Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia
Copyright © 2025 CroCEE
DOI: https://doi.org/10.5592/CO/3CroCEE.2025.132

- Longer frames, such as Model M2.1 (6.0 m), exhibit greater load capacity and more distributed stress patterns compared to shorter frames like Model M1.1 (4.0 m), demonstrating the critical role of geometric dimensions in structural performance.
- Longer frames with infill experience more evenly distributed and less severe cracking, while shorter frames show higher stress concentrations and localized crack formations near beam-column joints and column bases.
- Advanced finite element modelling and micro-modelling techniques provided detailed insights into the structural response under varying lengths, infill configurations, and loading scenarios, accurately capturing critical performance metrics like stiffness, ductility, and energy dissipation

References

[1] MagPage, A. W. (1978). "Finite element model for masonry." *Journal of Structural Division, ASCE*, 104(8), 1267–1285.

[2] Florim Grajcevci, Flamur Grajcevci, Fatos Tahiri, Hamdi Kurteshi, “Preliminary Seismic Vulnerability Assessment of Existing Historic Masonry Building in Pristina, Kosovo”, World Academy of Science, Engineering and Technology, International Journal of Geological and Environmental Engineering,

[3] J Ristic, V Veseli, L Misini, D Ristic; “Seismic protection of masonry and infilled frame buildings using upgraded sliding isolation system with sf devices”, 2CROCEE, Zagreb , Croatia, 2023.

[4] B Nela, F Grajçevci - Proceedings of the Congresso de Mīetodos Numīericos , “Numerical approach: fem testing of masonry specimens with different bond configurations of units” …, 2019

[5] Lourenço, P. B. (1996). "Computational strategies for masonry structures." *Ph.D. Dissertation*, Delft University of Technology, Netherlands.

[6] Tomaževič, M. (1999). *Earthquake-Resistant Design of Masonry Buildings*. Imperial College Press, London. ISBN: 978-1860940660.

[7] Roca, P., Molins, C., & Marí, A. R. (2005). "Strength capacity of masonry wall structures by the equivalent frame method." Journal of Structural Engineering, 131(10), 1601–1610.

[8] Grajcevci, Florim, Dumova-Jovanoska, Elena, “Vulnerability of existing masonry buildings in function of their heights” Scientific Journal of Civil Engineering, 2019

[9] Kaushik, H. B., Rai, D. C., & Jain, S. K. (2007). "Stress-strain characteristics of clay brick masonry under uniaxial compression." *Journal of Materials in Civil Engineering*, 19(9), 728-739.

[10] Paulay, T., & Priestley, M. J. N. (1992). Seismic Design of Reinforced Concrete and Masonry Buildings. Wiley.

[11] Lourenço, P. B., & Rots, J. G. (1997). "Multisurface Interface Model for Analysis of Masonry Structures." Journal of Engineering Mechanics, 123(7), 660–668.

[12] EN 1996-1-1: Eurocode 6: Design of Masonry Structures – Part 1-1: General Rules for Reinforced and Unreinforced Masonry Structures.

[13] Dolšek, M., & Fajfar, P. (2008). "The effect of masonry infills on the seismic response of a four-story reinforced concrete frame—a probabilistic assessment." Engineering Structures, 30(11), 3186-3192.

[14] Kose, M. M. (2009). "Parameters affecting the fundamental period of RC buildings with infill walls." Engineering Structures, 31(1), 93-102.

[15] Fardis, M. N. (2009). Seismic Design, Assessment, and Retrofitting of Concrete Buildings: Based on EN-Eurocode 8. Springer.

[16] DIANA FEA BV. (2023). DIANA User's Manual.

[17] DIANA FEA BV. (2023). Engineering Masonry Module – User Guide.

[18] DIANA FEA BV. (2023). Tutorials for Nonlinear Analysis.

[19] DIANA FEA BV. (2023). Material Models for Masonry and Concrete.

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.133

PUSHOVER ANALYSIS OF TIMBER BUILDINGS WITH LIGHT FRAME TIMBER PANELS FOR ENGINEERING PRACTICE: METHOD DESCRIPTION, DISCUSSION, AND HIGHLIGHTING RESEARCH GAPS

Meta Kržan (1)

(1) *Senior researcher, Slovenian National Building and Civil Engineering Institute (ZAG), meta.krzan@zag.si*

Abstract

Timber construction is increasingly recognized as a sustainable building solution due to its low carbon footprint, renewability, and prefabrication efficiency. Its lightweight nature, combined with steel connectors, makes it especially suitable for seismic regions where reducing mass and enabling energy dissipation are vital for resilience. Prefabricated timber systems also offer economic and practical advantages, enhancing their appeal in earthquake-prone areas.

This study presents a simplified pushover analysis method developed for timber buildings with light frame panels, and designed for practical use by structural engineers. Previous research at the Slovenian National Building and Civil Engineering Institute (ZAG), in collaboration with an industrial partner, involved extensive experimental testing of light frame timber panels. These panels, made of cement-particle boards stapled to timber frames, were subjected to full-scale cyclic shear tests. Results showed that panels with cement-particle boards exhibited more ductile behavior compared to those with gypsum-fibre boards.

