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PUSHOVER ANALYSIS OF A 12-STOREY CROSS-LAMINATED TIMBER BUILDING

Luka Naumovski (1), Matija Gams (2), Tomaž Pazlar (3), Boris Azinović (4)

(1) *Research Assistant, Slovenian National Building and Civil Engineering Institute, luka.naumovski@zag.si*
(2) *Assist. Prof., University of Ljubljana, Faculty of Civil and Geodetic Engineering, matija.gams@fgg.uni-lj.si*
(3) *Head of Section for Timber Structures, Slovenian National Building and Civil Engineering Institute, tomaz.pazlar@zag.si*
(4) *PostDoc Researcher, Slovenian National Building and Civil Engineering Institute, boris.azinovic@zag.si*

Abstract

Due to recent developments in net-zero policy, cross-laminated timber (CLT) structures, with their low carbon footprint and potential competitiveness with steel and concrete structures, have gained popularity and are also considered in earthquake-prone regions. The present study analyses whether a seismically inadequate 12-storey residential building from the 1960s can be replaced with a 12-storey platform-type CLT building. The structural model is developed in two steps. In the first step, a nonlinear model of a single shear wall is calibrated against experiments and data from the literature. The CLT shear walls are modelled as elastic orthotropic shell elements and the various connections (wall-to-foundation and wall-to-floor) as nonlinear spring-link elements. The model is expanded in the second step to the entire structure and analysed using the basic and extended N2 method and displacement-based design (DBD). Response and seismic behaviour are assessed by analysing global and local limit states. The critical results of the analyses reveal large lateral displacements and local failures of connections. Based on the nonlinear analysis and the assumptions used in the model, it is demonstrated that a multi-story CLT platform-type building could be considered also for regions with moderate seismic risk, such as Ljubljana.

Keywords: *CLT, timber engineering, multi-storey timber buildings, earthquake engineering, displacement-based seismic design, pushover analysis, seismic design, N2 method.*

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1. Introduction

The potential of Cross laminated timber (CLT) can play a vital role in meeting the challenges of the 21st century construction industry. As a result, it is increasingly being used for mid-rise multi-story buildings such as condominiums, commercial buildings, offices and public buildings. Nevertheless, seismic design of taller timber buildings has only been a topic of mostly isolated research for less than 20 years, with the current state of knowledge still lagging behind that of concrete or steel and currently not included in the EN 1998-1:2004 [1].

The most extensive experimental studies of full-scale single and multi-story CLT structures have been conducted as part of the SOFIE project (e.g. [2]–[4]). Experimental shake table tests were carried out on one, three and seven-story platform-type buildings. All investigated structures were erected with narrow or segmented CLT panels with typical hold-downs (HD) and angle brackets (AB). Other test programmes included full-scale CLT buildings and CLT as a seismic force resisting system (e.g., [5]–[7]), monotonic and cyclic loading tests of CLT walls (e.g., [8]–[10]), cyclic behaviour of HD and AB, and presence of shear-axial interaction (e.g., [11]–[15]). Nevertheless, the overall performance of CLT based structural systems is not fully understood and tested in real earthquake scenarios. Therefore, further experimental and numerical investigations of CLT structures, substructures, and connections are necessary to fully identify the potential undesirable behaviour.

Considering that CLT panels are relatively rigid and delivered prefabricated or in modules on the construction site, connections play a critical role in the assembly and behaviour of the structure. In addition, the properties of the connections (e.g., stiffness, slip modulus, orientation) affect the final stiffness of the system and thus the vibration period and mode shapes of the structure, which indirectly affects the seismic forces. Thus, it cannot be argued that the behaviour of CLT structures and the ability to dissipate energy are directly affected and modified by all types of connections. Several studies and manuals have introduced linear and nonlinear modelling techniques for the design. The mechanical connections have been modelled by elastic and nonlinear springs, links, trusses, and frame elements, however friction has not been directly considered (e.g., [10], [16]–[20]).

The new generation of Eurocodes, Chapter 8 within Eurocode 8, will address many topics and areas of the design process that are not present in the current version, including CLT-based structural systems with definitions of dissipative and non-dissipative zones, and q - behaviour factors recommendations based on different dynamic analyses for different ductility classes, see e.g. [21], [22]. Such an approach will lead to an optimisation of the design process, since the current practice, for linear analysis assumes parameters based on engineering judgement, which in most cases, for a more accurate investigation, requires implementation of the nonlinear analysis. Finally, a new procedure for the application of nonlinear static or pushover analysis will be defined [23].

This paper aims to provide basic information on the mechanical performance of CLT platform type structures consisting of CLT panels for high-rise buildings subjected to seismic loads. The response of individual walls and ductile connections was simulated based on literature data and previously conducted experiments on the cyclic shear resistance of individual walls and connection elements. A nonlinear computational shear wall model with material orthotropy and the nonlinear behaviour of the connection elements was developed and further validated. The modelling principle was later on utilised in a three-dimensional nonlinear model used for the nonlinear analysis of the case study high-rise building. The structure consisted of large monolithic walls with low to moderate energy dissipation capacity. Three types of traditional connection systems were implemented: hold-downs (HDs), angle brackets (ABs) and screwed or nailed panel-to-panel vertical joints. The spectral displacements obtained by the N2 method were compared with the predefined limit states for the global and local limit states. Finally, the extended N2 and the calculated correction factors were used to estimate torsional effects.

