which was satisfied during the analysis.

4.2. Numerical Validation

To validate the numerical model, a comparative analysis was conducted against the experimental campaign performed by De Matos et al. [6]. This campaign tested the seismic response of a CLT building in both the longitudinal and transverse directions. The transverse direction proved to be more critical for seismic analysis, as it induced greater structural damage.

The numerical model was calibrated using these experimental results, ensuring alignment between the observed and simulated structural responses. During the calibration process, static concentrated loads were applied to the model to represent the hydraulic jacks.

These loads were incrementally increased, and the resulting displacements were recorded at each step to construct displacement curves for comparison with the experimental data. The longitudinal direction, while also tested, exhibited significantly higher stiffness and was therefore less relevant for the current study's focus on seismic vulnerability.

Table 2 presents a detailed comparison of the results obtained from the numerical model and those observed during the experimental campaign. Key nodes in the computational model were selected for this analysis, ensuring consistency with the measurement points used in the experiments by De Matos et al. [6], as shown in Figure 11.

Table 2. Comparison between experimental and numerical displacements in the transverse direction.

Floor	Node	Experimental (mm)	Numerical (mm)	Absolute Difference (mm)	Relative Difference (%)
1st	1	57.6	49.5	8.1	14
	2	37.1	42.2	5.1	14
	3	57.4	47.1	10.3	18
2nd	1	81.6	72.8	8.8	11
	2	67.0	65.9	1.1	2
	3	84.4	72.9	11.5	14
	4	63.2	65.9	2.7	4

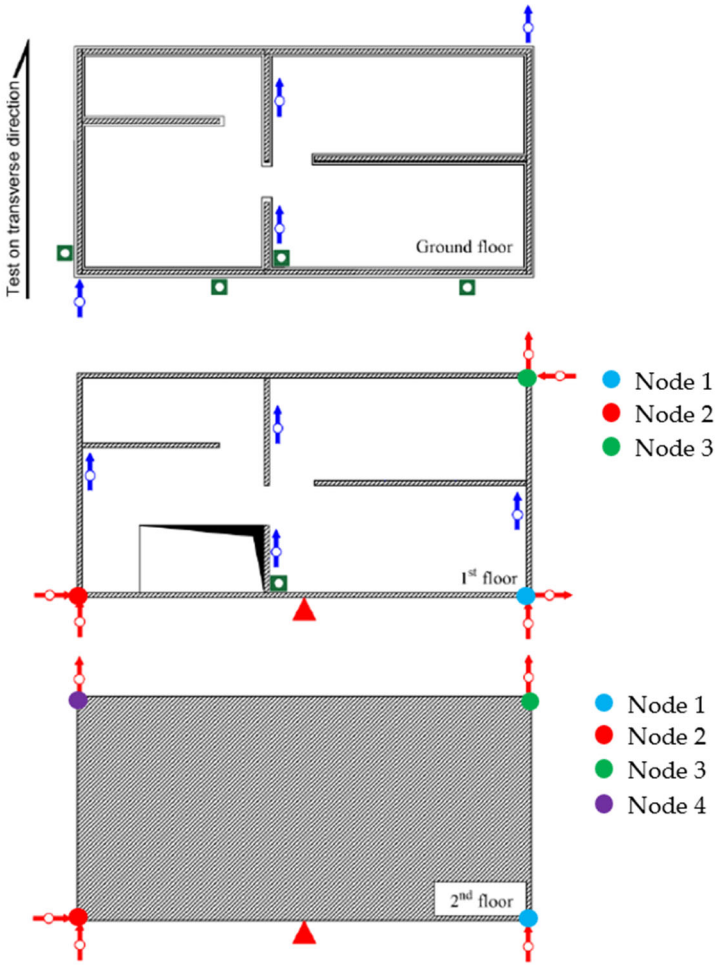


Figure 11. Node measurement location.

It can be observed that the structural response of the numerical model closely resembles the results from the experimental tests. This similarity is regarded as a reliable approximation, especially when accounting for differences in loading and the simplifying assumptions made. The average relative difference, calculated as the arithmetic mean of the nodal relative differences, was found to be 11%, excluding the effects of friction.

Regarding storey drift, the results are illustrated in Figure 12, which indicates a strong correlation between the numerical model's behaviour and the experiment. The analysis focuses on the centre of the building, specifically where the hydraulic jack load is applied. The numerical model shows greater floor slip compared to the experimental findings. This discrepancy may be attributed to the method of storey separation used and the assumption of zero friction between the CLT panels.