The study aimed to determine the behavior factor (q-factor) for a timber building under seismic loading, providing performance validation for the manufacturer’s systems and design assumptions. It also addresses a gap in current research and standards, as behavior factors for timber frames with stapled sheathing are not clearly defined in design codes, complicating engineering practice. Findings contribute valuable data for improving seismic design codes, particularly Eurocode 8 (EC8).

A pushover analysis, based on the SREMB method, was conducted on an existing building, assuming a critical-story failure mechanism. The methodology and its assumptions regarding the nonlinear behavior of timber panels are discussed, highlighting implications for seismic performance. The study underscores the need for further research and experimental validation to support the broader adoption of non-linear shar wall behaviour assumptions in engineering practice.

Keywords: Light-frame timber wall panels, Timber buildings, Simplified non-linear static (pushover) analysis, Critical storey mechanism, Shear wall nonlinear behaviour, EC5, Behaviour factor.

1. Introduction

The seismic design of timber buildings is typically carried out using linear elastic response spectrum analysis, where behavior factors (q-factors) prescribed by Eurocode 8 (EC8, i.e. EN 1998-1:2005: Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings [1]) are applied to account for the ductile capacity of the structure, depending on the type of structural system used. These q-factors reduce the elastic seismic demand to a design level by considering the expected energy dissipation capacity and ductility of the structure. However, for the tested type of wall panels used in construction - cement-bonded particle boards fastened to timber frames with metal staples [2] - the conservative q-factors recommended by EC8 may not accurately reflect their true seismic performance compared to other panel types. While experimental tests provide valuable insights into the resistance and ductility of individual panels, they do not directly allow for the evaluation of the overall behavior factor at the building level. The seismic response of a timber structure depends not only on the mechanical properties of individual panels but also on the

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.133

overall building configuration, including the interaction of structural elements and load distribution mechanisms.

Modern timber buildings, particularly residential and commercial structures, often feature irregular architectural layouts in both plan and elevation. Design trends such as large open spaces, extensive glazing on southern facades, and open-plan ground floors introduce significant structural irregularities. These irregularities can lead to uneven load distribution and complex seismic behavior, which linear elastic analysis may not fully capture. As a result, an accurate assessment of the global lateral deformation capacity of such structures is crucial to ensure reliable seismic performance. Nonlinear seismic analysis provides a more realistic evaluation by considering the behavior of structural systems beyond the elastic limit, capturing effects such as yielding, stiffness degradation, and energy dissipation. However, practical challenges exist in applying nonlinear analysis methods. Engineers often lack access to detailed structural properties, and the setup of sophisticated numerical models requires significant time and computational resources. Consequently, simplified yet reliable methods are needed to facilitate the assessment of timber structures in engineering practice, balancing accuracy with practical feasibility.

This paper presents a simplified pushover analysis method that is well-suited for engineering practice. The method is validated through a parametric study involving different nonlinear behavior assumptions for timber wall panels, derived from both experimental investigations and analytical models available in the current timber design standard, Eurocode 5 (EC5, i.e. EN 1995-1-1:2004: Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings, [3]). The study, conducted on a real timber building, highlights the significant influence of panel behavior assumptions on the calculated seismic response and underlines the need for practical guidelines for nonlinear seismic analysis of light-frame timber buildings. Additionally, based on the obtained results, the study provides an assessment of the actual seismic performance of the building under lateral loading, contributing valuable insights to support future standardization efforts.

2. Simplified pushover analysis incorporating nonlinear shear behavior of Light Frame Timber Wall Panels (LFTWPs)

2.1. Short overview of the critical storey pushover analysis method (POR/SREMB Method)

The method upgraded and used for the presented analysis of timber building with LFTP is a simple pushover analysis initially developed for the nonlinear static analysis of masonry buildings in the 1970s', i.e. POR Method [4], in engineering practice also commonly referred to as SREMB (Seismic Resistance Masonry Buildings). With certain adjustments in assumptions, it can also be applied also to other types of structure if the failure mechanism is such that the model is appropriate. The program is based on a simplified nonlinear static analysis, where vertical-loadbearing elements alone of the assumed critical story are modelled and, in the analysis, subsequently loaded with a shear force in the direction of seismic forces through controlled incremental displacements of the mass centre of the floor. At each step, the shear load and displacement of all individual load-bearing elements in the critical story are calculated and checked considering their capacity. As the walls gradually enter the plastic range, the overall stiffness of the structure decreases until the maximum shear capacity of the story is reached. Afterwards, with increasing displacement, the shear capacity of the story decreases. The limit deformation is reached either when the force drops to 80% of the maximum capacity, when a certain portion of structural elements fails, or when a specified story drift is exceeded.

