APEM journal
Advances in Production Engineering & Management
Volume 20 | Number 1 | March 2025 | pp 116–130
https://doi.org/10.14743/apem2025.1.531
ISSN 1854-6250
Journal home: apem-journal.org
Original scientific paper

Enhancing racking stiffness in tall timber buildings using double-skin façades: A numerical investigation

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A B S T R A C T

The main goal of the paper is to present possible benefits in application of previously developed innovative in-plane load-bearing timber double-skin façade elements (DSF) as additional bracing elements in tall timber buildings. Therefore, a six-storey prefabricated timber structure of a height of 15 m and with a regular floor-plan is analysed by a seismic excitation of a_g = 0.225·g with a strong asymmetrical position of transparent façade elements. Two structural solutions are analysed: a hybrid system combining CLT and Light Timber-Framed walls and a non-hybrid structure made entirely of CLT. In both cases, DSF elements are first considered non-resisting and later as racking-resisting bracing elements. Numerical results show that using racking-resisting DSF elements in a hybrid system (CLT+LTF) achieves a similar increase in overall racking stiffness as a non-hybrid CLT structure with non-resisting DSF. Previous studies highlight hybrid timber systems as the preferred approach due to structural, energy-efficient, and ecological advantages. This finding is significant, offering practical benefits and new design opportunities for modern tall timber buildings with asymmetrical transparent façades, improving both energy efficiency and interior illumination in contemporary prefabricated structures.

A R T I C L E   I N F O

Keywords:
Timber;
Glass;
Double-skin façades;
Racking resistance;
Mathematical modelling;
Tall timber buildings;
Seismic design

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Article history:
Received 24 July 2024
Revised 4 March 2025
Accepted 17 March 2025

1. Introduction

Due to the well-known environmental problem of harmful emissions of greenhouse gases, the profession is intensively seeking solutions in building design that will be as environmentally friendly as possible, that will produce the lowest possible CO_2 emissions, and that will also provide the highest possible standard of living comfort. Wood as a natural material has by far the best characteristics in environmental terms compared to other construction materials, as it is a CO_2-neutral material [1]. As a result, due to increased urbanisation and the concomitant need for environmentally friendly construction, there is an intense trend towards multi-storey timber buildings (MSTB), particularly in urban environments [2]. There are, of course, many limitations in this respect, particularly in terms of construction, since the modulus of elasticity of the timber elements is relatively low and therefore, particularly in areas of high wind or seismic activity, causes large horizontal displacements of the structure, which in most cases can exceed the values prescribed by the standards [3].

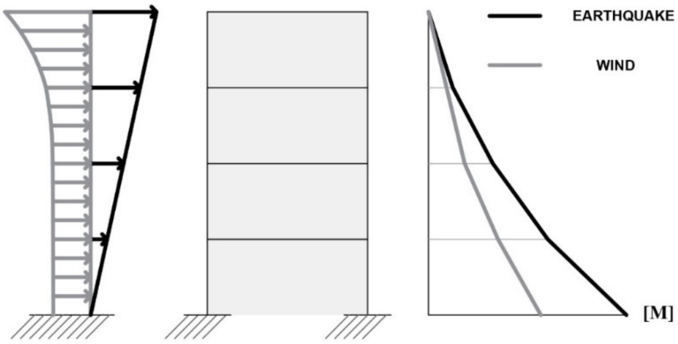


Fig. 1 Display of how wind and earthquake load increases with the height of a building at a certain location [2]

At the same time, to maximise living comfort, the profession has recently accelerated its efforts to use as much transparent glazing as possible, especially around the perimeter of the building. Such glazed surfaces allow for increased natural lighting of the interior living spaces and, on the other hand, also maximise solar heat gain, which can significantly reduce the energy demand for heating buildings during the heating season [4]. Of course, due to the increased solar radiation, such transparent elements are mostly located on the south side of the façade, i.e. they are rather asymmetrically distributed around the building perimeter. However, since such glazed elements are mostly considered in structural analysis as non-load-bearing in their plane to the action of horizontal loads, the asymmetrical plan layout of such glazed elements can results in high torsional loads at the levels of the individual storeys in the case of seismic loads, which are particularly acute in the case of multi-storey buildings, as schematically illustrated in Fig. 1 for the case of the action of two primary horizontal loads (wind and earthquake). The in-plane resistance and stiffness of such transparent elements are also not implemented in any standards yet.

Therefore, in such cases special diagonal bracing systems or other bracing solutions with common internal Light Timber-Framed (LTF) wall elements have to be incorporated into the structure of the building to satisfy all prescribed resisting requirements prescribed by the Eurocodes. However, all such structural solutions are visible and usually also not environmentally friendly and cannot contribute to any improved living comfort or they are sometimes not accepted by the architects at all. In view of the desire to provide a solution that would be at the same time optimal from in a sense of environmental performance and indoor living comfort, but also ensuring satisfactory structural resistance, transparent elements with single-panel glazing fixed to a timber frame were first developed. Such load-bearing timber-glass elements are referred to as single-skin façade (SSF) elements. However, from many experimental [5-9] and numerical studies [10, 11] it was conducted that by using only single-skin timber-glass wall elements, especially the racking stiffness did not increase in the expected manner and was not in the same range as LTF elements with the classical sheathing boards, such as OSB or fibre-plaster boards (FPB). Therefore, in this case only a relatively small additional contribution of such transparent façade elements to the overall racking stability of a whole building was achieved.

