Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2023) 3228-3245

ECCOMAS Proceedia

COMPDYN 2023
9th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Athens, Greece, 12-14 June 2023

LIGHT TIMBER FRAME WALLS WITH CLADDING – NUMERICAL INVESTIGATION OF THE SEISMIC PERFORMANCE OF A MULTI-STORY BUILDING BASED ON TEST RESULTS

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Abstract

Light Timber Frame Walls (LTFWs) are increasingly used to provide lateral stiffness and resistance to multi-story timber buildings. They consist of several components (frame, sheathing, fasteners and anchorage) and act as shear diaphragms for the global stability of the structure. Additional cladding, e.g. gypsum fiberboard (GFB), is common practice for fire protection in multi-story buildings. Yet, according to Eurocode 5, this cladding is not considered in the structural analyses. Three different LTFW configurations were tested under monotonic and cyclic loading: (i) typical LTFWs sheathed with OSB sheathing on each side; (ii) enhanced LTFWs with three OSB sheathings (two external and one internal); (iii) typical LTFWs with OSB sheathing covered by GFB cladding. Aim of the tests was to determine the – stiffness, resistance and ductility – of typical LTFWs, to examine the improved seismic behavior of the enhanced LTFWs and identify the influence of the fire protection cladding. The response of the investigated walls (i.e. initial stiffness, hysteretic behavior) was implemented in a structural model (SAP2000) using multi-linear plastic link elements. Nonlinear static and dynamic analyses were performed on a set of multi-story buildings containing the three LTFW typologies. The paper presents the following aspects: (i) overall experimental results of the tested LTFWs; (ii) numerical model of the 3D structure and modelling parameters of the link elements; (iii) outcomes of the nonlinear static/dynamic analyses; (iv) comparison of results and main conclusions. The seismic performance of the multi-story building is outlined for each LTFW typology, the strengths and weaknesses of each wall configuration are discussed.

Keywords: Light Timber Frame Walls; Nonlinear Static/Dynamic Analyses; Performance Based Design.

ISSN:2623-3347 © 2023 The Authors. Published by Eccomas Proceedia.
Peer-review under responsibility of the organizing committee of COMPDYN 2023.
doi: 10.7712/120123.10635.20758

1 INTRODUCTION

Light timber frame walls (LTFW) are commonly used to provide lateral stiffness in low and mid-rise buildings. Given the increasing interest for timber multi-story buildings, the demands for timber elements capable to provide lateral stiffness grows as well.

LTFWs are built-up members consisting of the frame, the sheathing(s) and the fasteners. Each component with its corresponding material properties contributes to the total load-bearing capacity of the walls. The frame usually consists of structural timber, while for sheathing oriented strand boards (OSB) are usually employed. Fasteners are typically nails, staples or – less common – screws. Due to the ductile behavior of the mechanical fasteners, LTFWs can be utilized in buildings for energy dissipation in seismic events. Design provisions provided by *Eurocode 5* [1] and *Eurocode 8* [2] are applied to ensure this desired fastener failure mode and exclude brittle ones. Within a dissipative design concept, the nonlinear behavior of the walls must be considered in the structural design process.

Eurocode 8 allows design against seismic actions either using force-based methods (equivalent static method, response spectrum analysis), in which the nonlinear response of the structural members can be considered only indirect through the behavior factor, or implementing more sophisticated methods, in which the nonlinear response of particular structural elements is directly simulated. Non-linear static analysis (pushover) and nonlinear dynamic analysis (response history analysis) can assess this possible nonlinear response of a structure under seismic actions. For a safe nonlinear design, the lateral load-bearing parameters (i.e. stiffness, load-bearing capacity and ductility) of all structural elements must be adequately estimated. These parameters can be derived from tests.

Experimental investigations at RWTH Aachen and other research projects demonstrated that the calculation of the load-bearing capacity and stiffness of LTFWs by the provisions given in *Eurocode 5* [1] are underestimated for OSB sheathing [3][4].

In Europe, it is common practice for timber multi-story buildings to clad the walls with fire protective boards, e.g. gypsum fiber boards (GFB). According to *Eurocode 5* [1], the stiffness and resistance of the cladding is not taken into consideration for the structural design. As a result, the initial stiffness and load-bearing capacity of the cladded LTFWs are even more undervalued by the analytical design model of *Eurocode 5* [1]. This was proven by recent experimental tests conducted at RWTH Aachen, whose results are present below.

The outcomes of the tests are of great importance, mainly regarding the seismic design of LTFWs. Their initial stiffness influences the modal properties of the structure and consequently the spectral acceleration. Moreover, in case timber walls are considered as dissipative elements, their resistance governs the design of non-dissipative elements (i.e. in case of the capacity-based design). A possible underestimation of resistance could lead to premature, undesired, brittle failure of non-dissipative elements (i.e. anchoring).

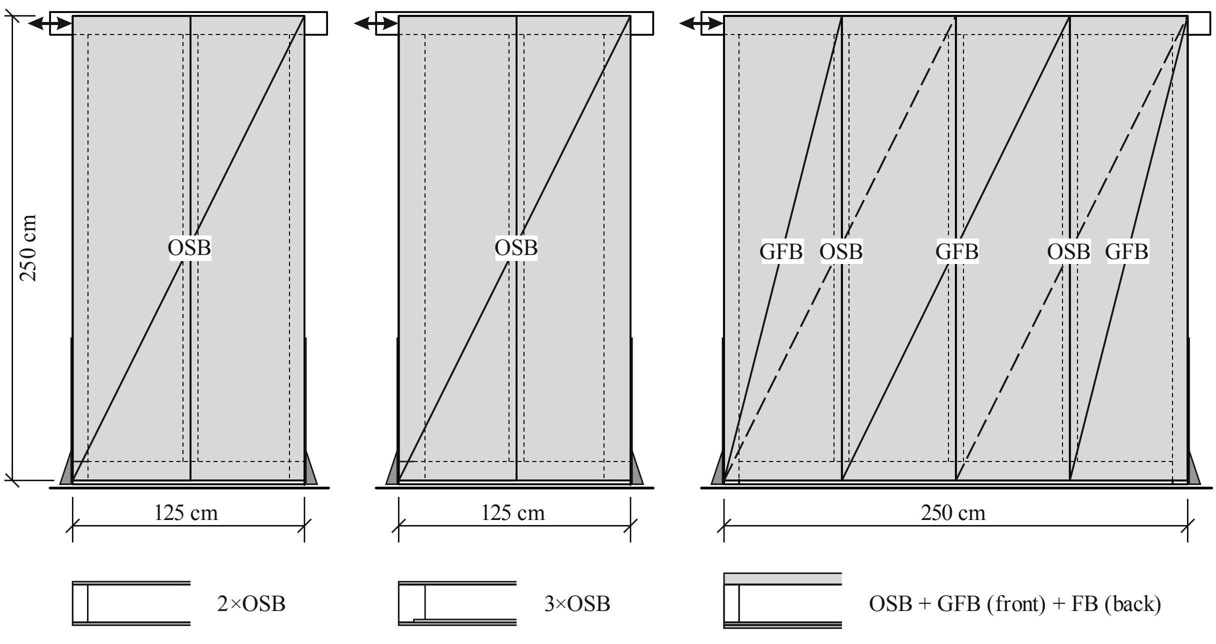
In the current paper, the seismic performance of conventional LTFWs, enhanced LTFWs and cladded LTFWs is outlined. First of all, the results of the experimental tests on real-scale LTFWs are briefly summarized. Furthermore, a mid-rise case study building – stabilized in both directions by the tested LTFWs – is numerically investigated (i.e. through non-linear static and dynamic analyses) – with the aim to quantify the response to lateral forces and to evaluate the seismic performance. Initially, the influence of the cladding was neglected, as it is done in practice. Afterwards, the contribution of the GFB-cladding was simulated. The outcomes of numerical investigations – on both case studies (without and with GFB-cladding) – are compared and discussed. The main conclusions regarding the performance of the investigated LTFWs, the seismic performance of the case study buildings, and the influence of GFB-cladding are finally presented.