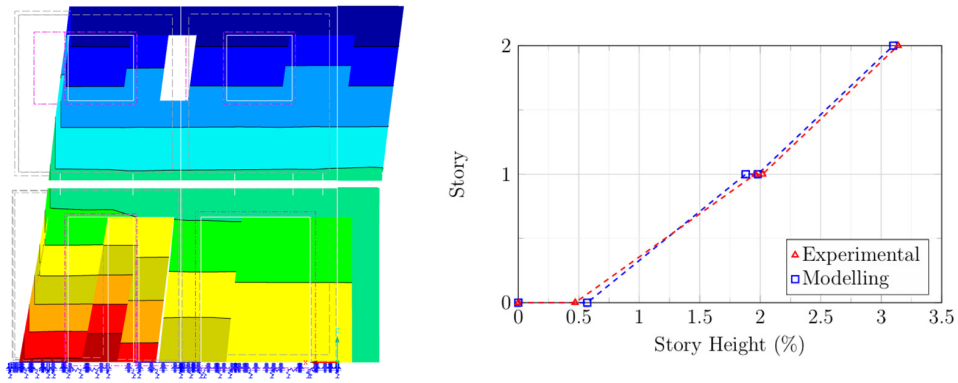


Figure 12. Experimental and numerical model comparison of storey drift in terms of its height (**right**) and deformed versus original configuration after loading (**left**).

Figure 13 shows both experimental and numerical curves of base shear versus displacement. No difference was noticed when comparing displacements of instantaneous application and stepped load during the simulations. One can note an initial discrepancy in the stiffness of the experimental and the numerical results. However, it is difficult to identify a single source for this difference due to the inherent uncertainties of the experimental campaign and the assumptions made in the numerical model. For instance, the difference could be traced to the behaviour of steel connectors assumed as the mean first load envelope curves obtained from experiments. In addition, the model is compared with only one building experimental curve, which is subjected to material variability not considered in the model. Therefore, the difference observed is expected, and it is possible to consider the model calibrated and suitable to be used in parametric studies on the seismic response of the CLT building.

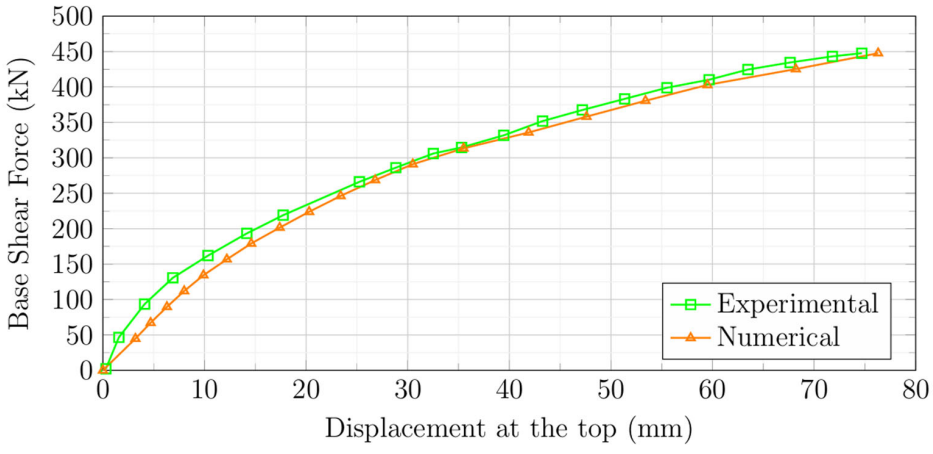


Figure 13. Experimental and numerical model comparison of base shear forces versus displacement at the top.

5. CLT Numerical Model Application: Assessment Under prEN 1998:2021 and NBR 15421:2023

This section presents the practical implementation of the developed numerical model, focusing on its adaptability and efficiency under varying configurations. A parametric study was conducted to investigate the impact of reducing the number of steel connectors—specifically AE116 angle brackets—on the structure's seismic performance and ductility. Both standards, prEN 1998:2021 [8,17] and NBR 15421:2023 [19], have been applied, and therefore, their efficiencies were evaluated. By varying this parameter, the building's performance under different scenarios was assessed.