Consequently, special double-skin façade (DSF) timber-glass wall elements were further developed, first by a wide experimental study [12] and followed by a specially developed linear-elastic spring Finite Element Model [13] analysing the influence of various parameters which most significantly effect on the racking stiffness of such wall elements. The results of the numerical study were implemented first in the case of a three-storey LTF building [14]. However, in this study the position of the DSF load-bearing elements were limited to the south side of the building only. The results of the study showed a satisfactory contribution to the increased horizontal stiffness of the whole building and also to the reduction of torsional effects, especially in the first storey of the building [14].

However, there is still an important question of the applicability of such load-bearing DSF elements in much taller prefabricated timber buildings, and also with more asymmetrical position of transparent areas around the building envelope. Therefore, in our analysis, a six-storey prefabricated timber building is analysed, where DSF elements are considered as structurally non-load-bearing in the first case and as load-bearing in the second case to judge on the influence of the horizontal load impacts, with the primary purpose of analysing the influence on the reduced distortion of the building and also on the increased overall racking stiffness. Additionally, this is also the first study where the influence of resisting DSF elements is tested on a hybrid timber structural system and not only in one load-bearing system. The selection of a suitable structural system, and the energy efficiency concept strongly namely depend on the specific features of the location, particularly climate conditions, wind exposure and seismic hazard [15-17]. To satisfy in an optimal way simultaneously structural, energy and ecological aspects the choice of a hybrid timber structure seems usually to be the most favoured approach [2, 17].

Respecting this fact, therefore, in our analysis, first a hybrid structural solution (CLT+LTF) will be performed. Due to the possible distortion effects when using non-resisting transparent façade elements and which are mostly summarized on the building envelope, the CLT elements are placed on the envelope of the building, while for internal walls less stiff LTF elements are first used. In the second case the LTF internal wall elements are replaced with the CLT elements to increase the overall racking stiffness of the selected six-storey timber building. The second goal of the performed analysis is further to investigate the influence of the load-bearing DSF elements in relation to the different basic structural systems of prefabricated timber buildings. The aim of our study is to identify potentials in designing tall, prefabricated timber buildings using different structural systems with a strong asymmetrical position of the transparent façade elements around the building envelope. The influence of additional racking resistance of any transparent timber-glass wall elements is currently also not covered with any standards [18]. The obtained results would significantly improve the energy performance of modern timber buildings, as well as the indices of living comfort due to an increased illuminance. Thus, they could open many new perspectives in designing contemporary tall timber buildings which is currently somehow limited because of timber mechanical properties [2, 17].

The content of the paper is systematically organized starting with all necessary presented theoretical backgrounds in Section 2, mathematical modelling of all prefabricated LTF elements used in the study in Section 3, numerical case study on a specially selected prefabricated timber building in Section 4 and with the most important conclusions presented in Section 5.

2. Theoretical backgrounds

2.1 Main structural systems in timber buildings

Timber structural systems differ from each other in the appearance of the structure, and in the approach to planning and designing a particular system. As presented in [15] and [4], structural systems of timber buildings can be classified into six main systems:

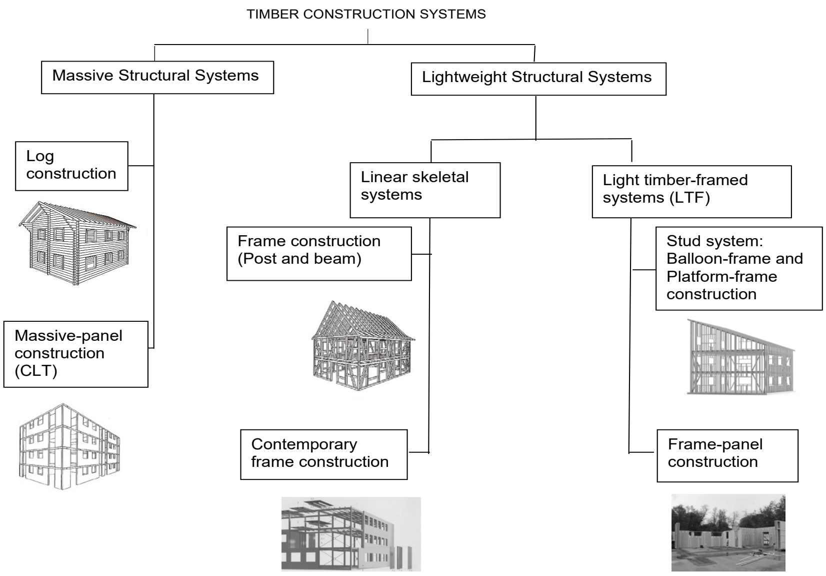


Fig. 2 Main timber structural systems [4]

- Log construction,
- Solid timber construction (CLT),
- Timber-frame construction,
- Contemporary frame construction,
- Light timber-framed construction (LTF); Balloon- and platform-frame construction,
- Light timber-framed construction (LTF); Frame-panel construction.

All systems are schematically presented in Fig. 2. However, it should be pointed out, that only the LTF Frame-panel construction and Solid timber construction as CLT are prefabricated and therefore will be used in our further study for the structural analysis in Section 4. Therefore, only these two structural wall systems are briefly presented in the following subsections.

Massive-panel construction (CLT)

A solid timber structural system is a prefabricated massive panel timber construction where the load-bearing wall and floor elements are produced as cross-laminated planar structural elements. The main benefit of this prefabricated cross-laminated structural (CLT) system is in the perpendicular orientation of timber boards to avoid anisotropy of wood as a raw natural material. The whole production process is schematically presented in Fig. 3.