The key results of the nonlinear static analysis are the information on the damage mechanism of load-bearing elements and finally the shear capacity curve or the hysteresis envelope of the critical story. This curve shows the sum of all horizontal forces in the critical story, i.e. total base shear force F_b, in dependence of the horizontal displacement of the mass centre at the top of the story d_{et} in the loading direction of the loading force F_b. The analysis is conducted separately for both orthogonal directions of the building.

The seismic resistance assessment is based on identifying the critical capacity curve that provides the lowest results in terms of evaluated intensity measure, which commonly are either the PGA (peak ground acceleration) or the SRC (seismic resistance coefficient, calculated as ratio of building shear resistance capacity to building weight).

2.2.Main assumptions of the method and input data

This method is particularly effective for assessing buildings with distinct structural characteristics and failure mechanism. The main assumptions on which the method is based are the following:

- The walls are adequately connected with horizontal ties and floor structures that are rigid within their plane, ensuring a uniform distribution of horizontal loads and imposed displacements across the walls based on their stiffness.
- The damage and failure occur in vertical load-bearing structural elements of critical storey - only vertical load-bearing elements (wall segments) between openings are considered, modelled as linear elements with a bilinear elasto-plastic response.
- Wall elements are fixed to the floor structures at their top and bottom edges.
- The seismic resistance of the story is determined by the stiffness, load-bearing capacity, and deformation capacity of the walls. Additionally, wall deformations are influenced by their position within the story due to torsional effects.
- The contribution of walls oriented both parallel and perpendicular to the earthquake direction is considered, even though the latter affects seismic resistance and stiffness to a lesser extent than walls aligned with the seismic load direction.
- Walls carry their share of the horizontal load until their deformations exceed limit deformations (failure).

The input data for the analysis therefore include:

- Wall geometry, which defines the shear response of the walls and how seismic forces are distributed to individual wall elements.
- Wall layout in the floor plan, crucial for identifying the storey mass and stiffness centre. Asymmetric distribution can lead to increased torsional effects and larger displacements of elements under the same forces.
- Individual walls' predicted shear response in terms of the shear force–horizontal displacement diagram of wall tops. This is a critical assumption that serves to predict wall behaviour under seismic loading by identifying where plastic deformations in the walls will occur and how individual wall elements will carry seismic loads.
- Vertical load on individual structural elements, used to define the seismic loads (determined by the weight of the structure) and analyse the potential overloading of wall elements, also commonly impacting the individual’s walls shear capacity.

In our analyses, the shear load-bearing capacity and deformation capacity were evaluated using distinct approaches. For each individual wall, the input parameters included wall dimensions, vertical load, and the precalculated characteristic points of the assumed bilinear shear force–horizontal displacement diagrams, as described below. The assumed nonlinear behaviour of the wall panels forms the foundation of the presented method and analysis.

2.3.Variations of the nonlinear shear response models for LFTWPs

In the analyses conducted, three variations of assumptions were adopted for the nonlinear shear force–horizontal displacement response of the panels: one based on experimental tests and two models derived from analytical calculations of the wall's shear load-bearing capacity $F_{\mathrm{i,v,Rd}}$ according to Eurocode 5 (EC5). Yield deformation and wall effective stiffness k_{w} for the analytical models were derived from a model in the literature. For all models, the ultimate deformation capacity ($u_{\mathrm{i,frame,u}}$) was evaluated based on the calculated yielding point ($u_{\mathrm{i,frame}}$, $F_{\mathrm{i,v,Rd}}$) and the minimum ductility factor μ of 4, obtained from experimental tests for the assumed wall panel type.

For comparison, the average values of the experimentally obtained and analytically calculated shear capacities, F_v, displacements at the elastic limit, u_{frame}, and shear stiffnesses, k_w, for the 250 cm wide panels used in construction are presented in Table 1.

Table 1. Various assumed bi-linear responses for B12-12 LFTWP of 250 cm length

Bi-linear wall response model	**Sheathing-to-timber frame connection lateral load capacity**	F_v **[kN]**	u_{frame} **[mm]**	k_w **[kN/mm]**	μ_w **[/]**
Method A, Källsner et al	$F_{f,m,EC5}$	22.7	2.8	8.19	4
	$F_{f,m,EXP}$	38.0	4.6	8.19	4
Method B, Källsner et al	$F_{f,m,EC5}$	45.8	5.6	16.4	4
	$F_{f,m,EXP}$	76.7	9.4	16.4	4
Experimental	/	76.0	15.0	5.07	4

Note: A staple spacing of 0.75 mm was used in the presented experimentally tested panels and in the calculations. In Method B, the shear capacity of the panel is calculated taking into account a uniform vertical panel load of 25 kN/m, i.e. the load that the panels were also subjected to in the experimental tests.