5.1. Parametric Study on the Number of Angle Brackets

A parametric analysis focused on reducing the number of steel connectors within the structure is proposed. This variation was chosen due to its substantial influence on ductility and structural behaviour. A reduction in the total amount of AE116 angle brackets in the CLT building structure was proposed to reduce its lateral resistance and to achieve plastic deformations. In total, ten models were used for those four loading types. Each used model is presented in Table 3, where the reductions adopted were 25%, 50%, and 75% of the angle brackets number used on each floor.

Table 3. Modified models in the number of AE116 used. Percentage of connectors kept in comparison to the model experimentally tested in De Matos et al. [6].

Model	1st Floor (%)	2nd Floor (%)
0	100	100
1	100	75
2	100	50
3	100	25
4	75	75
5	75	50
6	75	25
7	50	50
8	50	25
9	25	25

Concerning the connectors' location, the reduction technique consisted of increasing spacing while maintaining equal distances between them. A normative limitation was applied to address both Brazilian and European timber construction codes, which requires at least one angle bracket in each panel.

5.2. Analysis Under Ductile Class 1 (DC1)

The obtained results of the structure submitted to the RSCs in DC1 are presented in Table 4 according to the different design codes verified. It is important to highlight that the presented values do not represent the storey drift, since they are the average absolute displacement at the top of the first and second floors. Relative displacement is the percentage of the displacement compared to the height of the storey.

The reference building showed an excellent performance across all analysed codes, as all relative displacements remained below the regulatory drift limits—2.0% in the Brazilian code and 1.5% for the European code, under the Significant Damage Limit State [8,17,19]. This outcome highlights the overdesign of the original building under multiple regulatory frameworks.

Despite Brazil being classified as a region with low seismic activity, the analysis revealed a notable difference in seismic demands, with the Italian code (UNI) [40] yielding

results 160% higher than those obtained using NBR 15421 [19]. In contrast, the differences observed with the Portuguese National Annex were much smaller, resulting in only 1.5 mm. The Brazilian framework appears to be more conservative than necessary for the country's actual seismic risk. Based on the above results, the use of the behaviour factor, especially in DC1, represents an important tool available in the prEN 1998 [8,17]. The main explanation for this highly conservative behaviour is attributed to the absence of this hypothesis in the loading modelling of the Brazilian code. As a result, the seismic load obtained according to the Brazilian code is much higher than those provided by the European one, resulting in the observed higher lateral deflection.

Table 4. Average displacements (d), maximum base shear and average stiffness (K) by load case defined in each code adopted.

Code	1st Floor		2nd Floor		Base Shear (kN)	K (kN/m)
	d (mm)	Rel. d (%)	d (mm)	Rel. d (%)		
NBR 15421:2023	4.3	0.1780	5.6	0.2340	183	32,565
NP EN 1998:2010 Type 1 [39]	3.1	0.1292	4.1	0.1688	133	32,748
NP EN 1998:2010 Type 2 [39]	3.1	0.1292	4.1	0.1688	133	32,810
UNI EN 1998:2007 [40]	2.7	0.1104	3.4	0.1427	114	33,285

After a preliminary evaluation of performance carried out using the original connector configuration from the tested building by De Matos et al. [6], the same seismic loading was applied to the modified models. The obtained results can be seen in Tables 5, 6, 7 and 8.

Table 5. Average displacements (d), relative displacement considering the storey height (%), maximum base shear, and average stiffness (K) by model for NBR 15421:2023 [19].

Model	1st Floor		2nd Floor		Base Shear (kN)	K (kN/m)
	d (mm)	Rel. d (%)	d (mm)	Rel. d (%)		
1	4.2	17.81	5.6	23.33	183	32,957
2	4.2	17.60	5.5	23.02	181	32,702
3	4.2	17.50	5.5	22.81	179	32,760
4	5.3	22.08	6.6	27.40	190	28,966
5	5.2	21.67	6.6	27.29	189	28,869
6	5.2	21.67	6.6	27.29	189	28,869
7	6.3	25.94	7.6	31.46	193	25,539
8	7.1	29.48	8.4	35.00	195	23,174
9	11.0	45.94	12.3	51.15	197	16,083

Table 6. Average displacements (d), relative displacement considering the storey height (%), maximum base shear, and average stiffness (K) by model for NP EN 1998:2010 Type 1 [39].