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2. Model description and assumptions

In this study, the nonlinearities that occur in a CLT building during a seismic action are represented by nonlinear springs and link elements combined with rigid frame elements. In the proposed model for numerical simulations, which adopts a combination of approaches by Yasumura et al. [9] and Follesa et al. [16], they are represented by spring or link elements with negligible length between two nodes. Depending on the options provided by the software used - *RFEM* 5.24 or *ETABS 19* – spring or link elements were selected. *RFEM* was chosen for the design as it is more commonly used by practising engineers in the design of CLT structures, however, for the pushover analysis of the case study building, the used version lacked an appropriate force lead - displacement controlled solver. Therefore, *ETABS* was implemented as a substitute for the nonlinear pushover analysis.

As shown in Fig. 1 with a pair of CLT wall panels and connections at the base of the building and connections with the walls of the upper floor, five types of FE elements are used for a 3D model:

- 4-noded, 24 DOFs, shell elements with membrane and bending capabilities for the CLT wall panels with a typical mesh of 0.5 x 0.5 m and appropriate material properties,
- nonlinear tension-only link or spring elements as a ductile HD connection,
- nonlinear shear-only link or spring elements as a ductile AB connection,
- nonlinear compression and friction elements as parallel to grain compression and friction,
- linear link or spring elements with suitable stiffness as self-tapping screws (STS).

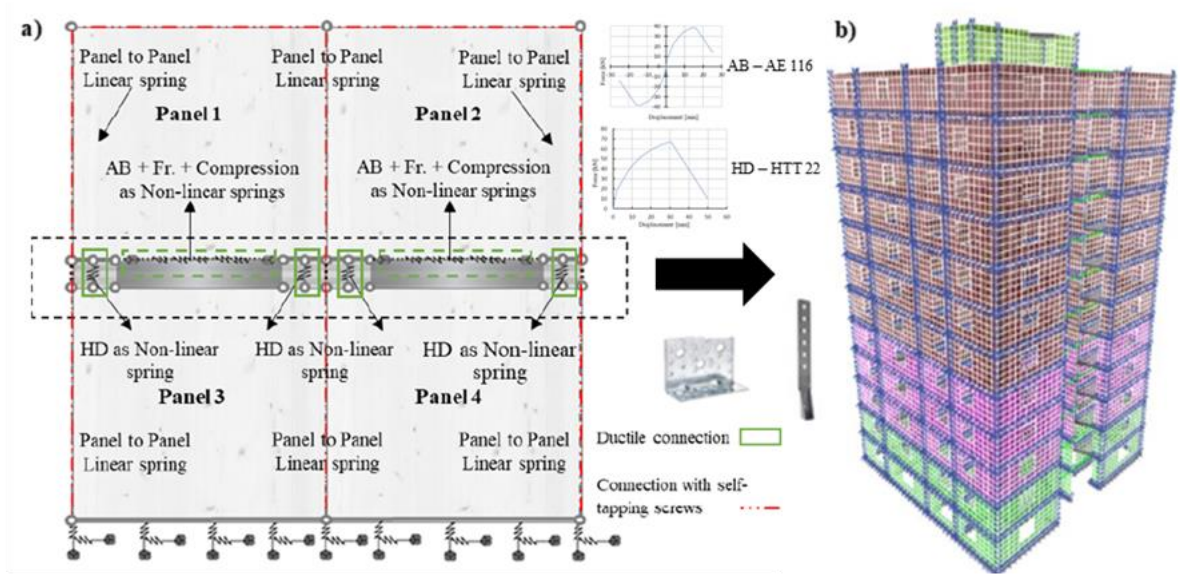

Figure 1. a) Proposed modelling principle and input parameters used for the shear-only and tension-only connection elements; b) Model application – a 12-storey case study building.

The model is based on some simplified assumptions:

- connections between perpendicular walls are assumed to be rigid,
- connections between floors and supporting walls are assumed to be rigid,
- perpendicular and parallel to grain compression properties of timber are not considered,
- the simultaneous presence of shear-axial interaction in the HDs and ABs is not addressed,
- the friction model is based on constant friction coefficient values from experimental data [24],
- the obtained vertical load from the top reactions of every wall panel, based on the linear analysis for vertical loads, is applied directly on wall panels,
- masses are concentrated at mass centres of floors,
- floors are considered as in-plane rigid diaphragms.

For more detailed information on the model principles and assumptions please refer to [25].

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The hysteretic and monotonic behaviour of six different configurations of tested CLT wall panels in the OPTIMBERQUAKE project was used to validate the proposed model, as listed in Table 1 and shown in Fig. 2. Please refer to [9] and [13] for further reading and additional data.

Table 1. List of the wall setups for model validation [9].