2 EXPERIMENTAL INVESTIGATIONS OF LTFWs

Within a recent research project (*HOLZBEBEN*), quasi-static monotonic and cyclic experimental investigations on real-scale LTFW's with different configurations were carried out at the *Centre of Wind and Earthquake Engineering / RWTH Aachen University*. The experimental campaign included tests on typical LTFWs (labeled as "TW"), enhanced walls (labeled as "EW") [3][5] and anchorage elements [6]. Tests on walls with additional fire-protection cladding (labeled as "CW") were conducted in the aftermath of the project and their results were the inspiration for the research project *HELEPOLIS* [7]. The test results are outlined briefly below.

The structure of the LTFW test specimens is shown in Figure 1. All specimens used structural timber C24 for the frame, resin coated staples (applied considering a spacing of 75 mm) as fasteners and two HTT31 (Simpson Strong-Tie) anchorage elements at the supports of each wall. The TW test specimen (see Figure 1a) had OSB sheathing (t=15 mm) on both sides. The EW test specimen (see Figure 1b) used OSB sheathing on both sides. Furthermore, one additional sheathing layer was placed in the interspace of the wall by recessing the studs on one side.

The CW test specimen (see Figure 1c) represented a common exterior wall. It was sheathed with OSB (t=15 mm) on one side. The sheathing was cladded by gypsum fiber boards (t=12.5 mm) with an offset of half the sheathing length – typical for fire protection. The staples were driven into the frame trough the sheathing – "doubling" shear joints. The results of both TW and CW are comparable, as the resistance is proportional to wall length and number of sheathings. The influence of the wood fiber board (FB) positioned on the back side can be neglected based on the outcomes within [7].



(a) Typical Wall (TW) (b) Enhanced Wall (EW) (c) Cladded Wall (CW)

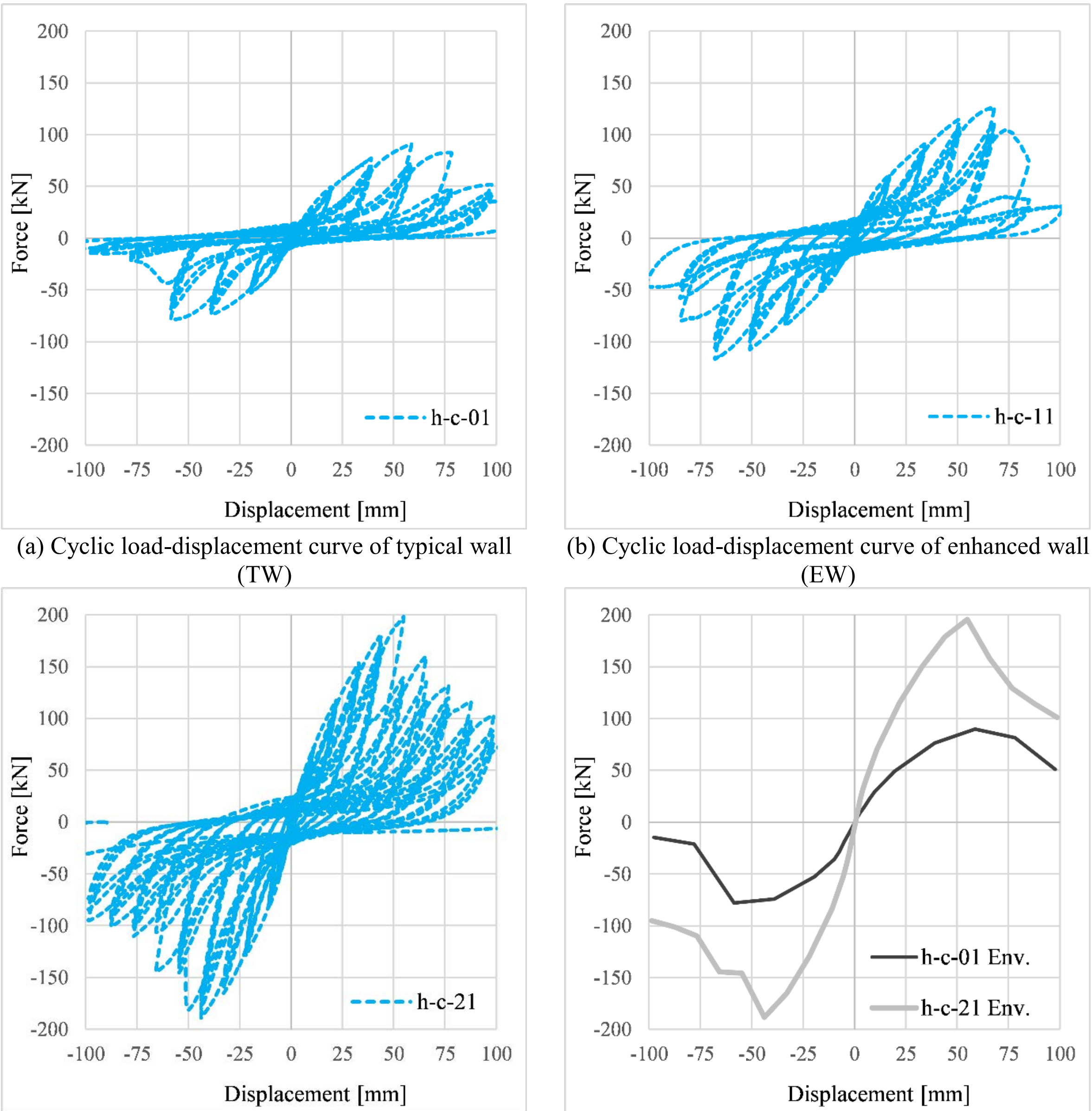
Figure 1: Structure of test specimens

The experimental program and the main properties of the specimens are summarized in Table 1. All mentioned specimens were tested without vertical loading. High performance anchorages were used to prevent a possible failure there and to allow the realization of a "shear mechanism" according to *Dujic* [8]. Horizontal cyclic loading was introduced according to *ISO*

21581 [9]. The results of the tests in terms of force-displacement curves are presented in Figure 2. As it can be observed, the additional sheathing layer of the EW leads to about 38% higher resistance. Moreover, the influence of cladding can be distinguished from the comparison of the envelope curves corresponding to TW and CW (see Figure 2d).

Name	L × h [m]	Sheathing t [mm]	Fasteners ⌀-length-spacing [mm]	Loading
h-c-01 (TW)	1.25 × 2.50	2 × OSB 15	Staples 1.80 - 65 - 75	Cyclic
h-c-11 (EW)	1.25 × 2.50	3 × OSB 15	Staples 1.80 - 65 - 75	Cyclic
h-c-21 (CW)	2.50 × 2.50	1 × OSB 15 (front) 1 × GFB 12.5 (on OSB) 1 × FB 60 (back)	Staples 1.80 - 65 - 75 Staples 1.80 - 75 - 75 Staples 2.00 - 80 - 95	Cyclic

Table 1: Test program of LTFWs with cyclic lateral loading according to *ISO 21581* [9]



(a) Cyclic load-displacement curve of typical wall (TW)

(b) Cyclic load-displacement curve of enhanced wall (EW)

(c) Cyclic load-displacement curve of cladded wall (CW)