Model	1st Floor		2nd Floor		Base Shear (kN)	K (kN/m)
	d (mm)	Rel. d (%)	d (mm)	Rel. d (%)		
1	2.5	10.42	3.3	13.44	106	32,958
2	2.5	10.21	3.3	13.54	105	32,305
3	2.5	10.21	3.3	13.54	104	32,569
4	3.1	12.71	3.9	16.04	111	28,743
5	3.1	12.71	3.8	15.73	106	28,156
6	3.1	12.71	3.8	15.73	106	28,156
7	3.6	15.00	4.4	18.33	112	25,461
8	4.1	17.08	4.9	20.31	113	23,202
9	6.4	26.67	7.1	29.58	115	16,159

Table 7. Average displacements (d), relative displacement considering the storey height (%), maximum base shear, and average stiffness (K) by model for NP EN 1998:2010 Type 2 [39].

Model	1st Floor		2nd Floor		Base Shear (kN)	K (kN/m)
	d (mm)	Rel. d (%)	d (mm)	Rel. d (%)		
1	3.1	12.92	4.1	16.88	133	32,738
2	3.1	12.71	4.0	16.77	113	32,544
3	3.1	12.71	4.0	16.67	130	32,505
4	3.8	15.94	4.8	19.79	138	29,067
5	3.8	15.73	4.8	19.79	137	28,861
6	3.8	15.73	4.8	19.79	137	28,861
7	4.6	18.96	5.5	22.71	140	25,649
8	5.2	21.46	6.1	25.42	141	23,134
9	8.0	33.23	8.9	36.88	143	16,174

Table 8. Average displacements (d), relative displacement considering the storey height (%), maximum base shear, and average stiffness (K) by model for UNI EN 1998:2007 [40].

Model	1st Floor		2nd Floor		Base Shear (kN)	K (kN/m)
	d (mm)	Rel. d (%)	d (mm)	Rel. d (%)		
1	2.7	11.04	3.4	14.27	114	33,203
2	2.7	11.04	3.4	14.38	112	33,577
3	2.6	10.83	3.4	14.17	111	32,821
4	3.4	13.96	4.2	17.50	121	28,693
5	3.3	13.65	4.1	17.08	119	29,153
6	3.3	13.65	4.1	17.08	119	29,153
7	4.1	16.77	4.9	20.42	124	25,308
8	4.6	19.27	5.5	22.92	127	23,093
9	7.6	31.56	8.5	35.21	136	16,109

As expected, the changes in the stiffness resulted in varying responses to the seismic excitation. The objective of the proposed parametric evaluation was to achieve less stiff structures by using fewer steel connectors, as they play a lead role in the overall response of CLT structures under lateral loading. These modifications resulted in slightly higher vibration modes and reduced the impact of the seismic action on the structure. This occurs because as the building's natural frequency increases, it moves away from the dominant frequencies of the response spectrum, where the energy associated with the excitation tends to decrease.

On the other hand, the differences noted in the results presented are insufficiently significant to induce a substantial change in natural frequency, which may lead to the previously mentioned remarks. In this context, Figure 14 illustrates the variations in stiffness resulting from the reduction in the AE116 angle brackets number. However, it is imperative to consider not only the overall stiffness but also the relative differences compared to prior modifications in the model to assess the benefits achieved through the reduction in the number of steel connectors.

Performance losses are deemed acceptable when they yield advantages in terms of a more cost-effective design, provided that safety and maintenance are not compromised. Notable reductions in stiffness were recorded with the decrease of angle brackets number at the base, as evidenced by the differences between models 3 and 4, as well as 6 and 7. Models 8 and 9 exhibited a significant decline in stiffness, rendering them unviable options for structural enhancement.