Test series	Test	Anchoring	Support	Vertical load	Protocol
I + II	W-CLT-1.1	2 HD, 3 AB	rigid (steel)	10 kN/m	monotonic
	W-CLT-1.2	2 HD, 3 AB	rigid (steel)	10 kN/m	ISO
	W-CLT-2.1	2 HD, 3 AB	rigid (steel)	50 kN/m	monotonic
	W-CLT-2.2	2 HD, 3 AB	rigid (steel)	50 kN/m	ISO
IV	W-CLT-4.3	2 HD, 3 AB	rigid (steel)	100 kN/m	ISO

For the sake of comparison, the numerical models reproduced the geometry, mechanical properties, loading, and boundary conditions of the tested specimens and were analysed using a multi-step nonlinear static analysis. The numerical model in *RFEM* was used for the purpose of investigating the influence of friction and contribution to overall top displacement, while the suitably adapted *ETABS* model was used further for analysing the case study building. The models were validated by comparing the shear load-displacement curve and contribution to the average total top displacement, as shown in Table 2. As expected, the deformations from the numerical simulations showed that if friction is considered, the resistance to shear and slip is higher. Moreover, a lower bearing capacity and a decrease in deformation capacity were also observed in models without friction, although the difference on a global scale was insignificant. The maximum deviation from the average test value of each contributor was ≈ 10 %. However, in the last test series, the observed strong nonlinear behaviour on both comparison levels is slightly different, indicating that the model is more suitable for low to midrange vertical loads since it does not include irreversible (plastic) deformation of CLT. Nevertheless, the observed numerical results were sufficiently accurate for using on a CLT building modelling scale, since as mentioned the margin of error between comparable values from experiments and numerical analysis was in the range of 10%.

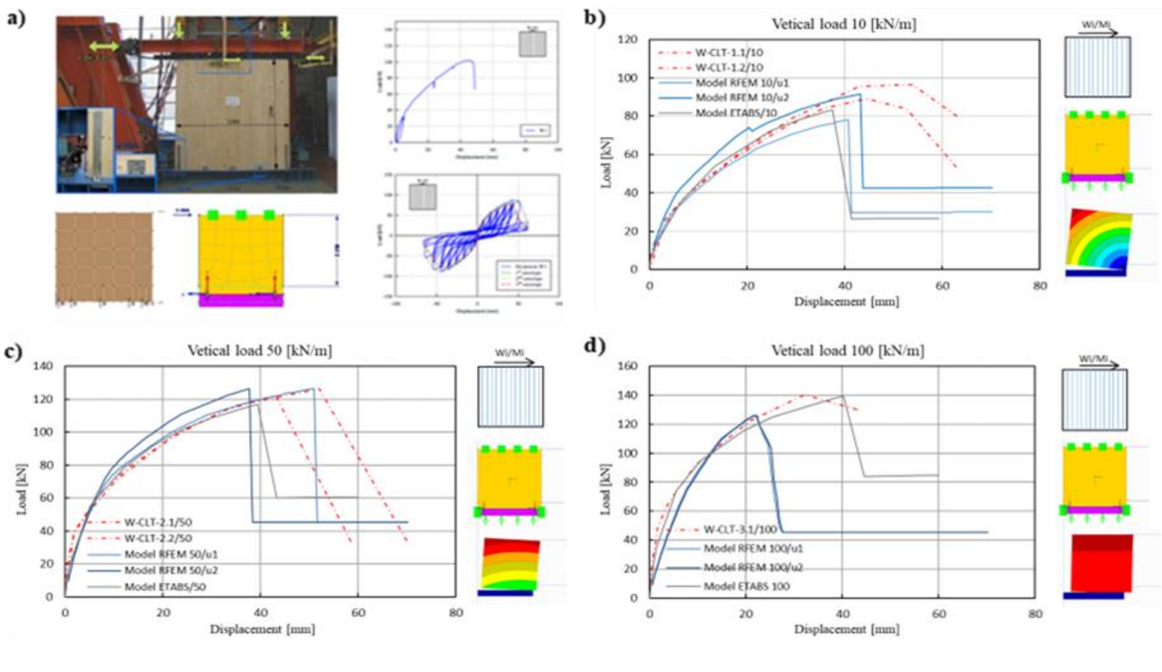

Figure 2. a) Example of a wall panel test setup [9] and nonlinear FE models [25]; Experimental and numerical shear load-displacement curves for series I, II & IV with vertical load: b) 10 kN/m, c) 50 kN/m & d) 100 kN/m.

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Table 2. Comparison of average numerical and experimental results for the overall deflection - rocking, slip and CLT deformation as a percentage of the total shear displacement at the top.

Test	Experimental value			Models 10/50/100 u1			Models 10/50/100 u2		
	Slip:	**Rocking:**	**CLT:**	**Slip:**	**Rocking:**	**CLT:**	**Slip:**	**Rocking:**	**CLT:**
W-CLT-1.1 W-CLT-1.2	26%	69.5%	4.5%	18 %	66 %	16 %	23 %	60 %	17 %
W-CLT-2.1 W-CLT-2.2	42%	47.5%	10.5%	37 %	44 %	19 %	45 %	33 %	22 %
W-CLT-4.3	43 %	44 %	13 %	64 %	9 %	27 %	66 %	7 %	27 %

**models u1 - friction coefficient = 0.5* **models u2 - without friction*

Based on the validated results, the proposed model and input data for the link and spring properties were used as a suitable approximation for the modelling and numerical simulations of the case study CLT building, which was considered as a replacement possibility of seismically inadequate unreinforced concrete and masonry multi-story apartment buildings built in the 1960s in Ljubljana, when suitable earthquake codes did not exist. The case study corresponds to the gross dimensions of the old building, with a typical floor plan in a rectangular shape of 21.7 m x 19.6 m used in all floors and a maximum height of 39.6 m with a storey height of 3.0 m. The CLT building was proposed due to its fast erection time and improved sustainability aspects when compared to other alternatives.