(d) Comparison of TW and CW "force-displacement" envelope curves

Figure 2: Experimental results of the different LTFW specimens

3 SEISMIC PERFORMANCE OF A MULTI-STORY BUILDING WITH LTFWs

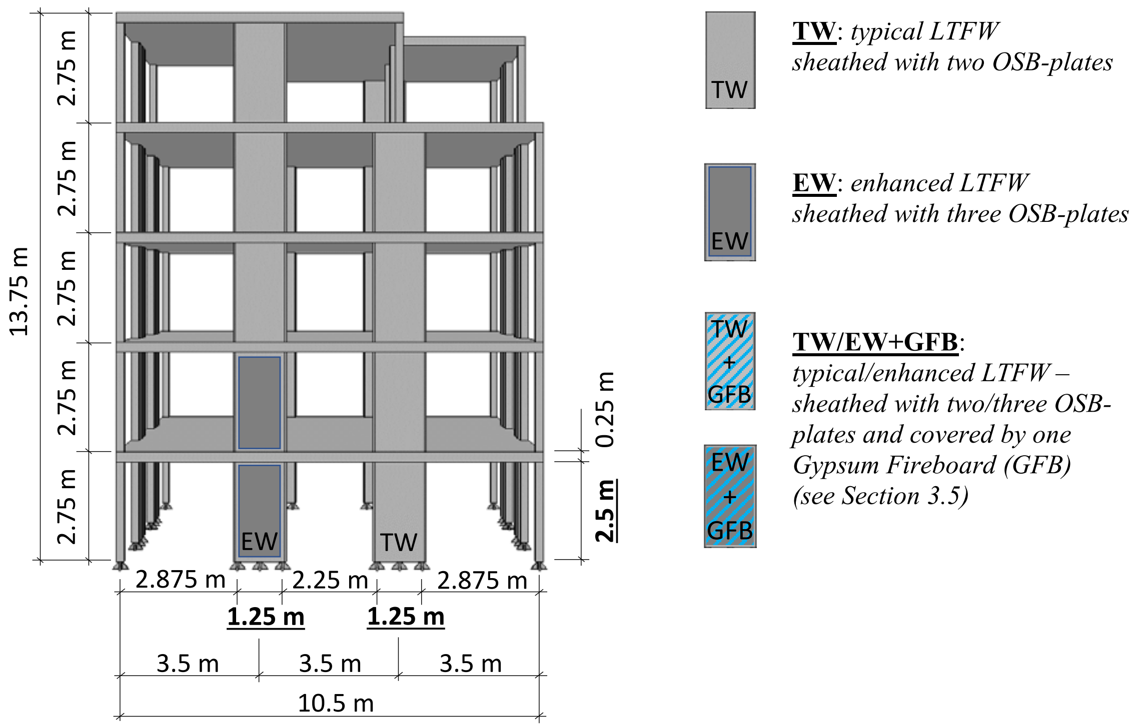
The aim of the current study was to investigate the seismic performance of a multi-story building with light timber frame walls (LTFW) in the following two configurations: (i) typical / enhanced LTFWs (see Figure 3); (ii) typical / enhanced LTFWs covered by one gypsum fireboard (GFB) (see Section 3.5). Furthermore, one of the main objectives was to compare the response of the two multi-story building structures and to show the influence of the gypsum fireboard with regard to the response and the seismic performance.

The current chapter is divided into the following main topics: (i) definition of the case study buildings (including a brief description of the structural configuration, geometry, loading, modeling procedure and simplifications); (ii) outcomes of the nonlinear static analysis performed on the multi-story building with typical / enhanced LTFWs; (iii) outcomes of the nonlinear dynamic analysis performed on the multi-story building with typical / enhanced LTFWs; (iv) comparison of nonlinear static and dynamic results; (v) influence of the gypsum fireboard (GFB) on the "nonlinear static and dynamic response" of the multi-story building with LTFWs.

3.1 General data (definition of the case study buildings)

3.1.1. Structural configuration / geometry / loading / seismic action

In order to illustrate the potential of LTFWs for the use as part of a lateral load resisting structure, as well as for the seismic performance evaluation – a 5-story building was developed and analyzed. The overall dimensions of the 3D building are shown in Figure 3a, while the types of LTFWs that were considered within the current study are presented in Figure 3b.



(a) (b)

Figure 3: 5-floor case study building with perimeter lateral load resisting timber structure: (a) overall dimensions and position of the walls; (b) types of light timber frame walls

As it can be observed in Figure 3a, the total height of the building is 13.75 m and the width is 10.5 m on both horizontal directions. Each of the five floors has 2.75 m height, out of which

0.25 m represents the thickness of the timber deck and 2.5 m is the available floor-to-ceiling height. With regard to the floor layout, it can be observed that the first four floors are identical, while the fifth floor has terrace space (i.e. 20% of the floor area) corresponding to two opposite corners. From the structural point of view, the loads are transferred from the various floors to the foundation as follows:

- The gravity loads (i.e. permanent and variable loads) are transferred from the "*timber floor systems*" to the timber beams (i.e. internal and on the perimeter), then to the timber columns (i.e. internal and on the perimeter), and further to the foundations;
- The lateral loads (i.e. seismic and wind loads) are transferred from the "*timber floor systems*" to the perimeter lateral load resisting structures and further to the foundations.

The perimeter lateral load resisting structures consist of: (i) timber beams which were considered to be continuous over a 5.25 m length (i.e. the connection between two beams is performed in the mid span); (ii) timber columns which were considered to be continuous just over a 2.5 m length (i.e. equal to the floor to ceiling height); (iii) LTFWs, which consist of timber studs and rails which form a "frame" and two or three layers of OSB-plates (leading to TW or EW – see Figure 1 and Figure 3b). Considering the floor layout and the perimeter lateral load resisting structure, it needs to be highlighted that the structural configuration is symmetric on the two orthogonal directions. Furthermore, as it can be observed in Figure 3a, the perimeter lateral load resisting structure consists of two rows of LTFWs. Aiming for an identical arrangement of the windows (i.e. within the two external spans 2.5 m × 2.875 m, as well as in the middle span 2.5 m × 2.25 m) over the height of the building, and for an optimal transfer of the lateral loads (e.g. avoiding a soft story mechanism) – the type and number of LTFWs was customized for each floor. The main components corresponding to each perimeter lateral load resisting system are summarized in Table 2. In addition, Table 3 and Table 4 show the adopted - permanent, variable and seismic - loads.

Floor	LTFW - Row I	LTFW - Row II	Timber columns (h×b)	Timber beams (h×b)
5th	TW	-	200×200 mm	250×400 mm
4th	TW	½ × TW	200×200 mm	250×400 mm
3rd	TW	TW	200×200 mm	250×400 mm
2nd	EW	TW	200×200 mm	250×400 mm
1st	EW	TW	200×200 mm	250×400 mm

Table 2: Main components of the perimeter lateral load resisting system

Design loads		Description	Load value
Roof	Permanent	Self-weight of: timber deck, isolation, etc.	2 kN/m^2
	Variable	Snow; personnel working on the roof, etc.	1 kN/m^2
Floors 1÷5	Permanent	Self-weight of: timber deck, finishing etc.	3 kN/m^2
		Perimeter walls	2.5 kN/m
	Variable	Offices (Class B)	3 kN/m^2

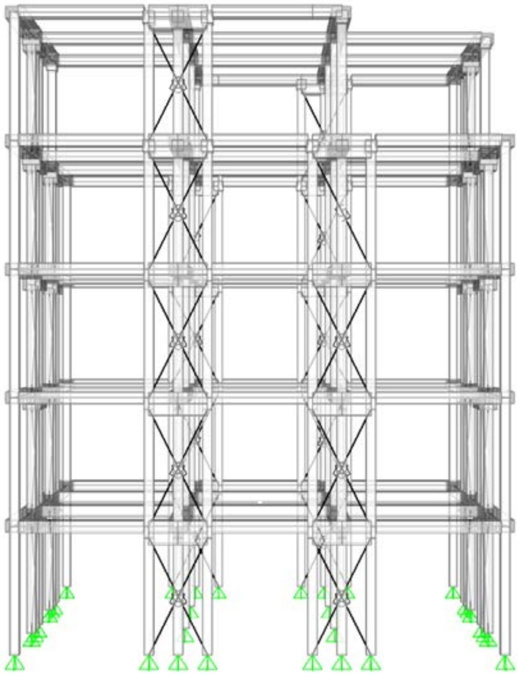
Table 3: Permanent and variable loads

Seismic design action	Value / Description
Peak ground acceleration	$a_{gR} = 0.24 \times g$
Importance factor	$\gamma_I = 1.0$ (Class II)
Ground type	C-R ($S = 1.15$, $T_B = 0.1$ s, $T_C = 0.3$ s, $T_D = 2.0$ s)

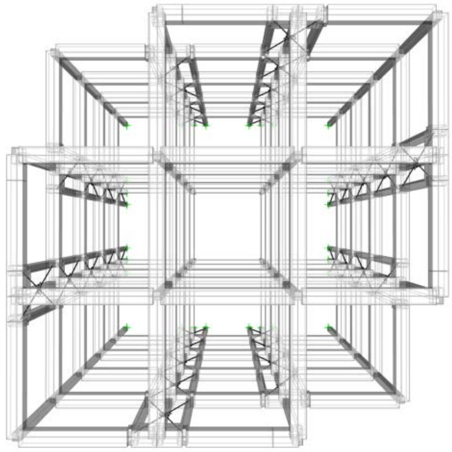
Table 4: Seismic design action according to DIN EN 1998/NA [10]

3.1.2. Modeling procedure and simplifications

The numerical modeling and analysis of the two case study buildings was performed using the SAP2000 [12] software. Consequently, an illustration of the 3D structural model (with extruded elements) – is shown in Figure 4.