Experimental Model: This model utilizes the shear capacity F_v and effective shear stiffness k_w of wall panels determined from experimental cyclic shear tests, presented in Figure 1 [2]. The B12-12 panels tested and used in the building construction and therefore considered in the analysis are briefly described in the following chapter. For wall panels of different lengths, both F_v and k_w are assumed proportional to the 250 cm wall length, as presented in Table 1.

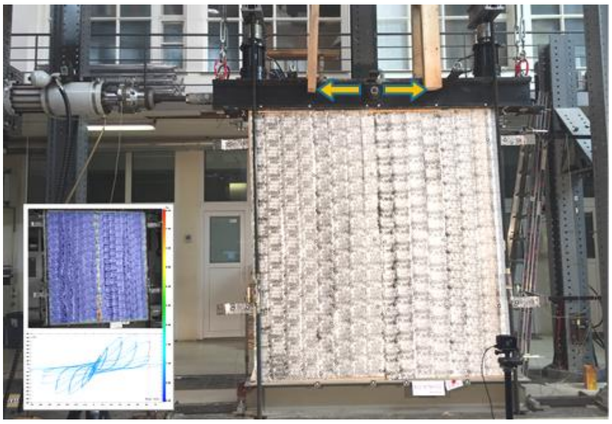


Figure 1: Test setup of experimental tests with some results for LFTWP, used for construction of the analysed timber building [2].

Analytical Models: For the other two variations, the shear capacity of individual wall panels F_v is calculated using either Method A (Eq. (1)) or Method B (Eq. (2)) from EC5. The effective shear stiffness k_w is derived from a displacement model for panels (u_{frame}) proposed by [5] according to Eq. (8), (9).

In Eq. (1)-(9), $F_{f,m}$ represents the lateral capacity of the timber-to-sheathing board connection with mechanical fasteners. The width and height of the sheathing panel are denoted by b and h, respectively. The parameter s refers to the distance between fasteners, while c is a factor that accounts for the panel's length-to-height ratio. The base distance between fasteners, s_0, is defined in Eq. (3) and depends on the fastener diameter (d) and the characteristic density of the timber frame (ρ_k).

Adjustment factors k_d, k_q, k_s, and k_n are used to account for various influences on panel behaviour: k_d is determined by panel dimensions (Eq. (4)), k_q by uniformly distributed vertical load (Eq. (5)), k_s by the arrangement of fasteners (Eq. (6)), and k_n by the contribution of sheathing boards on both sides of the panel (Eq. (7)). The terms $F_{v,Rd,max}$ and $F_{v,Rd,min}$ denote the design shear capacities of the stronger and weaker sides of the panel, respectively.

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.133

$$F_v = \frac{F_{f,m}\, b\, c}{s} \tag{1}$$

$$F_v = \frac{F_{f,m}\, b}{s_0} k_d k_q k_s k_n \tag{2}$$

$$s_0 = \frac{9700\, d}{\rho_k} \tag{3}$$

$$k_d = \frac{b}{h} \text{ for } \frac{b}{h} \leq 1.0 \tag{4}$$

$$k_q = 1 + (0.083q - 0.0008q^2)\left(\frac{2.4}{b}\right)^{0.4} \tag{5}$$

$$k_s = \frac{1}{0.86\frac{s}{s_0} + 0.57} \tag{6}$$

$$k_n = \frac{F_{v,max} + 0.5\, F_{v,min}}{F_{v,max}}, \text{ for panels with sheathing panels on both sides} \tag{7}$$

$$u_{frame} = \frac{4.52\, h\, s_r}{k_{ser} b} \tag{8}$$

$$k_w = \frac{F_v}{u_{frame}} \tag{9}$$

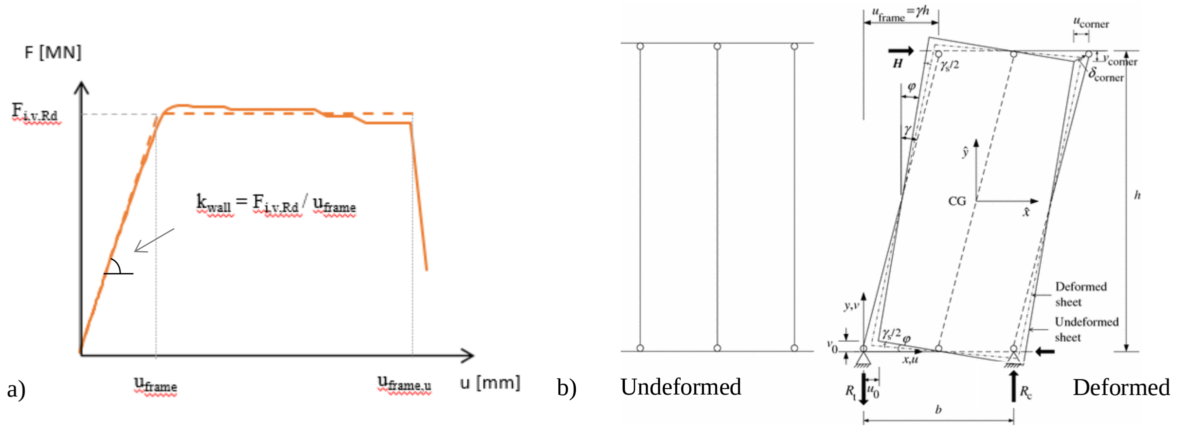


Figure 2: a) Bi-linear shear wall response model; b) model of wall panel deformation according to [5].