The main load-bearing structure of the proposed platform type building consists of massive CLT wall and floor panels with various thicknesses and lamellae layout (CLT cross-section reduces with storey level), and steel beams along the edges or the cantilever balconies for larger spans, as illustrated in Fig. 3. The vertical means of communication, elevators and stairs, are located in the CLT core, which consists of panels subdivided according to the maximum available production length and width. Transport logistics are also taken into account. Thus, the core is divided into three vertical segments, two segments of 16.0 m and 7.0 m, with different widths between 2.4 – 3.5 m and a total height of 39 m. This warrants a rigid box-like behaviour of the core. The perimeter of the building consists of panels with openings, while the other CLT panels are massive - without any openings. The complete design process of the cross-sections was carried out in three separate phases: initial selection with preliminary design tables, *RFEM* FE analysis for vertical loads, and final 2D check and utilisation of CLT panels performed in *Calculatis* [26]. In addition, the structure is assumed to be placed on a rigid reinforced concrete platform on soil type A. Finally, a conservative behaviour factor of 2.0 (as for monolithic shear walls) was assumed for the analysis.

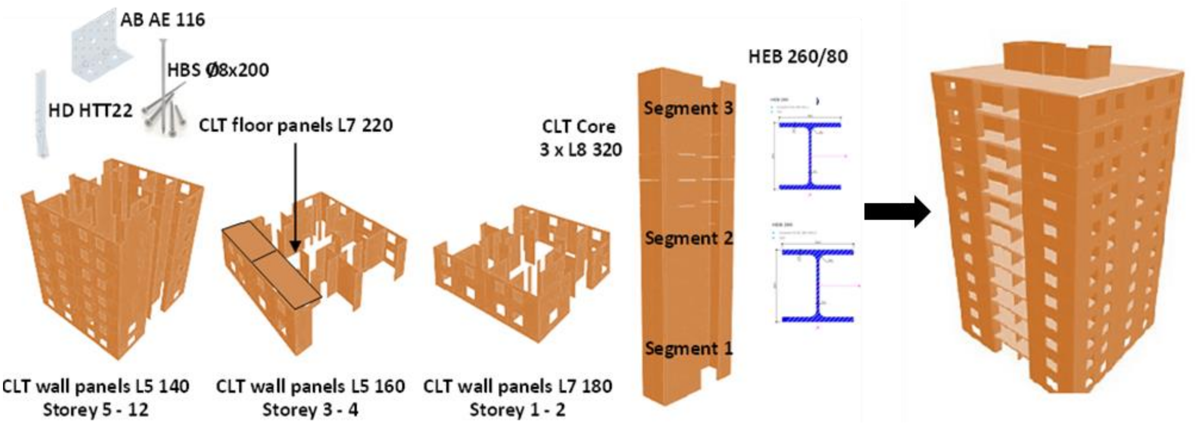

Figure 3. Final sizes of structural elements, CLT wall and floor panels type, CLT core segments and connection types considered for the design & seismic analysis of the case study building.

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3. Results

3.1 Linear analysis

The behaviour of the structure was studied with the numerical models in *RFEM 5.24*. and *ETABS 19*. Initially, only the vertical load model was created, followed by the modified seismic model that takes into account the stiffness parameters of the connections, the seismic joint masses and the mass moments of inertia. All CLT panels were modelled as 2D shell elements with respect to their principal bearing direction. The core was modelled as a continuous element along the length of one segment, connections between adjacent core panels were completely hinged. The steel members were modelled as 1D frame elements with moment end releases, while the connections between adjacent panels were modelled by releasing the corresponding rotational DOFs for moment transfer by implementing line-hinges. AB and HD stiffness parameters were assigned within the line hinges or line releases as linear springs. The rigid, box-like behaviour of the core was replicated with a nodal constraint inserted as a diaphragm at each floor level of the CLT core. All of the supports were hinged line supports. The automatic FE mesh size was set to 0.5 x 0.5 m.

The seismic design for the assumed location in Ljubljana, corresponding to a seismic action of 0.25g, was performed in three phases, as shown in Fig. 4. First, the lateral force method provided the required number of shear connectors calculated from the distributed base shear per story. This was followed by optimisation using the response spectrum method, which provided the final arrangement of AB connections: storey 1 – 5 at ≈ 0.5 m, storey 6 – 10 at ≈ 0.5 - 1.0 m, and storey 11 – 12 at ≈ 1.0 - 1.5 m with a shear stiffness of 5000 kN/m per AB connection. HD connections were placed only at panels edges with an axial stiffness of 10000 kN/m per connection. Finally, vertical step joints between wall panels with a stiffness of 1500 kN/m per spring or 10500 kN/m per wall-to-wall connection were introduced as Self-tapping screws (STS) connections.

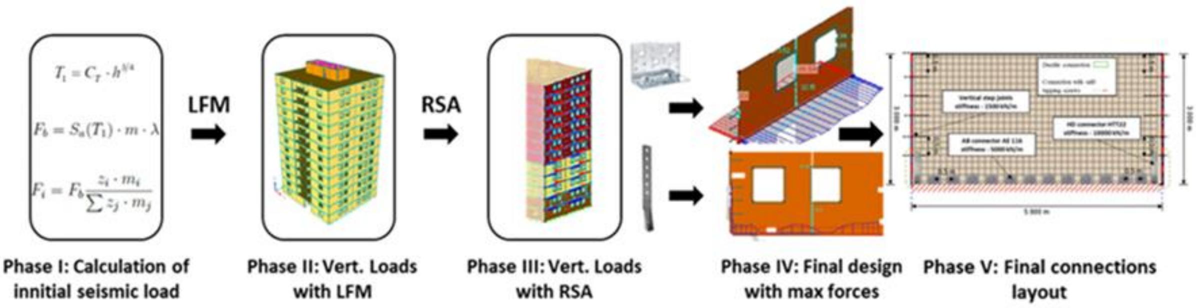

Figure 4. Phases of the seismic design procedure, with LFM as lateral force method and RSA as response spectrum analysis.