(a)



(b)

Figure 4: 3D structural model of the case study building: (a) side view; (b) top view

As can be observed in Figure 4, the timber beams and columns were modeled as beam elements considering the cross-sections described in Table 2. Even though the timber deck was not explicitly modeled, its weight was included as part of the permanent load (see Table 3), while its stiffening effect was accounted for through a diaphragm constraint, which was defined at each floor. All column bases were modeled considering pinned connections to the foundation. Furthermore, the connection between the columns (i.e. from one story to the other) was considered to be pinned as well based on the technical solutions that are current used in the practice. The main simplifications that were adopted in the numerical model are as follows:

- The anchorage devices (e.g. those used for the connection between columns from different floors, respectively between columns and foundation) have sufficient capacity;
- The timber decks have sufficient stiffness for working as rigid diaphragms, and consequently for transmitting the seismic forces to the perimeter lateral load resisting system;
- Torsional effects are not considered;
- While the case study buildings are modeled as 3D structures, the nonlinear static and dynamic analyses are performed only on the X-orthogonal direction (plane frame analysis);
- The investigated structures are characterized by a 5% damping coefficient, as within EN 1998-1 [2];
- The LTFWs transfer the lateral forces (i.e. from one floor to another) - through their capacity to resist in-plane shear forces.

Even though the connection between two timber columns cannot be realized as full-rigid, and the actual stiffness of the connection should be included within the numerical model – the first simplification was considered to be conservative. Indeed, by introducing in the structural model – the actual stiffness of the connection between timber columns, the fundamental period of the structure would increase and the seismic demand would decrease.

With regard to the LTFWs that are part of the perimeter lateral load resisting systems, as it can be observed in Figure 4 – these were modeled through multi-linear plastic link elements – as within [11]. The input (in terms of force-deformation curve) for the different LTFWs was defined based on the cyclic experimental investigations described in Section 2. In particular, an envelope curve was constructed corresponding to the first, second and third force-deformation cycle (see Figure 5a for TW and Figure 5c for EW). Additionally, an average envelope curve was computed corresponding to each loading direction (i.e. positive and negative – see Figure 5b for TW and Figure 5d for EW). From the average positive / negative envelope curve, the least favorable (i.e. with lower capacity / ductility) was used to define the input for the multi-linear plastic link elements.

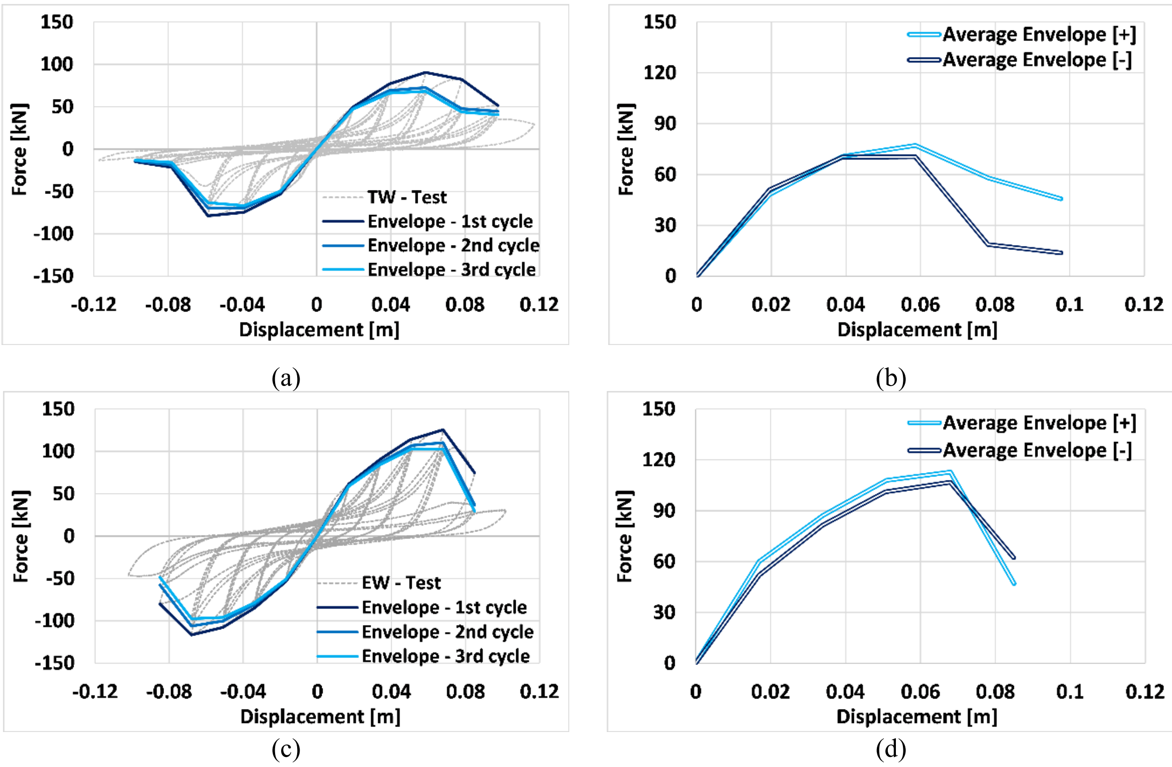


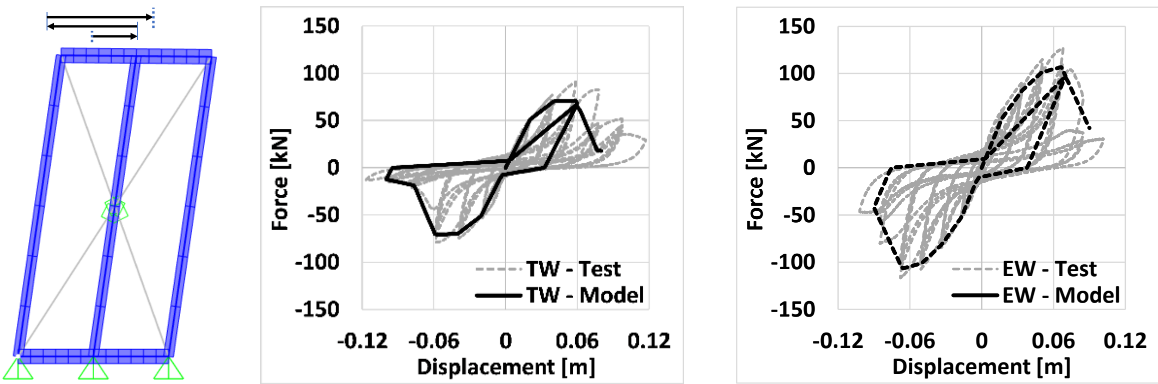
Figure 5: Envelope curves for the 1st, 2nd and 3rd "force-deformation" cycle and average envelope curves corresponding to: (a)-(b) typical wall (TW); (c)-(d) enhanced wall (EW)

A summary of the input for the multi-linear plastic link elements corresponding to the TW and EW is presented in Table 5. In particular the hysteretic behavior of the link elements was defined by a force-displacement curve and by a set of parameters for the "pivot" hysteresis. It is to be noted that the force-displacement curves within Table 5 characterize one link element, which means that the deformation and force correspond to the direction of the link (i.e. the diagonal of the LTFW panel).

In order to validate the input for the multi-linear plastic link elements summarized in Table 5, a simplified numerical model was developed in SAP2000 [12] for both typical and enhanced LTFWs (see Figure 6a). Two link elements were used for each LTFW with the input in Table 5. Pinned connections were used at the base of the panel as well as between timber studs and rails. Furthermore, a "pushover" analysis was performed considering the deformation pattern illustrated in Figure 6a, i.e.: (i) displacement to the right (approx. +0.06 m); (ii) displacement to the left (approx. -0.16 m); (iii) displacement to the right (approx. +0.18 m). The outcomes (in terms of force-displacement curves) of these numerical investigations were compared to the experimental response of the LTFWs under cyclic loading (see the comparison in Figure 6b corresponding to the TW, and in Figure 6c corresponding to the EW). As it can be observed, the input for the multi-linear plastic link elements (i.e. force-deformation curve and parameters for the pivot hysteresis) from Table 5 – lead to a similar hysteretic response of the LTFWs as the one obtained experimentally. Some minor differences can be observed in terms of capacity; nevertheless, these originate from the use of an average envelope curve and from the selection of the most detrimental of the "+/-" average envelope curves.