For both Method A and Method B, the panels analytical shear capacity F_v is linearly dependent on the lateral load-bearing capacity of the dowel-type connectors $F_{f,m}$ (staples in this case). The parametric building pushover analyses incorporated both the EC5-estimated mean shear capacity of the stapled connection ($F_{f,m,EC5}$ = 0.681 kN, evaluated as 125% of the characteristic value) and the experimentally determined shear capacity ($F_{f,m,EXP}$ = 1.14 kN). Due to the significant difference in the lateral capacities of the sheathing-to-timber connections, the analytically estimated wall shear resistances also differ notably. However, the slip modulus of the connection is consistent between the EC5 calculation and experimental tests. A slip modulus (K_{ser}) of 1.11 kN/mm was assumed for all wall properties in subsequent analytical calculations.

3. Comments and Discussion on the Assumptions of the Nonlinear Static Analysis Methods and the Bi-linear Shear Response of Wall Panels

In buildings with a regular distribution of vertical load-bearing elements across the height, damage does not occur solely at the critical floor. Therefore, the adopted method is most suitable for single-storey buildings and can provide informative conservative results for buildings with one or two floors regarding shear resistance.

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.133

The assumption of a rigid diaphragm and proportional distribution of shear load according to shear stiffness may be questionable in buildings with flexible timber floors or poor wall-to-wall and wall-to-timber connections. However, in modern timber buildings, floor construction should ensure adequate load transfer.

A critical factor in the global seismic response of buildings is the assumed wall shear stiffness, which in our model directly influences the ultimate deformation capacity, as it is defined through ductility in relation to yield displacement. Assuming shear stiffness proportional to wall length in the experimental model results in a numerically stiffer response for panels with a lower length-to-height ratio, potentially underestimating displacements due to rocking (under equal vertical stress and anchoring).

In the analytical calculations, wall stiffness is also assumed to be proportional to panel length. The model considers wall yield deformation dependent on panel height, sheathing-to-timber connection slip modulus, and fastener spacing—all consistent across similarly constructed panels—leaving panel length as the primary variable. This assumption leads to a stiffer response than in actual behaviour, as it neglects displacements from uplift, anchor system shear deformation, and panel shear deformation.

Although future EC models will address panel deformation estimation issues by incorporating geometry, angle bracket characteristics, and vertical load, they currently lack consideration of wall-to-perpendicular wall connections, which could significantly enhance the actual shear stiffness of the building.

Lastly, the assumed ductility in evaluating deformation capacity may limit the accuracy of predicting real panel behaviour under seismic loads, potentially underestimating deformation demands and overestimating safety. While high ductility was experimentally confirmed for the specific panels used, such behaviour is not guaranteed for panels in general and must be verified experimentally before use in analysis. These considerations are essential when interpreting results and comparing them with design standards.

4. Case study building

4.1.Building

To apply the method and investigate the buildings non-linear shear behavior and the influence of panel behaviour assumptions, a real residential building with a regular architectural design in Styria, Austria, was selected as the case study. The building measuring 11.4 m in length and 11.1 m in width, consists of three floors with a floor height of 2.67 m. It features a gable roof with a 22° slope. The first floor differs structurally from the upper floors due to integrated garage spaces. A staircase, measuring 4.4 m in length and 2.4 m in width, is positioned on the left side of the building across all three floors. The total floor area, including the staircase, is 136 m². The ground floor plan, highlighting the arrangement of the light frame timber wall panels, can be seen in Figure 1.

The building’s load-bearing system is composed of light frame timber wall panels, timber floors, and a timber roof structure. For the presented building seismic analyses, already mentioned B12-12 panel type was considered for the construction and analysis. This panels consist of a solid timber frame sheathed on both sides with 12 mm thick cement-particle boards, attached to the timber frame using 1.53 x 11.25 x 45 mm metal staples with a characteristic tensile strength of 800 MPa, designed for effective load transfer between the sheathing and the frame. The panels exhibit ductile failure with plastic deformations of the staple under panel shear loading.