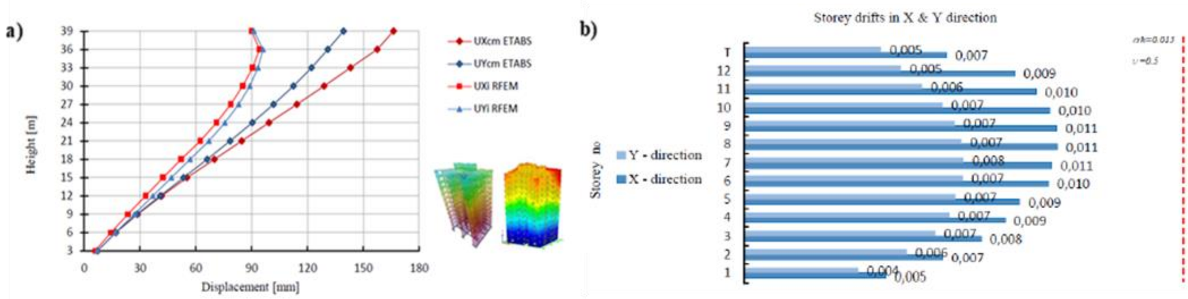

Figure 5. a) Displacements (U) of mass centres (cm/i) in X and Y direction for the ETABS and RFEM model – elastic response spectrum analysis; b) Limitation of interstorey drift check for damage limitation requirement as specified in EN 1998-1:2004 in X and Y direction for the ETABS model.

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The obtained results from the linear analysis, as shown in Fig. 5, as expected indicate that the model in *RFEM* which incorporates line-hinges as linear springs is stiffer than the one made in *ETABS* with individual joint links. The difference is even more evident in higher stories, where the distance between individual elements becomes larger. Furthermore, the modal mass participation factor of the *RFEM* model is lower for the first two periods of vibration than in the *ETABS* model, which could also lead to an overall decrease of top storey displacements. Finally, the intersotey drift check as defined in EN 1998-1:2004 satisfied the selected design criteria.

3.2 Pushover analysis

The input parameters of the nonlinear ductile connection elements of the nonlinear model in *ETABS* were assumed to be the same as in the validated nonlinear modelling proposal. In addition, the typical wall scheme of the proposed model, as shown in Fig. 1, was extended to the entire structure. The first step of the pushover analysis was to apply gravity loads, followed by the lateral loads, which were displacement controlled at every storey centre of mass for X and Y direction separately and were monotonically increased in proportion to the predefined load pattern, uniform or modal, as specified in Chapter 4.3.3.4.2.2 of EN 1998-1:2004. The reference point, which was selected as the specified displacement control node for automatic termination of the analysis, was located at the roof slab. The pushover curve was created using the base shear, which was calculated as the sum of all reaction forces in the direction of lateral load application, and the top displacement at the reference point, which was recorded at each incremental step during the analysis, see Fig. 6.

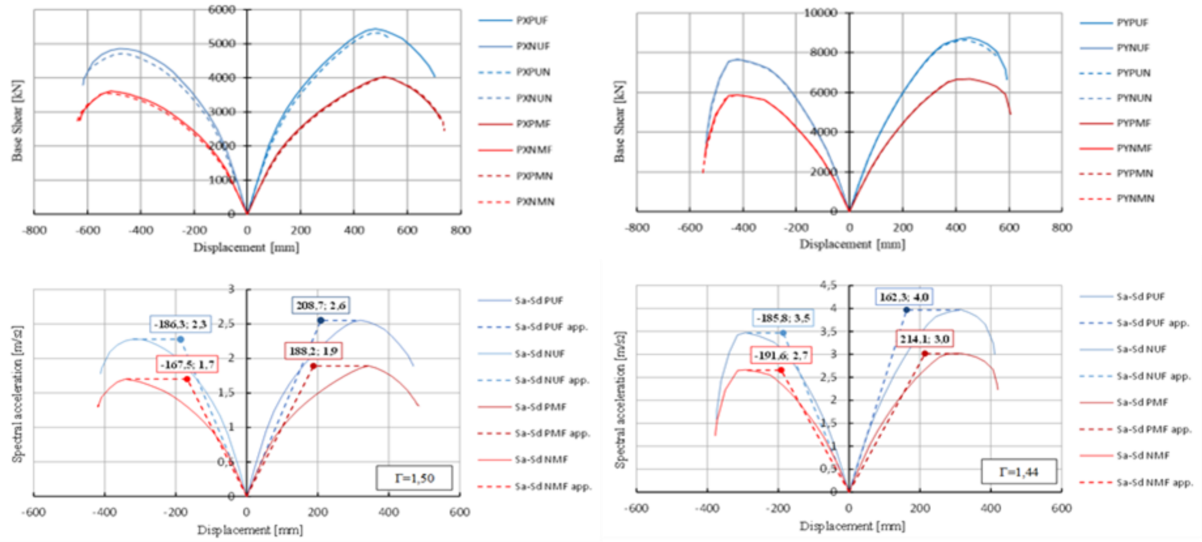

Figure 6. (above) Capacity curves from nonlinear pushover analysis in the positive (P) and negative (N) X (left) & Y (right) -direction for modal (M) and uniform (U) load pattern, model with friction (F) and without (N); (below) Bilinear approximation of transformed SDOF capacity spectrum curves (Sa – Sd) in positive (P) and negative (N) X (left) & Y (right) -direction for modal (M) and uniform (U) load pattern with friction (F) and without (N).