TW - LTFW			EW - LTFW		
Displacement [m]	Force [kN]	Pivot hysteresis parameters	Displacement [m]	Force [kN]	Pivot hysteresis parameters
-0.042	-15.4	α_1=100	-0.037	-69.6	α_1=100
-0.034	-20.9	α_2=100	-0.030	-119.4	α_2=100
-0.026	-78.8	β_1=0.15	-0.022	-113.1	β_1=0.2
-0.017	-78.5	β_2=0.15	-0.015	-91.2	β_2=0.2
-0.009	-57.2	η=0.2	-0.007	-58.3	η=0.2
0	0		0	0	
0.009	57.2		0.007	58.3	
0.017	78.5		0.015	91.2	
0.026	78.8		0.022	113.1	
0.034	20.9		0.030	119.4	
0.042	15.4		0.037	69.6	

Table 5: Input for multi-linear plastic link elements – force-displacement curve & "pivot" hysteresis parameters



(a) (b) (c)

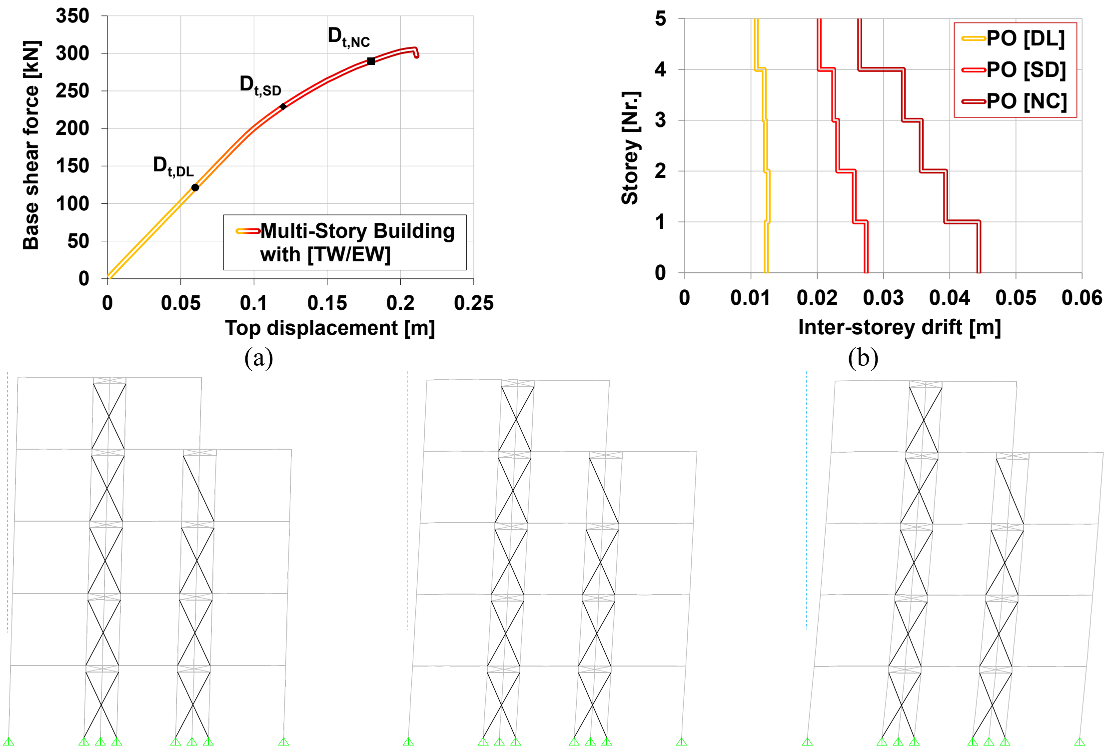
Figure 6: (a) Numerical model of a LTFW with "loading pattern"; (b) response of FEA model for TW; (c) response of FEA model for EW.

3.2 Nonlinear static analysis (Pushover + N2)

The first step for the seismic performance evaluation of the [TW+EW] case study building – was represented by the nonlinear static analysis (pushover - PO) and evaluation of the target displacements with the N2 method [13]. The 3D structural model (see Figure 4 and Section 3.1.2), together with the input for link elements (see Table 5) was used for this purpose. Under a constant gravity load (i.e. from the seismic combination), the structure was laterally loaded (i.e. displacement control at the top floor up to 0.211 m) – considering a modal distribution of the lateral forces. Consequently, the main outcome of the nonlinear static analysis was represented by the capacity curve illustrated in Figure 7a. It is to be noted that the analysis was performed on the 3D structural model but considering only the X-direction. As the structural configuration is symmetric, the response on the Y-direction would be identical. Furthermore, the capacity curve in Figure 7a characterizes the response of two perimeter lateral load resisting structures. The capacity curve and the N2 method [13] allowed to determine the target displacements corresponding to the following limit states (LS): (i) damage limitation (DL); significant damage (SD); near collapse (NC). The obtained target displacements are reported in Table 6 and marked on the capacity curve from Figure 7a. The response of the perimeter lateral load resisting structure corresponding to each of the three limit states is also shown in terms of: (i) inter-story drift (see Figure 7b); (ii) deformed shape (see Figure 7c-e).

Limit States (LS)	Return period	Performance factors	Target displacements
Damage Limitation (DL)	60 years	0.5	$D_{t,DL} = 0.060$ m
Significant Damage (SD)	475 years	1.0	$D_{t,SD} = 0.119$ m
Near Collapse (NC)	1600 years	1.5	$D_{t,NC} = 0.179$ m

Table 6: Limit states, corresponding scaling factors for the seismic input and target displacements for computed for the [TW/EW] case study building



(a) (b)

(c) (d) (e)

Figure 7: Outcomes of the nonlinear static analysis on the case study building with [TW/EW]: (a) capacity curve with marked target displacements; (b) inter-story drifts at DL / SD / NC limit states; (c) / (d) / (e) deformed shape of the perimeter lateral load resisting structure with [TW/EW] at DL / SD / NC limit states

Based on the outcomes of the nonlinear static analysis with N2 method (i.e. capacity curve, target displacements and inter-story drifts at the three limit states), and the comparison with the experimental outcomes (see Figure 5), the following were observed:

- at DL limit state: the response of the structure was in the elastic range – as confirmed by the computed target displacement (see $D_{t,DL}$ on the capacity curve in Figure 7a), and the maximum inter-story drift value (i.e. 0.012 m);

- at SD limit state: the response of the structure was in the nonlinear range – with relatively low but nevertheless plastic deformations occurring at each story except at the 5th floor; the maximum inter-story drift (i.e. 0.027 m) occurred at the ground floor, while at the upper stories the drifts slightly decreased in intensity;
- at NC limit state: the response of the structure was completely in the nonlinear range – with significant plastic deformation and energy dissipation at each floor; the inter-story drifts ranged between 0.026 m at the 5th floor and 0.044 m at the 1st floor; despite the overall significant plastic deformations and a slightly higher inter-story drift at the 1st floor – the partial or total collapse of the structure was not evidenced.

Based on the pushover analysis and the above observations (DL / SD / NC limit states) – the seismic performance of the investigated case study building was found to be adequate.