4.2.Seismic load

The building's mass for seismic performance assessment is determined by applying a vertical load that according to Eurocode combines various effects as defined in Eq. (10), where G_{kj} represents the characteristic value of the permanent load j, Q_{ki} denotes the characteristic value of the variable load i, and Ψ_{Ei} is the combination factor for the variable load i. In the non-linear static analysis, the vertical

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia
Copyright © 2025 CroCEE
DOI: https://doi.org/10.5592/CO/3CroCEE.2025.133

load on each floor is uniformly distributed across the walls in both horizontal directions. The calculation of vertical loads includes the self-weight of the wall panels, assumed to be 3.0 kN/m³. For the seismic load combination, the total vertical load is considered as 2.7 kN/m² for the floors above the ground and first floors, and 1.7 kN/m² for the roof. The total weight calculated for the building analysis is 1.12 kN.

$$\sum G_{kj} \text{ "+" } \sum \psi_{Ei} Q_{ki} \tag{10}$$

5. Results of the pushover analysis and comparison of the results

The analysis offers insight in the building deformation and damage mechanisms. In Figure 3, the wall panels lateral displacements in longitudinal (x) seismic loading direction are presented for two characteristic limit states, i.e. at first panel damage $d_{1.cr}$ and maximum shear resistance F_{max}, respectively. Furthermore, Figure 4 present the utilisation of the ultimate deformation capacity of individual walls, i.e. ration of achieved lateral displacements to ultimate available displacement capacity, for limit states at maximum shear resistance F_{max} and ultimate buildings deformation capacity d_u.both again for the critical longitudinal loading direction. All results are presented for analysis case with considered shear wall nonlinear behaviour according to experimental tests.

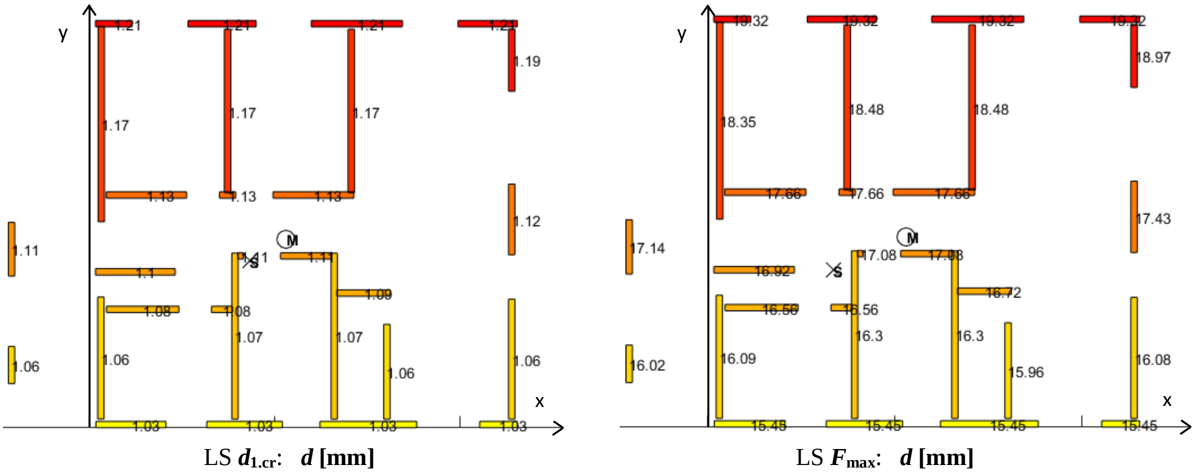
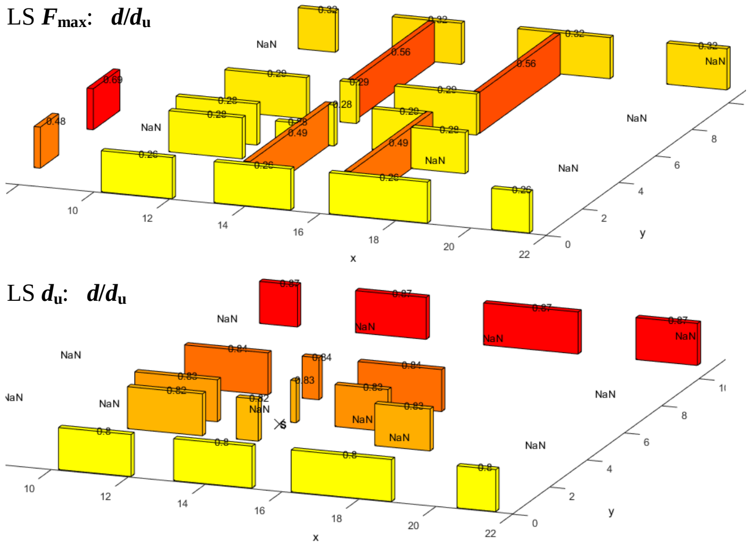


Figure 3: Walls displacements in characteristic limit states of obtained capacity curves in critical loading direction (x) at damage of the 1st panel pa $d_{1.cr}$ and maximum shear load-bearing resistance F_{max}.



Characteristic points of hysteresis envelope and bi-linearly idealised building shear capacity curves:

- $d_{1.cr}$, damage of the 1st panel
- d_e, idealized elastic displacement
- F_{max}, maximum shear resistance
- d_u, building ultimate deformation capacity.