Another important advantage of CLT system over the LTF system is that its horizontal load-bearing capacity and stiffness are significantly higher, and thus such structural wall elements are mainly placed in lower storeys of a prefabricated timber building, where the internal forces due to horizontal load impacts (wind, earthquake) are the highest (Fig. 1). In the case of hybrid structural systems CLT elements are primarily placed at the envelope of the building, where the asymmetrical floor plan and the resulting distortion on the individual floors results in significantly different loads on the load-bearing wall elements due to the action of horizontal loads. In this case, the additional distortion loads are highest at the envelope of the building and lowest at the load-bearing wall elements closest to the floor shear centre [2, 16].

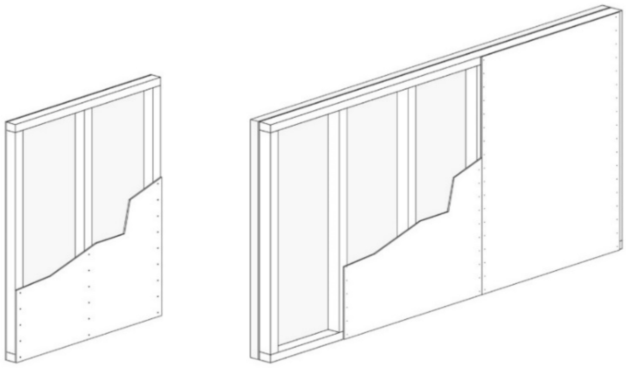
Fig. 3 Production process of CLT structural wall elements [16]

Light timber-framed construction (LTF); frame-panel construction

Light Timber-Framed wall elements are subdivided into two different types of technological prefabrication (Stud system – non-prefabricated and the Frame-Panel system – prefabricated). In our future implementations, we will limit ourselves to prefabricated Frame-Panel system only. The Frame-Panel system originates from the Scandinavian-American construction methods, i.e. balloon-frame and platform-frame construction types (Fig. 2), whose assembly takes place on-site. The advantages of the Frame-Panel construction system over the above-mentioned traditional timber-frame construction systems were first noticed at the beginning of the 1980s and made a significant contribution to the development of such prefabricated timber construction [16].

The load-bearing wall element consists of a timber frame, usually made up of three posts and an upper and lower beam. The upper beam transfers the vertical loads to the lower columns, which in turn transfer the vertical loads to the lower support members. The sheathing boards are attached to the timber frame by means of fasteners (staples, nails) and its tensile diagonal is of the utmost importance to transfer the loads due to the action of horizontal loads (wind, earthquake). Due to the typical dimensions of the prefabricated sheathing boards, the spacing between the columns is usually 600 mm to 625 mm. In practice, however, there are two different possible technological versions of this wall system; Single-panel (Fig. 4a) and Macro-panel system (Fig. 4b). In a statical view the Macro-panel wall assembly is considered as a sum of the contribution of all load-bearing single-panel wall elements [4, 17].

Although CLT and LTF structural systems are quite similar in terms of technology, there are significant differences in terms of construction and building-physical aspects. Thus, the CLT system is more structurally load-bearing to the effects of vertical and horizontal loads, while the LTF system shows better characteristics in terms of better thermal insulation performance for the same thickness of both wall elements. Comparison of these characteristics is widely analysed in [17].



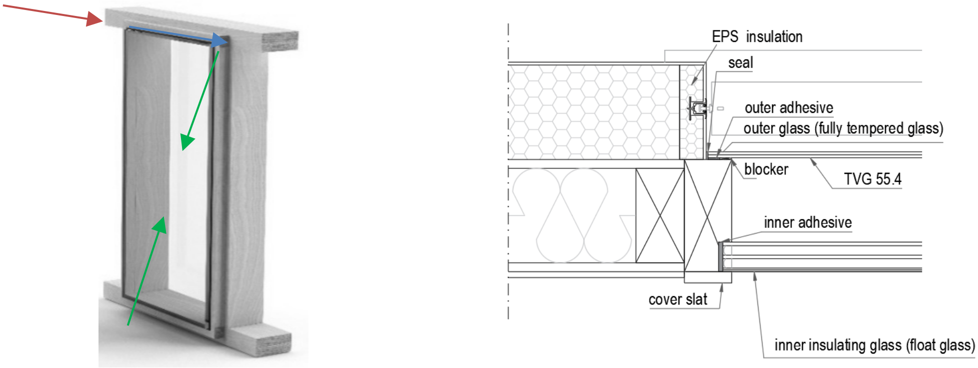
a) b)

Fig. 4 a) Single-panel and b) Macro-panel prefabricated LTF wall element [16]

2.2 Load-bearing timber double-skin façade (DSF) elements

As mentioned in the introductory chapter, there is a strong tendency in contemporary timber construction to incorporate an increased proportion of glazed surfaces, both glass openings (windows, doors) and fixed glazing, which can allow increased solar radiation and better natural lighting through their transparent surfaces [4]. Usually, the asymmetric floor-plan position of such transparent facade elements and the resulting distortion on the individual floors of the building, particularly due to the action of seismic loads, has shown an increasing need to develop appropriately load-bearing timber-glass LTF wall elements that can significantly reduce these torsional effects. Thus, the so-called single-skin façade elements (SSF) were first developed as load-bearing elements. In such timber-glass wall elements a classical sheathing board (OSB or FPB) in LTF wall element presented in subsection 2.1.2. is replaced with a single glass pane which is rigidly bonded to the timber frame (Fig. 5a). The load transformation mechanism thus include a shear transmission over the glass-timber frame bonding line and the resistance in the tensile diagonal of the glass pane, as it is schematically presented in Fig. 5a. However, during many experiemental and numerical studies it was demonstrated that such elements do not prove sufficiently increased racking load-bearing capacity and in particular, do not demonstrate an importance increase in racking stiffness to improve the horizontal stiffness of the whole building [5-11]. Consequently, double-skin facade (DSF) elements were further developed in a sense to additionally improve especially the in-plane stiffness of the load-bearing transparent timber-glass wall elements.