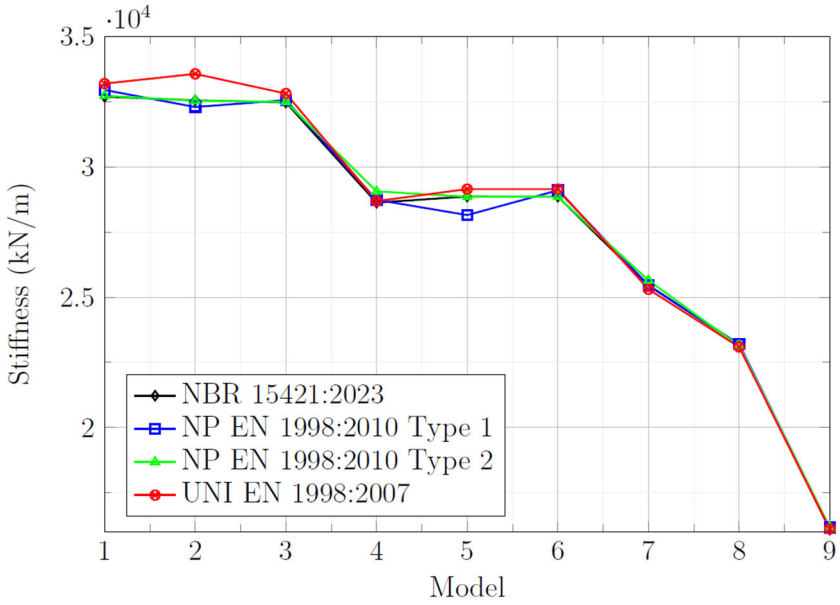


Figure 14. Stiffness changes through considered models.

The reduction in structural components observed between models 3 and 6 merits further examination, as it may not indicate a global stability risk but rather a potential advantage in terms of cost-effective design. Model 3 reached a 37.5% reduction in the total number of angle brackets, while model 6 achieved a higher reduction of 50%. Despite this outcome, the impact on structural stiffness was relatively minor: model 3 achieved an average stiffness of 32,644 kN/m across all load cases, while model 6 achieved 28,997 kN/m. These values represent a change of less than 1% and 13%, respectively, decrease when compared to the original configuration tested by De Matos et al. [6]. Both models remain within normative limits for structural performance, suggesting that these reductions will not compromise safety. However, when comparing the two, model 6 stands out as the more economical option, particularly in terms of seismic performance criteria, making it the preferred choice for a cost-effective yet compliant design.

5.3. Analysis Under Ductile Class 2 (DC2)

As noted previously, assuming ductile classes beyond DC1 requires improved detailing of the numerical model to enhance its energy dissipation capabilities. The present study incorporated screwed connections in accordance with values documented in the existing literature. However, the analysis performed in the present section was exclusively conducted on the models selected in the prior section, following the examination detailed in the preceding analysis. The results obtained under DC2 for models 3 and 6 of the original configuration are presented in Tables 9 and 10.

Table 9. Average displacements (d), maximum base shear and average stiffness (K) by load case for model 3—DC2.

Code	1st Floor		2nd Floor		Base Shear (kN)	K (kN/m)
	d (mm)	Rel. d (%)	d (mm)	Rel. d (%)		
NP EN 1998:2010 Type 1	2.3	0.0927	2.9	0.1208	91	31,376
NP EN 1998:2010 Type 2	2.1	0.0875	2.7	0.1125	85	31,541
UNI EN 1998:2007	2.4	0.0990	3.1	0.1292	98	31,723

Table 10. Average displacements (d), maximum base shear and average stiffness (K) by load case for model 6—DC2.

Code	1st Floor		2nd Floor		Base Shear (kN)	K (kN/m)
	d (mm)	Rel. d (%)	d (mm)	Rel. d (%)		
NP EN 1998:2010 Type 1	2.3	0.0938	2.8	0.1167	78	27,886
NP EN 1998:2010 Type 2	2.8	0.1167	3.5	0.1458	97	28,235
UNI EN 1998:2007	2.6	0.1083	3.3	0.1354	91	28,003

It was possible to notice the higher differences in determined load cases, such as the Type 2 Portuguese RSC. The range was from 26.70% to 32.38% in this scenario. A comparison between ductile classes is presented in Tables 11 and 12.

Table 11. Model 3 displacement results comparison between DC1 and DC2.

Code	1st Floor		2nd Floor		Avg. Relative Difference (%)
	DC1 (mm)	DC2 (mm)	DC1 (mm)	DC2 (mm)	
NP EN 1998:2010 Type 1	2.5	2.3	3.3	2.9	10.06
NP EN 1998:2010 Type 2	3.1	2.1	4.0	2.7	32.38
UNI EN 1998:2007	2.6	2.4	3.4	3.1	8.26

Table 12. Model 6 displacement results comparison between DC1 and DC2.