An important step before determining the performance points, as the intersection between the demand spectrum and the capacity spectrum, is to convert the capacity curves into a capacity spectrum type curves, i.e., *Sa* - *Sd* diagram. The method is based on the idea that an MDOF system of a multi-story building can be transformed into an equivalent SDOF system [27]. This is necessary because it allows the response spectra to represent the seismic action and the building capacity curve to be compared with the demand. In addition, to determine the performance points using the N2 method based on the inelastic spectrum and the $R_\mu - \mu - T$ relationship given in EC8, a bilinear idealisation that applies the equal energy principle is required.

The performance evaluation of the reference building was performed using limit states predefined at two levels: (*i*) locally on individual elements and (*ii*) globally as structure occupancy levels. The local

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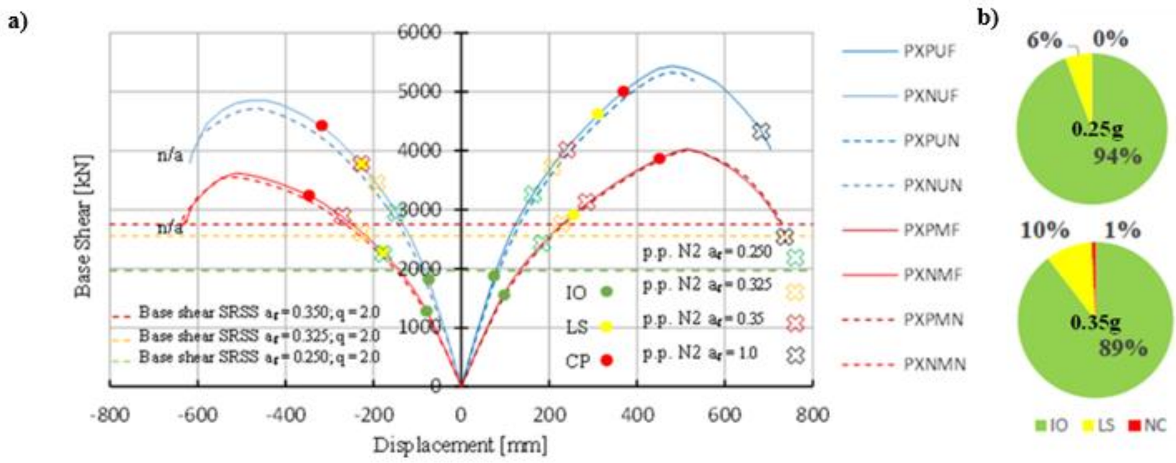
levels were defined as specified in FEMA 356 (IO – Immediate Occupancy, LS – Life Safety and NC – Near Collapse) and are based on the achieved deformation of the implemented nonlinear ductile connections, AB and HD units, respectively. On the other hand, the global occupancy levels are associated with the predefined local levels, where: IO - the first element reaches the LS acceptance limit; LS – the first element reaches the NC acceptance limit; NC – more than 10 elements reach NC acceptance limit.

Based on the obtained results, listed in Table 3, it is noticeable that the performance points are mostly in a range where the period is higher than the upper limit of the period of the constant spectral acceleration branch – T_C. At the same time, performance points for the modal load pattern in the X-direction were mainly found near the branch with constant spectral displacement.

Table 3. Calculated spectral displacement values of performance points for different demand spectra, direction and load pattern – uniform (E) and modal (M) for an equivalent SDOF system – N2 method.

Demand a_g max [g]	**Spectral displacement of the performance point [mm]**							
	XM+	XM-	YM+	YM-	XE+	XE-	YE+	YE-
0.250	122.7	122.5	104.0	105.0	111.7	108.1	80.3	93.8
0.325	159.9	159.3	135.3	136.5	145.0	140.5	102.7	121.9
0.350	172.7	171.9	145.7	146.8	159.2	151.3	110.8	131.5

The capacity curves shown in Fig. 7 and Fig. 8 with the indicated global limit states and the calculated spectral displacements for different demand spectra – performance points, and the total base shear for different demand spectra, are used to show the behaviour of the building on a global level. The results confirm that both loading patterns – uniform and modal should be considered for design, as both could be critical for undesirable structural behaviour at both global and local level. The patterns implicitly account for the different deformations considered in the simplified model – bending, rocking, shear, and sliding – inherent for CLT structures. Nevertheless, the spectral displacement from the N2 method is significantly higher for the modal load pattern. As we can see from the obtained capacity curves in both perpendicular directions, the analysis in positive and negative direction is mandatory since the curves differ significantly.



**g - acceleration of gravity (9.81 m/s²); SRSS - Square Root of the Sum of the Squares; p.p – perfomance point N2 method.*

Figure 7. a) Capacity curves in positive (P) & negative (N) X-direction for uniform (U) and modal (M) load pattern with friction (F) and without (N), indicated global limit states, N2 p.p., and total base shear obtained based on different design spectrum; b) Yielding of connection elements with reference to local limit states for p.p. N2 a_g 0.25 g & 0.35 g and pushover analysis in positive X – direction with a modal load pattern.