3.3 Nonlinear dynamic analysis (RHA – response history analysis)

For the nonlinear dynamic analyses, a set of seven artificial accelerograms (see Table 7) were chosen from the ESM ground motion database [14] [15]. A tolerance of 30% was imposed to the ratio between the pseudo-acceleration spectrum of the accelerograms and the target spectrum. Based on the modal analysis, the investigated structure was characterized by a fundamental period of vibration in amount of $T_1 = 1.475$ *sec.*

#A	ESM ID	Date	Earthquake Name	Event ID	Magnitude	Sc. Fact.
1	EU.ULA-HNN.D	1979-04-15	Montenegro	ME-1979-0003	6.9	1.03
2	HI.KAL1-HN2.D	1986-09-13	Southern Greece	GR-1986-0006	5.9	0.80
3	IT.NRC-HGE.D	2016-10-26	Central Italy	EMSC-20161026_77	5.4	1.10
4	IT.CSC-HGN.D	2016-10-30	Central Italy	EMSC-20161030_29	6.5	1.85
5	IT.PCB-HGE.D	2017-01-18	Central Italy	EMSC-20170118_37	5.4	1.95
6	IT.TLN-HGE.D	2016-10-30	Central Italy	EMSC-20161030_29	6.5	2.60
7	TK.1607-HNN.D	2006-10-24	Turkey	TK-2006-0107	4.8	2.60

Table 7: Recorded acceleration time histories used for dynamic analyses [14] [15]

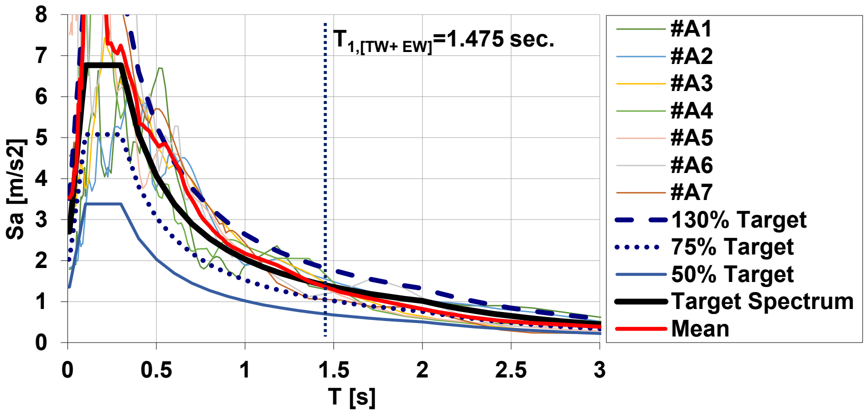
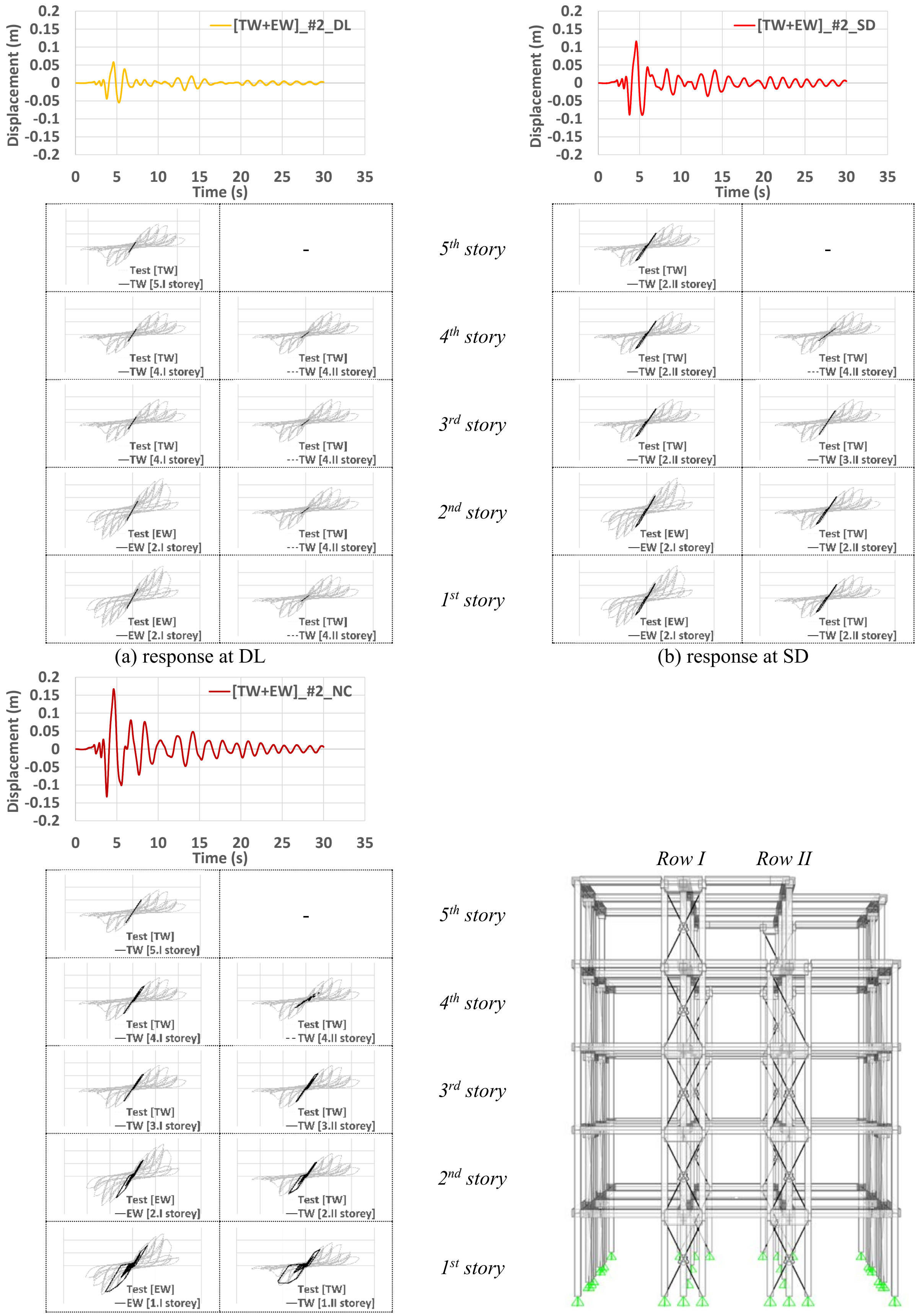


Figure 8: Elastic response spectra of mean and individual accelerograms compared to the target spectrum

The nonlinear dynamic analysis (or response history analysis - RHA) was performed on the X-direction only. In order to evaluate the dynamic response and the seismic performance of the case study building with [TW/EW], the following parameters were monitored corresponding to the three limit states (DL / SD / NC) and the seven accelerograms: (i) displacement at the top floor vs. time; (ii) force-deformation within each LTFW; (iii) inter-story drifts. An overview of the structural response, corresponding to the most demanding accelerogram (i.e. Acc.#2) and to the three limit states, is illustrated in Figure 9.



(a) response at DL

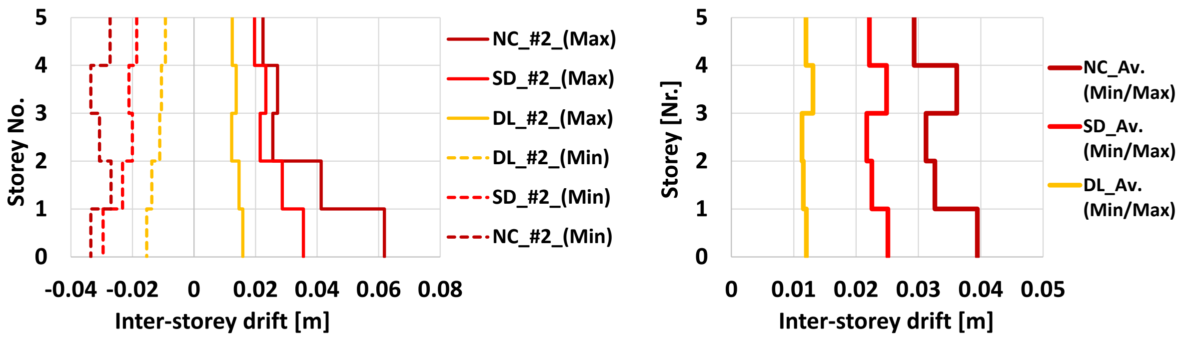
(b) response at SD

(c) response at NC

(d) location of LTFWs: [story].[row]

Figure 9: RHA outcomes (i.e. top displacement vs. time, force-deformation of LTFWs) corresponding to Acc.#2

Additional to the outcomes in Figure 9, the response of the case study building with [TW/EW] is also illustrated in terms of inter-story drifts computed as: (i) min. / max. drifts from Acc.#2 (see Figure 10a); (ii) average of the min. / max. from the 7-selected accelerograms (see Figure 10a). As can be observed, for each of the three limit states – the average inter-story drifts are lower compared to the min. / max drifts resulted from accelerogram #2.



(a) (b)

Figure 10: Inter-story drifts for the case study building with [TW/EW] computed as: (a) min. / max. from Acc.#2; (b) average of the min. / max. from the 7-selected accelerograms.