Code	1st Floor		2nd Floor		Avg. Relative Difference (%)
	DC1 (mm)	DC2 (mm)	DC1 (mm)	DC2 (mm)	
NP EN 1998:2010 Type 1	3.1	2.3	3.8	2.8	26.06
NP EN 1998:2010 Type 2	3.8	2.8	4.8	3.5	26.70
UNI EN 1998:2007	3.3	2.6	4.1	3.3	20.36

The findings indicate that increasing the flexibility of the structural model combined with the effective implementation of energy dissipation mechanisms has a notable impact on the overall performance, especially when Ductile Class 2 is considered in the design. The behaviour factor implemented by the European standard was able to reduce seismic loading and consequently the effects on lateral displacements, even though a less stiff structure was being tested. This is particularly evident in model 6, which achieved a maximum displacement reduction of 32.52% on the second floor. Model 3, on the other hand, achieved a 25.14% reduction. The approximate 7% difference between these two models suggests that using the energy dissipation consideration presented by DC2 can be even more effective in more economical configurations.

It is worth noting that these findings emphasise the importance of enhancing both structural flexibility and energy dissipation mechanisms. By doing so, engineers can not only meet normative requirements but also achieve more cost-effective designs. For instance, model 6′s significant displacement reductions, despite a reduction in the number of connectors, point to the potential for reducing material costs without compromising safety or performance. Moreover, the results support the broader notion that structures designed for ductile response are better equipped to handle seismic forces, reducing the likelihood of catastrophic failure during an earthquake. Thus, the use of DC2 shall be a key tool in seismic design strategies, particularly for regions that may not traditionally experience high seismic activity but still require resilient structures. The balance between structural stiffness, energy dissipation, and material consumption and efficiency is crucial for safety

and costs, making efficient design strategies, such as model number 6, a compelling case for future design practices.

6. Conclusions

A numerical analysis was conducted on a two-storey cross-laminated timber building to assess its performance under seismic loading. The validation was carried out considering experimental data from previous research [6]. The study accounted for differences in soil and regional characteristics according to prEN1998:2021 [16,17] and NBR 15421:2023 [19], applying seismic loading to the structure. A parametric analysis of the building's geometry was conducted, comparing the Brazilian and European calculation methods, particularly evaluating energy dissipation in the latter. The use of steel connectors, specifically the number of AE116 angle brackets, across both floors was examined in nine models, which indicated that the reference building tested was overdesigned for the region considered.

During the seismic analysis within Ductile Class 1 (DC1), it was possible to observe that the change in the structure's overall stiffness did not significantly increase its natural frequency, representing a minor impact on the structural outcome. The structural response in all scenarios was under the limits imposed by the standards. The induced displacement in the structure was 160% higher according to the Brazilian code than the Italian one, which would represent, theoretically, a higher seismic hazard. When compared to the Portuguese National Annex, the difference achieved with the Brazilian code was 138%. These results highlight a need for revision in NBR 15421:2023, since locations with higher seismic activity resulted in significantly lower seismic loads.

The parametric study revealed that the reduction in the number of steel connectors enabled the increase in the displacements within the safety limits in all scenarios. Still, the reductions carried out in models 7, 8, and 9 could increase displacements and reduce lateral stiffness in the order of 30%. While this change in stiffness is significant, it is not considered economically relevant when compared to the other scenarios. As expected, reducing the number of ground connectors resulted in considerable reductions in lateral stiffness, as observed in the differences in change from models 3 to 4 and 6 to 7. Thus, reducing the number of connectors on higher floors could lead to a more economical design without significantly compromising lateral stiffness. This is a topic that merits further studies to evaluate its application in multi-storey buildings.

Regarding the results obtained using the DC2 hypothesis, the Portuguese National Annex induced a displacement reduction of up to 32.38%, while in the Italian one this maximum reduction was 20.36%. The results show that the use of behaviour factors could lead to important displacement reductions. The required counterparts by Eurocode 8 are of easy implementation to computational models, especially when considering their benefits to the design outcome. Thus, this research showed the importance of considering energy dissipation when assessing the seismic impact on buildings. The European code, combined with both national annexes studied, was found to be less conservative than the Brazilian code. This is related to the behaviour factor used in the load calculation in DC1 and DC2. The Brazilian standard only considers the characteristic load values, which resulted in significantly higher displacements. The disparity between codes became even higher with improved results using the European methodology under the DC2 hypothesis. Therefore, it can be concluded that the Brazilian standard would benefit from a comprehensive review to improve the consideration of energy dissipation, thereby ensuring safer and more efficient seismic designs.

The presented modelling framework is efficient and easy to implement in future projects. The assumptions made could be further explored in future studies to assess their

impact on the building's response, such as the consideration of friction, the hysteretic behaviour of metallic connectors, and the incorporation of new pavements.

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