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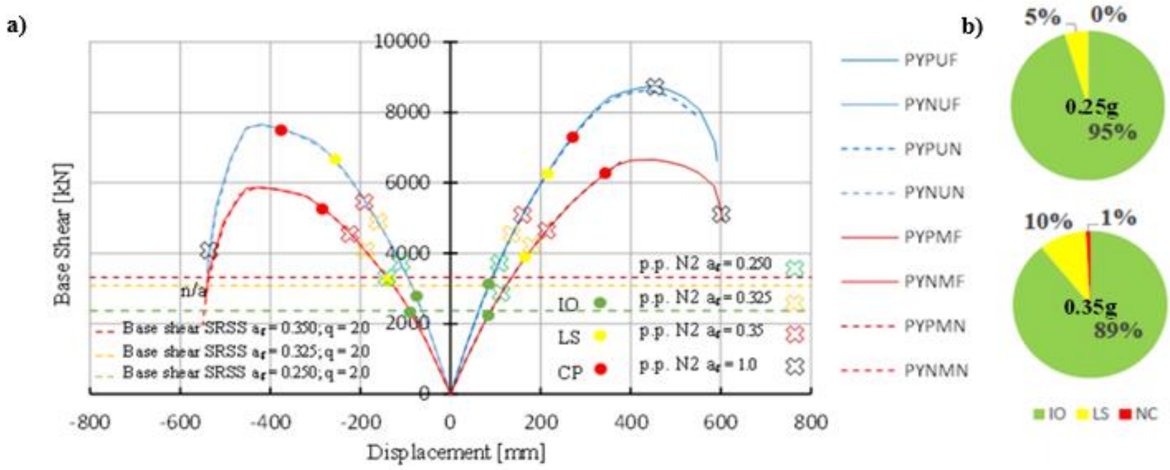

**g - acceleration of gravity (9.81 m/s²); SRSS - Square Root of the Sum of the Squares; p.p – perfomance point N2 method.*

Figure 8. a) Capacity curves in positive (P) & negative (N) Y-direction for uniform (U) and modal (M) load pattern with friction (F) and without (N), indicated global limit states, N2 p.p., and total base shear obtained based on different design spectrum; b) Yielding of connection elements with reference to local limit states for p.p. N2 a_g 0.25 g & 0.35 g and pushover analysis in positive Y – direction with modal load pattern.

As further confirmation of the overall obtained results, the calculated values for the high peak ground accelerations were in agreement with the shake table tests conducted as part of the SOFIE project, where the unscaled Kobe (1995) earthquake - 0.91g resulted in horizontal displacements of the uppermost floors of about 1.0 % to 1.6 %. In comparison, the simplified numerical model and the spectral displacement determined by the N2 method were between 1.5 % and 1.7 %, and this for a much higher and heavier building. The analysis with reference to the local limit states of connections revealed that for both directions, on average, the highest degree of damage - above the IO state - of the connecting elements occurred on the outer walls with openings. As expected, the core segments were more subjected to rocking behaviour, resulting in an increased damage to the HD units, particularly those on the ground floor. In contrast to the core wall segments, pure shear behaviour was observed in the section with the highest number of shear walls, resulting in a greater number of damaged AB units. The earlier discussed performance evaluation with reference to the local limit states and the level of reached limit state of connection elements for selected axis and the least favourable load scenarios for 0.35g peak ground acceleration are illustrated in Fig. 9. However, as already discussed, the simplified model does not account for the shear-axial interaction, especially the uplift load capacity of the ABs, which partially increases the load bearing capacity and changes the overall behaviour.

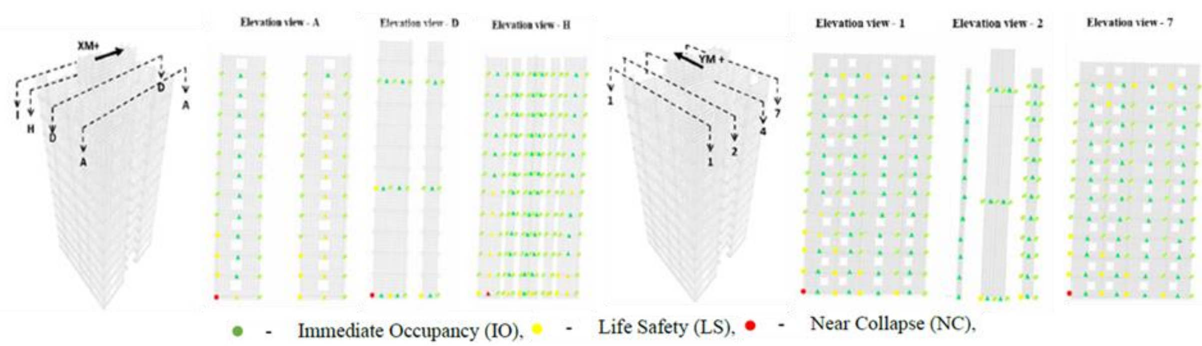

Figure 9. Yielding of connection elements for selected axes, AB and HD, with reference to local limit states for p.p. N2 a_g=0.35g and pushover analysis in positive X and Y – direction with modal load pattern.