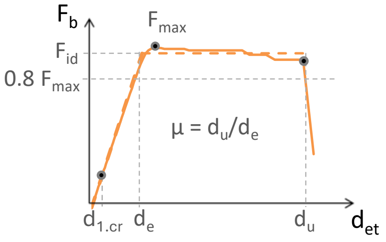


Figure 4: Walls' displacements to ultimate displacements ratio d/d_u in critical loading direction (x) for characteristic limit states at F_{max} and d_u of the building hysteresis envelope capacity curve.

Building global capacity curves, i.e. base shear force in dependence of lateral displacement of the mass centre, obtained for different panels' nonlinear shear behaviour model assumptions, are for the two directions of loading presented in Figure 5. The results clearly confirm the significant influence of the different assumptions and show, that the seismic loading in longitudinal (x) direction results in lower base shear capacity as well as in lower ultimate lateral displacements than the transversal (y) direction.

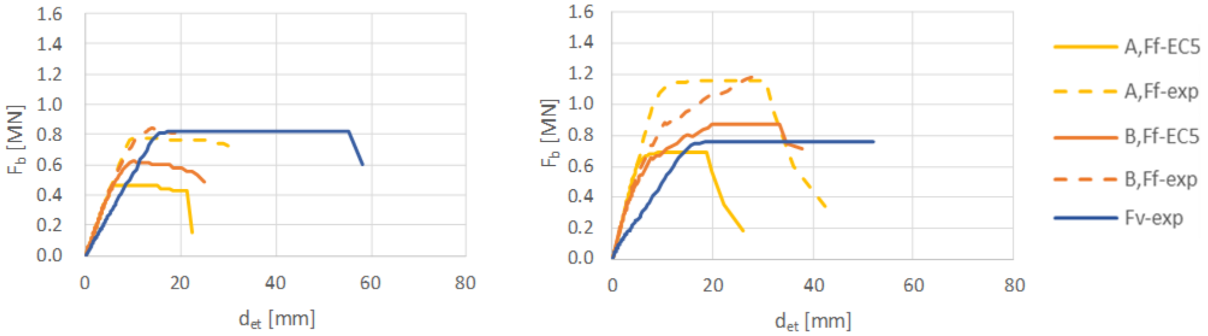


Figure 5: Pushover capacity curves obtained for x (left) and y (right) loading directions in analysis with different panel nonlinear shear behaviour assumptions.

To determine the more critical results in terms of chosen intensity measure, the results are idealised considering equal energy criteria of hysteresis envelope and bi-linearly idealised capacity curves. For all the conducted analyses the results are in terms of characteristic points of bi-linearly idealised curves (presented in Figure 4, right) summarised in Table 2. Furthermore, from the total shear capacity and ductility of the bi-linear capacity curves, the behaviour factors, denoted as $q_{,\text{dej}}$, and the limit ground accelerations $a_{\text{g,u}}$, that the structure can sustain in ultimate limit states, were calculated using the N2 method [6]. Since the assumed shape of the seismic loading is uniform, the participation factor for conversion of a multi-degree-of-freedom system to a single-degree-of-freedom system is equal to 1, and the ratio of the ductility μ to the behaviour factor $q_{,\text{dej}}$ calculated according to Eq. (11), where T^* is the equivalent modal time period calculated from the seismic curves and the value of T_c adopted as specified in the EC8 standard. The values of the q_{dej} factors are given in Table 2, as well as $a_{\text{g,u}}$ for the individual results of the response curves corresponding to the significant damage limit state ("SD"), which should according to the standards achieve the project ground acceleration for 475 year return period earthquake. It is according to EC8 evaluated as 3/4 of ultimate ("NC") displacement d_u.

$$q = (\mu - 1)\frac{T^*}{T_C} + 1, T < T_C \qquad (11)$$

Table 2. Results of analyses - characteristic values of idealised seismic capacity curves

Assumed wall nonlinear shear response	F_{id} [MN]	d_e [cm]	T^* [s]	d_u [cm]	μ	$q_{\text{dej,SD}}$	$a_{g,u}$ [g]	F_{id} [MN]	d_e [cm]	T^* [s]	d_u [cm]	μ	$q_{\text{dej,SD}}$	$a_{g,u}$ [g]
	X direction							Y direction						
Meth. A, $F_{\text{f,m,EC5}}$	0.45	5.09	0.227	21.5	4.23	1.82	0.29	0.68	5.49	0.192	19.9	3.63	1.55	0.38
Meth. A, $F_{\text{f,m,EXP}}$	0.76	8.59	0.227	31.9	3.71	1.67	0.45	1.13	9.17	0.192	32.5	3.54	1.53	0.62
Meth. B, $F_{\text{f,m,EC5}}$	0.58	6.56	0.227	24.4	3.72	1.68	0.34	0.79	6.41	0.192	37.9	5.91	2.10	0.60
Meth. B, $F_{\text{f,m,EXP}}$	0.78	8.79	0.227	32.1	3.65	1.66	0.46	1.06	8.58	0.192	50.7	5.91	2.10	0.80
Experimental	0.81	14.8	0.289	57.4	3.88	1.92	0.55	0.71	9.30	0.244	52.0	5.59	2.30	0.58

Comment: the values of μ refer to d_u (Figure 4) and the "NC" limit state, respectively, and $q_{\text{dej,SD}}$ and $a_{\text{g,u}}$ are calculated for the "SD" limit state according to the N2 method.

Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia
Copyright © 2025 CroCEE
DOI: https://doi.org/10.5592/CO/3CroCEE.2025.133

For the analyzed building, assuming analytical models for panel behavior, the longitudinal (x) direction is found to be critical in terms of shear resistance capacity across all analyses. In contrast, when experimentally obtained panel behavior is considered, the transverse (y) direction becomes critical, although the difference in capacity between the two directions is relatively minor.

Regarding the limiting accelerations the structure can withstand $a_{g,u}$, the x-direction remains the critical direction for all assumed panel behavior models. However, the difference in performance between the two directions is very small under experimental assumptions. This is primarily due to the greater ductility observed in the y-direction compared to the x-direction in the experimental model, as well as the smaller stiffness difference between the two directions compared to analytical assumptions based on Methods A and B or literature-based stiffness models.

Furthermore, the y-direction exhibits notably higher ductility in the experimental model and when using the Method B panel behavior model, whereas this trend is not necessarily observed in the Method A model.

These findings highlight the influence of panel behavior assumptions on the building's seismic response showing how different combinations of stiffness, base shear resistance capacity and deformation capacity yield various behaviour factors and peak ground accelerations that the building can sustain. Furthermore, the calculated behaviour factors are in fact lower than the ones assumed according to the EC8 for the type of panel, since they are for critical direction in all cases lower than 2.0. This presents a challenge for the EC8 guidelines regarding certain other panel types, as the analysis presented here assumes a relatively high ductility value of 4 for the panels.

6. Conclusions

This study presents a simplified and conservative nonlinear static analysis using a modified POR method, applied to a real timber building constructed with light frame timber wall panels type tested in the preceding experimental campaign. The goal was to assess how different assumptions about panel shear behaviour affect the building's nonlinear seismic response. Three modelling approaches were used: one based on experimental shear tests and two analytical models following Eurocode 5 (EC5). These models accounted also for different stapled connections load-bearing capacity assumptions, directly impacting panels' shear capacity.

The results revealed significant differences in the building's seismic resistance depending on the assumed panel behaviour, highlighting the sensitivity of the analysis to shear capacity and stiffness assumptions. This underscores the need for clear and standardized guidelines for assessing the seismic performance of light frame timber buildings, as current standards like Eurocode 8 (EC8) lack more detail information on which assumptions to adopt for such systems in the analysis.

Key research gaps include the method's limited suitability for multi-storey buildings, simplifications in modelling wall shear stiffness, and the assumption of rigid diaphragms, which may not reflect real structural behaviour. Additionally, the assumed ductility may overestimate safety without broader experimental validation. Addressing these gaps through experimental tests, refined modelling and improved design standards is essential for the reliable seismic design of timber structures.

Acknowledgements

The development of the method and numerical research was financially supported by the Slovenian Research and Innovation Agency (ARIS) through Research Core Funding No. P2-0273. The preceding experimental tests and their analyses were funded under Project No. C3330-19-952045 "Raziskovalci-2.1-ZAG-952045" and "Operational Programme TIGR4smart", both co-financed by the Republic of Slovenia, Ministry of Education, Science and Sport, and the European Regional Development Fund through the EU Cohesion Policy 2014-2020. The valuable contribution of Lucija Balant in conducting parametric numerical analyses is sincerely acknowledged and greatly appreciated.

Special thanks are also extended to Jelovica Hiše for motivation and support in the development of this study, generously providing the building design plans.

References

[1] "EN 1998-1:2005: Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings," 2005.
[2] M. Kržan, T. Pazlar, and B. Ber, "Improved seismic response of light-frame-timber panels with cement-particle-board sheathing of various thicknesses and different configurations of fasteners," *Eng. Struct.*, vol. 253, p. 113757, 2022, doi: https://doi.org/10.1016/j.engstruct.2021.113757.
[3] "EN 1995-1-1:2004: Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings," 2004, *CEN*.
[4] M. Tomazevic, "The computer program POR: institute for testing and research in materials and structures-ZRMK," *Ljubl. Slov.*, 1978.
[5] B. Källsner and U. A. Girhammar, "Plastic models for analysis of fully anchored light-frame timber shear walls," *Eng. Struct.*, vol. 31, no. 9, pp. 2171–2181, 2009, doi: https://doi.org/10.1016/j.engstruct.2009.03.023.
[6] P. Fajfar, "A Nonlinear Analysis Method for Performance-Based Seismic Design," *Earthq. Spectra*, vol. 16, no. 3, pp. 573–592, 2000, doi: 10.1193/1.1586128.