Based on the outcomes of the nonlinear dynamic analysis (i.e. displacements at the top floor, force-deformation response of the LTFWs, and inter-story drifts – see Figure 9 and Figure 10), and the comparison with the experimental outcomes (see Figure 5), the following were observed:

- at DL limit state: the response of the structure was in the elastic range – as confirmed by the force-deformation curves within TW and EW panels (see Figure 9a), and the maximum of the average inter-story drifts (i.e. 0.013 m – see Figure 10b);
- at SD limit state: the response of the structure was in the nonlinear range – with relatively low but nevertheless plastic deformations occurring at each story including the 5th floor (see Figure 9b); the maximum of the average inter-story drifts (i.e. 0.025 m) occurred at the 1st and 3rd floor, while at the other three stories the drifts were slightly lower – and therefore a relative uniform distribution of deformations was evidenced – as for DL;
- at NC limit state: the response of the structure was completely in the nonlinear range – with significant plastic deformation and energy dissipation at each floor (see Figure 9c and Figure 10b); the average inter-story drifts ranged between 0.029 m at the 5th floor and 0.039 m at the 1st floor (see Figure 10b); despite the overall significant plastic deformations and a slightly higher inter-story drift at the 1st floor (see Figure 9c and Figure 10a corresponding to Acc.#2) – the partial or total collapse of the structure was not evidenced.

Based on the RHA and the above observations (i.e. response at the DL / SD / NC limit states) – the seismic performance of the investigated case study building with LTFWs was found to be adequate.

3.4 Comparison of nonlinear static and dynamic results

For the investigated case study building with LTFWs, a brief comparison was performed – for each of the three limit states – between the nonlinear static and nonlinear dynamic analyses in terms of the following:

- Displacement at the top floor (see Table 8);
- Inter-story drifts (see Figure 11).

As can be observed in Table 8, the outcomes of the nonlinear static analysis are conservative – with values of the displacement at the top floor that are closer to the values from the most demanding accelerogram (Acc. #2). The average values of the displacement at the top floor (i.e. from the RHA), were approximately 13% lower compared to the target displacements obtained with the capacity curve and the N2 method [13].

Regarding inter-story drifts, differences between the nonlinear static and dynamic analyses – were observed in terms of both values and pattern. As can be observed in Figure 11, while for the pushover analysis the highest drifts occur at the lower floors, in case of the RHA a relative uniform distribution of drifts was evidenced – with maximum values corresponding to the 1st and 4th floor.

Limit States (LS)	Pushover	Response History Analysis (RHA)	
	Target displacements (at the top floor)	Maximum top displace-ments (i.e. from Acc. #2)	Average of maximum top displacements
Damage Limitation (DL)	$D_{t,DL} = 0.060$ m	$D_{RHA\#2,DL} = 0.058$ m	$D_{RHA\text{-}avg,DL} = 0.052$ m
Significant Damage (SD)	$D_{t,SD} = 0.119$ m	$D_{RHA\#2,SD} = 0.116$ m	$D_{RHA\text{-}avg,SD} = 0.103$ m
Near Collapse (NC)	$D_{t,NC} = 0.179$ m	$D_{RHA\#2,NC} = 0.167$ m	$D_{RHA\text{-}avg,NC} = 0.145$ m

Table 8: Comparison of the nonlinear static (pushover) and the nonlinear dynamic (RHA) analyses – in terms of displacement at the top floor

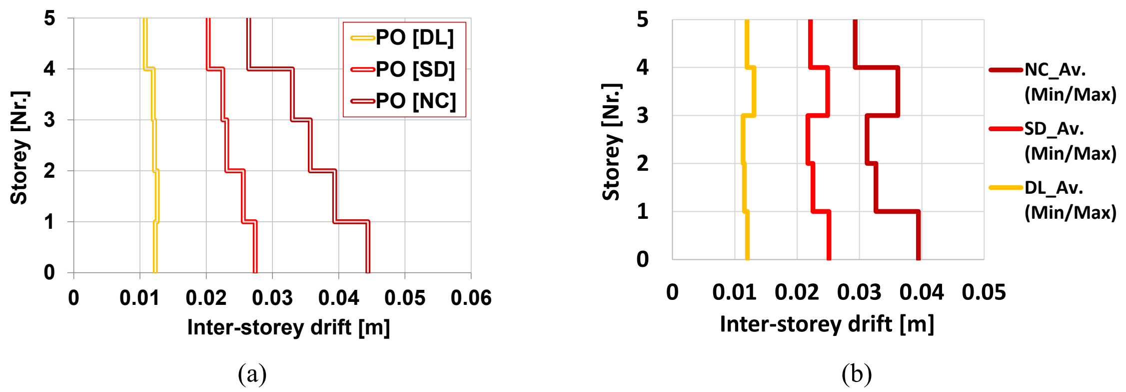


Figure 11: Comparison of the (a) nonlinear static analysis and (b) nonlinear dynamic analysis in terms of inter-story drifts at DL / SD / NC limit states

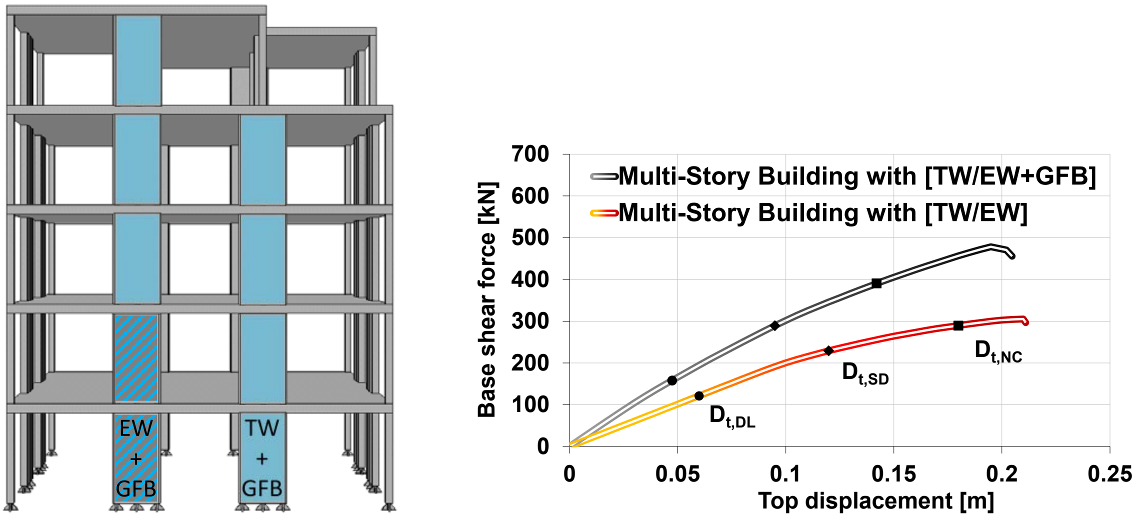
3.5 Influence of GFB on the "nonlinear response" of the multi-story building

One of the main objectives of the current study was to investigate the influence of the gypsum fireboard (GFB) on the nonlinear static and nonlinear dynamic response of the case study building with LTFWs. Based on the experimental outcomes of the LTFWs with GFB (see Section 2), the effect of the gypsum fireboard panels was filtered and inserted within the structural model (i.e. corresponding to each TW and EW – see Figure 12a) as a multi-linear plastic link element. The influence of the GFBs on the structural response, was evaluated in terms of the following:

- Fundamental period of vibration;
- Capacity curve (see Figure 12b);
- Reaction force (i.e. tension force in the connection between LTFWs and foundation – see Table 9);
- Displacement at the top floor corresponding to the DL / SD / NC seismic intensity – as resulted from the nonlinear dynamic analysis using the most demanding accelerogram (i.e. Acc. #2 – see Figure 13).

Based on the outcomes of the modal analysis, the following periods of vibration were obtained: $T_{1,[TW/EW]}$=1.475 seconds; $T_{1,[TW/EW+GFB]}$=1.189 seconds. The use of GFBs lead to a stiffening effect and therefore to a reduction of the fundamental period of vibration.

The influence of the GFBs was observed to be significant also corresponding to the other monitored indicators. In terms of capacity curve and target displacements, as can be observed in Figure 12b, the use of GFBs lead to an increase of the stiffness and capacity, as well as to a reduction of the target displacements.