In a case of DSF elements an additional glazing pane is added. It is important to point out that the thermal-insulating three-layered glazing is placed on the internal side of the façade element and a single-layer non-insulating glazing on the external side, as schematically presented in Fig. 5b. For exterior glazing usually a laminated heat strengthened glass is prescribed, while two- or three-layered thermal insulating glazing on inner side consists of two annealed glass panes and a safety laminated heat strengthened glass for safety reasons and for thermal insulation. The width of the cavity between the both glass panes can vary from 200 mm to even more than 2 m and can importantly influence on the U-value of a such DSF element. The frame structure can be made of steel, aluminium, plastic or timber material. However, respecting ecological impacts only the case with the timber frame will be further studied in our case in the structural analysis.



a) b)

Fig. 5 a) Horizontal force distribution in SSF element; shear (in blue), tensile diagonal (in green); b) Composition of DSF load-bearing wall element [14]

Recently, many studies have analysed the thermal and acoustic performance of DSF elements [19-21]. There are also some important studies analysing the ecological impacts with various frame material [22], but almost none of them have analysed their structural behaviour, especially in terms of determining their racking resistance. All such DSF elements have been considered as in-plane non-resisting and of course in this sense also not implemented in any standards yet [18]. The numerical study in [22] is focused on the vertical load impact but does not address any racking resistance range. In a sense to study the racking behaviour of DSF elements wide experimental research was done in [12] finally resulting in European patent application product in [23]. Among that, a parametrical numerical study analysing some of the most important parameters influencing the racking resistance of DSF elements was done in [13]. Findings of this study will be directly implemented in our study of the 6-storey building in Section 4.

3. Mathematical modelling of load-bearing timber wall elements in prefabricated structures

A multi-storey prefabricated Light Timber-Framed (LTF) and cross-laminated timber (CLT) load-bearing wall elements can be effectively modelled using fictive diagonals for each lateral load-bearing wall element, as shown in Fig. 6. This approach simplifies the structural analysis of complex multi-storey timber buildings and requires significantly less computational time in comparison with all other possible approaches.

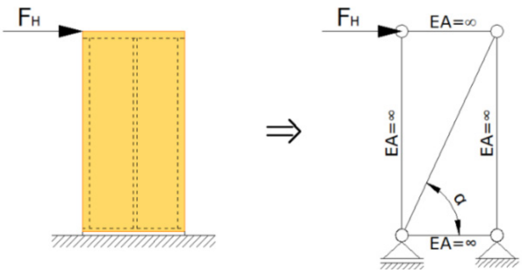


Fig. 6 Schematic presentation of the fictive diagonal model [14]

For Light Timber-Framed (LTF) walls, conventional sheathing material such as OSB or FPB are standard, but can be replaced with glass panes for single-skin (SSF) or double-skin (DSF) façade configurations. Walls with openings, such as windows or doors which are not stiff connected to the timber frame, are modelled without any diagonals and considered as non-load-bearing, as shown in Fig. 7b. The primary method for analysing these structures involves a calculation model with fictive diagonals, which simplifies the estimation of the horizontal stiffness and reduces the computational demands. The diameter of each fictive diagonal d_{fic} is calculated to ensure that the horizontal displacement of the modelled LTF wall matches that of the actual LTF wall, using the following formula:

$$d_{fic} = \sqrt{\frac{4 \cdot R \cdot L}{(\cos \alpha)^2 \cdot \pi \cdot E}} \tag{1}$$

where L is the length of the fictive diagonal, E is the elastic modulus of the fictive diagonal (E = 210 GPa if a steel bar is used for the diagonal in a calculation), and α is the inclination angle of the fictive diagonal. For LTF walls with OSB or FPB sheathing boards which are by mechanical fasteners connected to the timber frame, the racking stiffness R can be determined semi-analytically using Eq. 2. In this case, the γ-method prescribed by Eurocode 5 [18] can be used, taking into account the significant flexibility between the sheathing material and the timber frame elements. The diameter of the fictive diagonal element (Eq. 1) can be determined very rapidly in a semi-analytical form based on the effective bending stiffness $(EI)_{eff}$ calculated using the simple-beam theory and γ-method in a form of Eq. 2 by respecting the Eurocode 5 [17] expressions for the γ_i coefficient:

$$(EI)_{eff} = E_b I_b + E_t I_t = E_b \cdot \frac{n_b \cdot t \cdot b^3}{12} + E_t \cdot \left(\frac{2 \cdot a^3 \cdot c}{12} + \frac{d^3 \cdot c}{12} + 2 \cdot \gamma_i \cdot A_t \cdot z^2\right) \tag{2}$$

If the horizontal force F_H is acting at the top of the LTF wall element (Fig. 6) with the height H and the flexibility of the rocking and bottom real deformation are in this case both neglected (the both supports are for this study considered as rigid), the total flexibility of the wall element D is the sum of the in-line bending flexibility D_1 and the shear flexibility D_2 in the form of:

$$D = D_1 + D_2 = \frac{H^3}{3\,(EI)_{eff}} + \frac{H}{(GA)_{eff}} \tag{3}$$

The racking stiffness is then finally calculated in the form of:

$$R = \frac{1}{D} \tag{4}$$

For cross-laminated timber (CLT) wall elements, the racking stiffness R can be numerically obtained using special software program such as Calculatis [24], with the diameter of the fictive diagonal d_{fic} subsequently calculated for use in Eq. 1.