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Although the effects of irregularities can have a significant impact on the behaviour of the structure, they are not well known for CLT structures. Here, the extended N2 estimates the possible unfavourable effect of irregularities in the geometry. The calculated correction factors differed depending on the subsets of the load pattern considered and the load direction. For both directions and patterns, the factors corresponded to values between 1.0 and 1.4, with the less stiff X - direction leading to higher values, as shown in Fig. 10. Since the extended method provides adequately conservative results for structures that are not too torsionally flexible, the estimations of the increase of the absolute displacements seem probable.

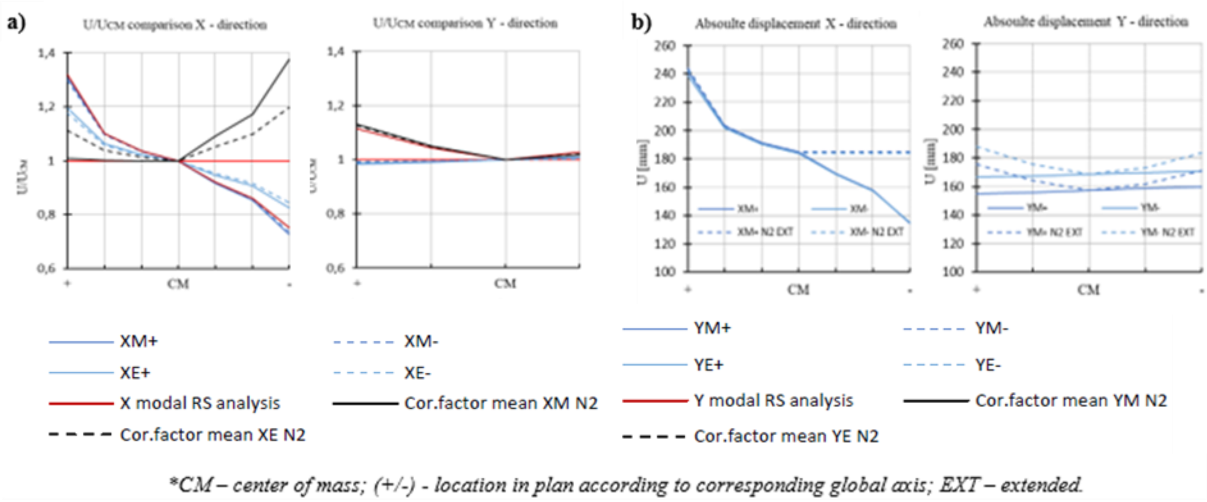

Figure 10. a) Torsional effects in terms of normalised top displacements obtained by elastic modal analysis, by pushover analysis for modal (M) and uniform (E) load pattern for the positive (+) and negative (-) direction and the corresponding correction factors for the extended N2 method – X (left) and Y (right); b) Torsional effects in terms of absolute top displacements - 0.25 g, obtained by the normal and the extended N2 method for modal load pattern for the positive (+) and negative (-) direction with the associated correction factor – X (left) and Y (right).

For further reading and more information regarding the modelling procedure and additional input data used for the case study building, as well as detailed representation of reached damage levels of connection elements with reference to local limit states please refer to [25].

4. Conclusion and future work

As architectural design proposals for tall timber buildings are becoming increasingly more present in earthquake-prone areas, seismic design is becoming increasingly important. Since multi-storey CLT-based structures are a relatively new structural system, there is still a lack of experience, uncertainty about modelling assumptions, and insufficient references, thus it is a challenge to develop an accurate nonlinear model for the design. As a result, this study investigated the seismic performance of CLT structures by developing a nonlinear computational shear-wall model of a 12-storey platform-type building. The results presented met the predefined performance design objectives for a maximum considered earthquake on both levels. At local level, the connections damage level above the IO – state ranged between 5% and 11%. In addition, a satisfactory global behaviour with yielding of connection elements distributed throughout the whole structure was also observed. The extended N2 lead to an increase of all relevant quantities on both edges in both directions, especially X. However, it was shown that the torsional effects are not significant and do not cause global failure of the analysed structure, even in cases where the global level limit states are decided upon rather conservative criteria (effects on non-structural secondary elements were not considered).

Although the results of the nonlinear static analysis and the N2 method (for a_g = 0.25 – 0.35 g), did not expose a significantly damaged CLT structure or an undesirable global failure mechanism, it should be noted that such structures in seismically active areas should be designed with caution.

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Based on the findings, we assume that the building would have likely sustained the 2020 Petrinja earthquake, which in the epicentral area resulted in PGA bedrock values from 0.29 to 0.44 g [28], with mild to moderate damage levels. In this context, PGA is not a completely reliable indicator of the damaging capacity of the earthquake, as it does include its frequency. Nevertheless, a holistic design approach considering all accidental load situations might require contradictory solutions for high rise CLT buildings, such as for example an increase of mass at top stories in order to improve wind serviceability issues. Moreover, a direct comparison between the available test results acquired from simplified numerical models and real earthquake scenarios where the increase of the building height leads to complex material and geometric nonlinearities, is not always possible. Future experimental and numerical research should therefore aim at full-scale CLT structures testing campaigns, such as the ongoing NHERI Tall Wood Project of a 10-story timber building shake table test [29] and nonlinear time-history analyses with hysteric behaviour of the ductile connections implemented into the actual model, estimation of IDA curves, and calculation of fragility curves. Finally, a resilient-based seismic design methodology for future tall timber buildings should be developed.

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