(a) (b)

Figure 12: (a) Case study building with fire protected LTFWs [TW/EW+GFB]; (b) Influence of the gypsum fireboard (GFB) on the nonlinear static response (i.e. capacity curve).

With regard to the reaction forces corresponding to the LTFWs with GFBs, as summarized in Table 9 – corresponding to the nonlinear static analysis and to the SD limit state – it was observed that reaction forces (e.g. the tension forces within the connections between LTFWs and foundations) increased. For the current case study, the reaction forces increased with more than 26%, which can be explained by the fact that the demand increased with approximately 32% (i.e. the spectral acceleration corresponding to $T_{1,[TW/EW]}$=1.475 sec. was 1.55 m/s^2, while the spectral acceleration corresponding to $T_{1,[TW/EW+GFB]}$=1.189 sec. was 2.046 m/s^2).

LTFWs without cladding		LTFWs with GFB cladding		Increase factor
RF from PO at $D_{t,SD}$=0.119 m		RF from PO at $D_{t,SD}$=0.095 m		$RF_{w/o}/RF_{GFB}$
[TW]	106 kN	[TW+GFB]	154 kN	45%
[EW]	218 kN	[EW+GFB]	275 kN	26%

Table 9: Influence of the gypsum fireboard (GFB) on the reaction forces (RF)

Regarding the influence on the dynamic response, as can be observed in Figure 13, the use of GFBs lead to a decrease of the period of vibration, as well as to a reduction of the displacements at the top floor – corresponding to each of the three limit states. These two observations were also confirmed by the modal analysis (i.e. lower fundamental period of vibration) and the nonlinear static analysis combined with the N2 method (i.e. lower values for the computed target displacements).

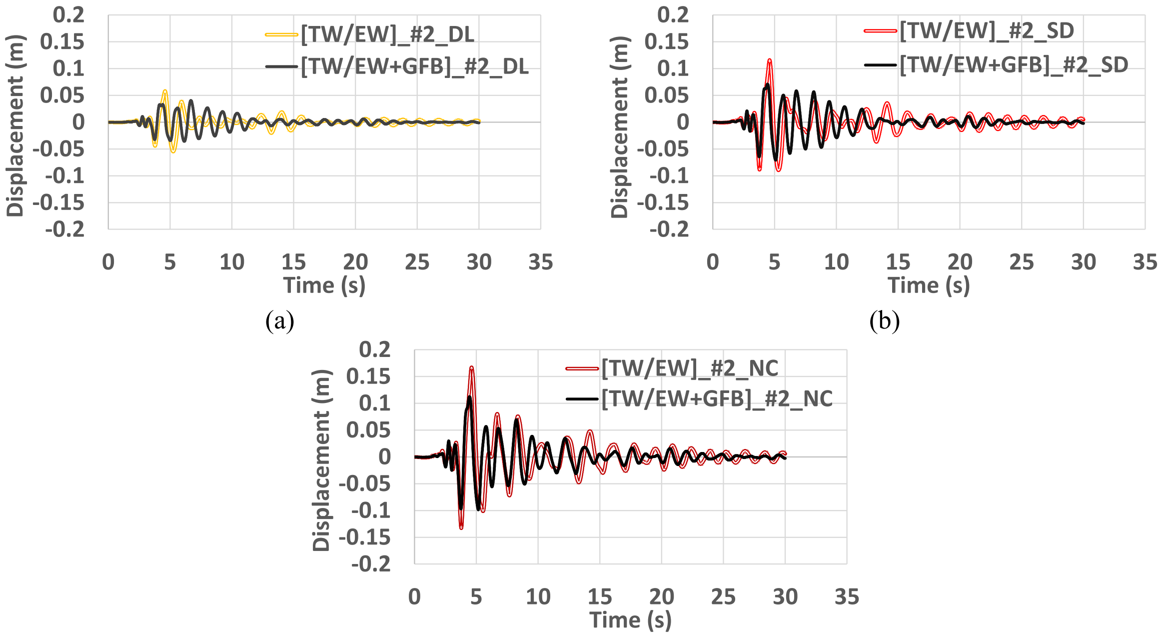


Figure 13: Influence of the gypsum fireboard (GFB) on the nonlinear dynamic response (top displacement vs. time) corresponding to – (a) DL, (b) SD, (c) NC – seismic intensity.

4 CONCLUSIONS

The objective of this contribution was to evaluate the safety of neglecting fire protection cladding in seismic design. For this purpose, the following aspects were presented: (i) overall experimental results of tested light timber frame walls (LTFWs) without and with additional GFB cladding; (ii) numerical model of a 3D reference structure in SAP2000 and modelling parameters of the used link elements representing the LTFWs; (iii) outcomes of the nonlinear static/dynamic analyses for both structure configurations (i.e. without and with cladding).

From the presented experimental investigations on typical walls (TW), enhanced walls (EW) and cladded walls (CW) the following could be concluded: (i) the EW were characterized by an increase of the load-bearing capacity in comparison with the TW (i.e. by 38%); (ii) the CW were characterized by an increase of the load-bearing capacity in comparison with the TW (i.e. by 109%). The initial stiffness was doubled (i.e. increase of 102%). The increase in stiffness and load-bearing capacity is mainly related to the doubling of the shear planes (i.e. number of fasteners) between sheathing and frame, but also, to the additional diaphragm action of the GFB cladding.

Basic “multi-linear plastic” link elements were presented for modeling the hysteretic behavior of LTFWs. These link elements were calibrated and validated with the presented test results for each wall configuration (i.e. TW / EW / CW) on wall level. The calibrated wall models were used as part of the lateral load resisting structure on a 5-storey case study building – modeled and analyzed with SAP2000 [12]. Numerical investigations (i.e. nonlinear static and dynamic analyses) were performed with the aim to evaluate the seismic performance of the case study buildings.

Based on the outcomes of the nonlinear static (pushover) analysis in combination with the N2 method - the seismic performance of the case study building was found to be adequate (i.e. satisfied DL / SD / NC limit states). Furthermore, the seismic performance was confirmed also by the outcomes of the nonlinear dynamic analysis (RHA). In particular, the average values of the displacements at the top floor and the inter-story drifts were slightly lower compared to the outcomes of the nonlinear static analysis. The response at DL limit state was in the elastic range, while corresponding to SD and NC limit states – the structure responded in the nonlinear domain: (i) at SD seismic intensity – only with minor plastic deformations corresponding to the LTFWs; (ii) at NC seismic intensity – with significant plastic deformations mainly corresponding to the 1st and 2nd floors, but without any partial or total collapse of the structure.

For investigating the influence of GFBs, the case study building was adapted by additional multi-linear plastic link elements representing the GFB cladding. The following was observed within the comparison of the structures without and with additional GFB elements – based on the nonlinear static and dynamic analyses:

- The cladding decreased the fundamental period of the case study building to 1.189 seconds compared to the 1.475 seconds corresponding to the building without cladding;
- The response of the structure (i.e. in terms of base shear force) was increased by the GFB cladding;
- The top displacement was decreased by the presence of the GFB cladding;
- The total capacity reserves of the LTFWs was increased by the cladding;
- The decisive reaction forces increased by a minimum of 26%.

The results of the presented investigations are of interest in two respects: (i) economic efficiency and (ii) safe-sided seismic design.

The unused enhancement of the load-bearing behavior by claddings in the structural design is not resource efficient, as the full capacity is not utilized. By adding the effects in the design process, timber frame buildings could become more economic and architectonically sophisticated – as fewer stiffening walls are required.

Capacity design of brittle components (e.g. anchorages) with use of sufficient overstrength factors is key to safe seismic design. This contribution shows, that a neglection of fire protective cladding would lead to a non “safe sided” capacity design of the anchorage for the presented case study building. Considering the fire protection cladding in seismic design is therefore not only recommended, but also classified as relevant for safety.

Further, more elaborate studies should be carried out with regard to the influence of cladding on the seismic performance of multi-story timber buildings. Overstrength factors for cladded LTFWs should be elaborated for practice. An adaption of the design provisions of *Eurocode 5* and *Eurocode 8* with regard to cladding is proposed.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the financial support of the ZIM research project *HOLZBEBEN* by the BMWI and the FNR research project *HELEPOLIS* funded by BMEL. We are also grateful for the cooperation with the project partners Walter • Reif Ingenieurgesellschaft mbH and Adams Holzbau-Fertigbau GmbH.

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