On the other hand, in the case of prefabricated DSF elements, the bonding line is fixed with a continuously distributed adhesive, and there are no mechanical fasteners that are point-connected to the timber frame. Therefore, respecting the Eurocode 5 [17], the γ-method cannot be adopted for the calculation for $(EI)_{eff}$ for LTF elements and consequent racking stiffness R at all and the calculation process for determining the fictive diagonal diameter d_{fic} is in this case much more complex and time consuming. In this case determination for R usually requires data from experimental studies [12] or at least a special spring model results using the finite element method (FEM) to calculate first the horizontal displacement under the acting horizontal point load F_H at the top of the wall element (Fig. 6). Crucial point in such FEM modelling is the approximation of sliding in the bonding line between both glass panes and the timber frame. This effect can be modelled by using two elastic springs in perpendicular directions (K_1 and K_2 respectively) in the form of:

$$K_1 = \frac{E_a \cdot A_a}{t_a} = \frac{E_a \cdot (w_a \cdot l_a)}{t_a} \qquad K_2 = \frac{G_a \cdot A_a}{t_a} = \frac{G_a \cdot (w_a \cdot l_a)}{t_a} \tag{5}$$

where E_a and G_a represent the modulus of elasticity and shear modulus of the adhesive, respectively, while t_a and w_a denote the thickness and width of the adhesive, respectively. The bonding length l_a serves as a parameter equal to the distance between selected springs. The mathematical modeling procedure with these springs, extensively detailed elsewhere, facilitates the determination of the racking stiffness R of load-bearing DSF elements, which can further be utilized to calculate the fictive diagonal diameter d_{fic} using appropriate equations. The whole calculation procedure is already fully described in [7, 14]. Once the horizontal displacement $u_H$under an acting force F_H is calculated the racking stiffness is finally calculated in the form of:

$$R = \frac{F_H}{u_H} \tag{6}$$

It is important to point out that this FEM calculation procedure allows for the determination of the DSF racking stiffness RRR in Eq. (1) without the need for costly and time-consuming experimental tests. This applies to DSF wall elements of arbitrary dimensions, glass pane thicknesses, and adhesive types and thicknesses.

4. Numerical study of a six-storey prefabricated timber building

4.1 Building design

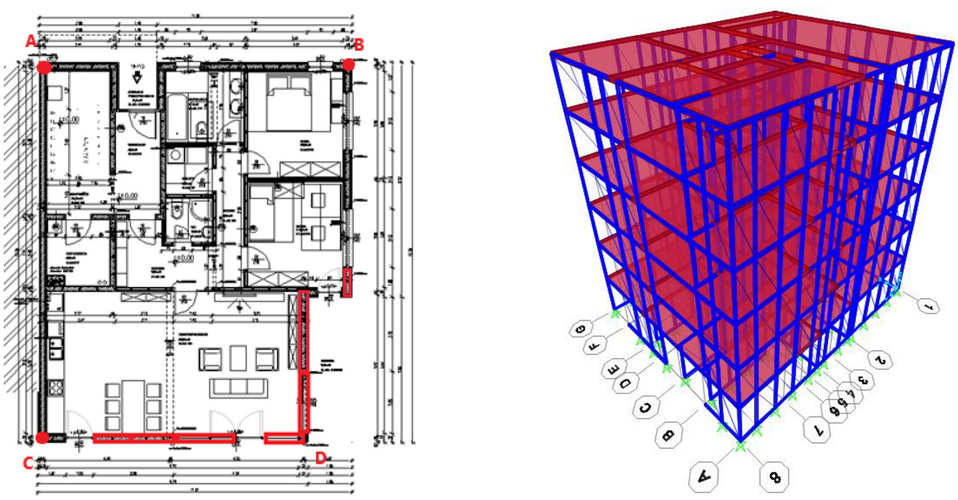
This study examines a six-storey prefabricated building specifically selected to evaluate the effects of installing additional double-skin façade (DSF) elements to increase lateral load resistance and stiffness and ensure compliance with Eurocode 5 [18] and Eurocode 8 [25] structural requirements. However, Eurocode 8 [25] does not specifically address any earthquake resistant DSF configurations. Therefore, two different calculation cases for two different structural systems (hybrid and non-hybrid) are performed:

- Considering DSF wall elements as non-load-bearing to evaluate their effect on horizontal loads according to current Eurocode 8 [25] requirements;
- DSF wall elements are treated as in-plane load-bearing to evaluate their effect on increased horizontal load resistance of the whole building.

A mathematical analysis of a six-storey prefabricated timber building with a maximal height H of 15 m is carried out, with particular emphasis on the horizontal stiffness and the natural frequencies of oscillation. The ground floor plan is shown in Fig. 7a and the building complies with the height requirements of Eurocode 8 [25]. The building envelope is bounded by load-bearing walls from the first floor to the top floor.

In the first studied case (Case 1), all internal wall elements consist of load-bearing walls in Light Timber-Framed (LTF) construction accompanied by traditional fibreboard (FPB) sheathing, identified on the floor plans by black filler. In this hybrid structural wall system solution all external walls on the building envelope are constructed from higher resistant cross-laminated timber (CLT) panels and, like the interior walls, are made from C24 class timber.

In the second analysed case (Case 2) in a sense to increase the overall racking stiffness of the whole building and to ensure the prescribed Eurocode conditions for maximal horizontal displacements [18, 25], both the internal and external wall elements are composed of CLT timber components only and the structural system is thus non-hybrid. There are many advantages and disadvantages of CLT and LTF structural systems according to the structural and non-structural facts which are deeply studied in [17] where also many benefits of hybrid structural solutions are discussed.



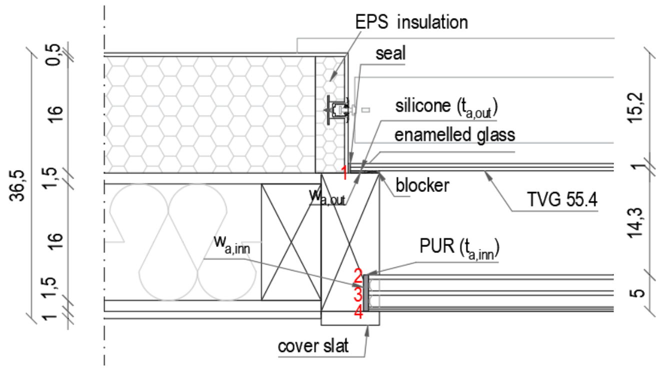
a) b)

Fig. 7 a) Floor-plan of the building; b) Computational model of the structure made by SAP 2000 software

In both cases the transparent DSF wall sections are marked with red markings (Fig. 7a). Other transparent areas, which represent windows and doors, are shown with white infill and are treated as non-load-bearing elements in the structural analysis in performed numerical cases. These components do not contribute to the load-bearing capacity of the building and are excluded from the structural integrity calculations at all.

The computational model shown in Fig. 7b was developed using the structural analysis and dynamics software SAP 2000. Although the building meets the plan correctness required by the Eurocode 8 [25], a 3D structural model was used. The analysis includes all relevant load-bearing wall elements, which are represented by fictive diagonals. Their effective cross-sectional areas are calculated based on Eqs. 1-3 for Light timber-framed (LTF) walls with FPB sheathing and Eqs. 2 and 4 for load-bearing DSF walls. For CLT walls, the racking stiffness R is determined using the Calculatis program [24] and further used in Eq. 1 to calculate the effective cross-sectional diameter d_{fic}.

Fig. 8 shows a cross-sectional view of a DSF wall element with different adhesives. Two types of adhesives are used: Polyurethane Ködiglaze P for the inner triple thermal insulation glazing and Silicone Ködiglaze S for the outer single glazing. The timber frame of the construction consists of glulam (GL24h) in accordance with classification EN 1194 [26]. The internal glass pane consists of thermal-insulating three-layered glazing while the outer glass pane is single-layer and is made of toughened laminated glass.



1. Laminated outer glass (5+5 mm)
2. Float glass (6 mm)
3. Float glass (6 mm)
4. Laminated glass (4+4 mm)

Fig. 8 Schematic presentation of a DSF load-bearing structural wall element

The input data for two types of adhesives and the material properties of the glass and timber elements are listed in Tables 1 and 2, respectively.

Table 1 Material properties of the adhesives [27, 28]

	Poisson's ratio ν	Shear modulus G_s (MPa)	Elastic modulus E_0 (MPa)	Decay time t_d (s)	Decay constant β
Silicone (Ködiglaze S)	0.5	0.351	1.053	100	0.0026
Polyurethane (Ködiglaze P)	0.49	0.454	1.354	290	0.0016

Table 2 Material properties of the timber [26] and glass components [29, 30]

	Timber frame GL24h [26]	Float glass [29]	Thermally toughened glass [30]
Standard	EN 1194	EN 12150	EN 12150
E (Mpa)	II 11,600 ⊥ 720	70,000	70,000
ν (-)	II 0.25 ⊥ 0.45	0.23	0.23
G (MPa)	II 720 ⊥ 35	0.45	0.45
f_t (MPa)	II 14 ⊥ 0.5	45	120
f_c (MPa)	II 14 ⊥ 0.5	500	500
ρ (kg/m^3)	380	2,500	2,500

4.2 Numerical analysis

The polyurethane adhesive used in the calculations for fixing the internal glazing has a thickness of $t_{a,inn}$ = 7 mm and a width of $w_{a,inn}$ = 28 mm, parameters that match those of the experimentally tested DSF specimens [12]. The influence of the polyurethane adhesive with additional parametrically chosen values of thickness $t_{a,inn}$ = 3, 5, 7 and 9 mm) on a racking stiffness of a single DSF wall element is numerically studied in [13]. Additionally, a huge experimental study using polyurethane and epoxy adhesive with an emphasis on the comparison of experimental results with SSF elements is given in [12].

Considering the extensive load-bearing structure of the whole building, the application of the spring model calculation is computationally prohibitive. Therefore, we use the mathematical model with fictive diagonals as described in Section 3. The cross-section diameter d_{fic} of the fictive diagonal elements is determined using Eqs. 2-6, with the calculated values varying depending on the type of a prefabricated timber wall element (LTF, CLT, DSF).

Table 3 presents the calculated fictive diagonal diameters and the racking stiffness for each type of resistant wall element: FPB sheathing boards for internal timber wall elements, load-bearing CLT elements for external/internal wall elements and load-bearing DSF elements. Both presented values for the racking stiffness R could be used for the DSF elements: 909 N/mm from the comprehensive experimental study [12] or 857 N/mm from the elastic FEM spring model [13]. However, to simplify the whole procedure only the value of 857 N/mm is further used for the numerical analysis of the entire six-storey building.

The calculated cross-sectional values of the fictive diagonal d_{fic} show that the racking stiffness of CLT wall elements is significantly higher than that of Light Timber-Framed (LTF) wall elements with conventional fibre-plaster sheathing boards (FPB) and also significantly higher compared to the resistant DSF elements. The problem of the relatively low in-plane stiffness of DSF elements was comprehensively analysed and discussed in [13] both based on experimental results and in a subsequent parametric numerical study using an elastic spring FEM model.

Table 3 Diameter of the fictive diagonals and load-bearing capacities of the wall elements

Load-bearing wall elements	Racking stiffness R of the resisting wall elements (N/mm)	Diameter of the fictive diagonal d_{fic} (mm)
DSF (experimental)	909	8.78
DSF (spring model)	857	8.52
CLT external wall	5602	21.79
LTF internal wall	3425	17.04

4.3 Numerical results and discussion

The oscillation times (first three modes) of the six-storey building for the two analysed cases load-bearing and non-load-bearing DSF elements are first calculated using a 3D FEM model (Fig. 7b) with fictive diagonals and SAP 2000 software. In the first case, the interior of the building consists of load-bearing walls in Light Timber-Framed (LTF) construction, supplemented by traditional fibreboard (FPB) sheathing. The exterior walls are made of cross-laminated timber (CLT) panels and like the interior walls are made from C24 timber. In the second case in a sense to enlarge the overall racking stiffness of the building, both the internal and external wall elements are made of CLT timber exclusively. The results for the calculated first three natural oscillation modes (T_1, T_2, T_3) for both cases, taking into account the stiffness contribution of non-load-bearing and load-bearing DSF elements, are shown in Table 4.

Table 4 Oscillation times (T_1, T_2, T_3) of the six-storey building considering both cases for load-bearing and non-load-bearing DSF wall elements

DSF element	Non-load-bearing DSF elements		Load-bearing DSF elements	
Oscillation mode	Oscillation times T (s)		Oscillation times T (s)	
	Case 1 (CLT+LTF)	Case 2 (all in CLT)	Case 1 (CLT+LTF)	Case 2 (all in CLT)
1. (T_1)	0.934	0.840	0.839	0.765
2. (T_2)	0.721	0.672	0.689	0.645
3. (T_3)	0.516	0.506	0.507	0.498

As expected, the oscillation times are higher when non-load-bearing DSF elements are considered, which is due to the lower overall racking stiffness of the structure, while the mass remains unchanged. Also, in the first case (hybrid CLT+LTF), where the interior of the building consists of load-bearing walls of Light timber-framed construction (LTF) and the exterior walls of cross-laminated timber panels (CLT), the oscillation times are higher than in the second case, where the internal and external wall elements consist of CLT timber. Among that, it can be observed, that the oscillation times are significantly reduced in both cases by considering the additional stiffness of the DSF elements. This is particularly evident for the first oscillation time T_1 and least evident for the third oscillation time T_3. The decrease in T_1 for Case 1 is 11.13 % and for Case 2 8.23 %. It is a quite logical because the overall stiffness is higher in the case of non-hybrid CLT structural wall system and therefore an additional contribution of DSF is less evident.

Tables 5 and 6 present the calculated horizontal racking stiffnesses R and displacements of the structure in the two global orthogonal directions of seismic action (X and Y directions) at selected control points (A-D, see Fig. 9). For the calculation of the displacements, a rather large seismic intensity with a_g = 0.225·g is deliberately chosen. Numerical analysis under a higher random excitation of a_g = 0.30· g and a_g = 0.40·g performed on one and two-storey timber box-house models previously experimentally tested in [8] with SSF elements is additionally presented in [11].

Both cases with load-bearing and non-load-bearing DSF elements are considered in these calculations. According to [25], the allowed value of horizontal displacements in a multi-storey building is H/500, which corresponds to 30 mm in our case.

Table 5 Racking stiffnesses R and displacements u of the corner points on the top storey of the six-storey building for Case 1 (hybrid CLT+LTF)

	DSF Element	Non-load-bearing DSF elements		Load-bearing DSF elements	
	Earthquake	Direction X	Direction Y	Direction X	Direction Y
Point	Displacement (mm)				
A	u_x	19.27	21.51	19.56	20.46
	u_y	7.59	15.51	6.20	15.05
	u_R	20.71	26.52	20.52	25.40
B	u_x	19.26	21.50	19.56	20.45
	u_y	28.12	26.81	22.61	26.87
	u_R	34.08	34.37	29.90	33.77
C	u_x	36.32	22.38	33.58	17.97
	u_y	7.60	15.51	6.21	15.06
	u_R	37.11	27.23	34.15	23.45
D	u_x	36.33	22.39	33.58	17.97
	u_y	28.15	26.85	22.63	26.89
	u_R	45.96	34.96	40.49	32.34
	R (N/mm)	6639	9209	8168	10169
EC 8 requirement [25] u_{max} = $H/500$		30.00	30.00	30.00	30.00

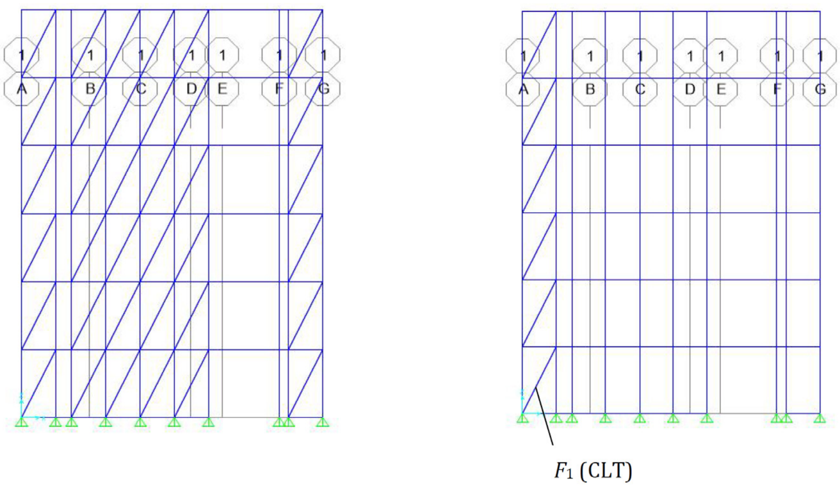
It is presented again that the increase in overall structure racking stiffness (R) by using additional DSF elements as load-bearing is very influent. For instance, in the X-direction this increase is of 23.03 % and in the Y-direction 10.42 %. Again, respecting the floor-plan design in Fig. 7a, it is logical, because practically all façade elements in south (X) direction are made from DSF wall components. On the other hand, in the east orientation (Y) direction the percentage of DSF elements is essentially lower.

In this case of completely non-hybrid and more rigid CLT structure the increase of overall racking stiffness R is of 17.26 % in X-direction and 16.27 % in the Y-direction. So, compared to Case 1, the increase in the X-direction is significantly smaller, but even slightly larger in the Y-direction, which is due to the fact that more load-bearing DSF elements are placed on the south façade (X-direction) than on the east façade (Y-direction). Additionally, the observed maximal horizontal displacements exceed the limits prescribed by Eurocode 8 [25] by approximately 53 % (Case 1) and 33 % (Case 2) if the DSF elements are considered as non-resisting. However, by considering DSF elements as resisting elements, the exceeded values for horizontal displacements essentially decrease and are higher only for 35 % (Case 1) and 20 % (Case 2). These values are highlighted in red in the table. Of course, if the selected seismic excitation would be a little lower (less than $0.15 \cdot g$), these Eurocode conditions could easily be met for the case of a 6-storey building.

Table 6 Racking stiffnesses R and displacements u of the corner points on the top storey of the six-storey building for Case 2 (all in CLT)

	DSF Element	Non-load-bearing DSF elements		Load-bearing DSF elements	
	Earthquake	Direction X	Direction Y	Direction X	Direction Y
Point	Displacement (mm)				
A	u_x	20.21	19.48	19.35	18.79
	u_y	7.36	15.20	5.99	14.57
	u_R	21.51	24.71	20.26	23.78
B	u_x	20.21	19.48	19.35	18.78
	u_y	26.08	25.22	22.17	24.56
	u_R	32.99	31.87	29.43	30.92
C	u_x	30.01	23.00	28.02	19.26
	u_y	7.36	15.20	5.99	14.57
	u_R	30.90	27.57	28.65	24.15
D	u_x	30.01	23.00	28.02	19.26
	u_y	26.11	25.25	22.18	24.59
	u_R	39.78	34.15	35.74	31.23
	R (N/mm)	8623	9865	10111	11470
EC 8 requirement [25] $u_{max} = H/500$		30.00	30.00	30.00	30.00

As this is a 6-storey timber structure, where usually considerable problems with the prescribed Eurocode Serviceability limit state conditions [17, 25] in ensuring for maximum horizontal displacements ($H/500$) already exist, it is also interesting to compare the contribution of the load-bearing DSF elements compared to the contribution of the use of CLT wall elements also for the internal wall elements. Comparing the values for R in the X- and Y-direction between Tables 6 and 5, it can be seen that the use of load-bearing DSF elements in the hybrid structural system (CLT+LTF; Case 1) results in essentially very similar stiffnesses (R_x = 8168 N/mm and R_y = 10169 N/mm) as in the CLT-only structural system (Case 2), and where all DSF elements are non-load-bearing (R_x = 8623 N/mm and R_y = 9865 N/mm).



a) b)

Fig. 9. View along Axis 1 of the six-storey wall model using: a) load-bearing DSF elements and b) non-load-bearing DSF elements

Finally, at the end of the study, the horizontal force acting on the CLT corner wall element 1 along the axis 1 (F_1 in Fig. 9b) is specifically monitored to evaluate a possible reduction of the torsional effects caused by the asymmetric plan due to the positioning of the transparent DSF elements. It is assumed that the use of additional DSF elements as load-bearing components, especially on the south façade, will help to reduce the force acting on the primary load-bearing CLT corner element. Tables 7 and 8 show the calculated horizontal forces for Cases 1 and 2 in both directions (F_x and F_y) acting on this CLT wall element due to seismic actions in the two global orthogonal directions (X and Y). In these tables, the results are compared for both configurations with non-load-bearing and with load-bearing DSF elements. The resulting diagonal force F_R acting on this CLT element is also calculated.

Table 7 Horizontal forces in the exterior walls of the building (Case 1 – hybrid CLT+LTF)

DSF Element		Non-Load-Bearing DSF Elements		Load-Bearing DSF Elements	
Earthquake		Direction X	Direction Y	Direction X	Direction Y
Axis	Force (kN)				
1	F_x	29.25	18.37	29.78	16.02
	F_y	89.56	63.05	96.38	62.95
	F_R	94.22	65.67	100.88	64.96
8	F_x	16.56	19.17	15.87	19.29
	F_y	66.40	78.94	59.81	80.81
	F_R	68.43	81.23	61.88	83.08
A	F_x	8.02	16.76	6.68	16.25
	F_y	89.56	63.05	96.38	62.95
	F_R	89.92	65.24	96.61	65.01
G	F_x	0.09	0.08	0.08	0.08
	F_y	4.15	2.90	20.93	11.68
	F_R	4.15	2.90	20.93	11.68
Axis 1:	F_1	29.25	63.05	29.78	62.95

Table 8 Horizontal forces in the exterior walls of the building (Case 2 – all in CLT)

DSF Element		Non-Load-Bearing DSF Elements		Load-Bearing DSF Elements	
Earthquake		Direction X	Direction Y	Direction X	Direction Y
Axis	Force (kN)				
1	F_x	23.52	18.35	24.08	16.76
	F_y	71.98	62.23	96.38	62.10
	F_R	75.73	65.84	78.98	64.34
8	F_x	17.41	16.63	16.18	16.95
	F_y	72.54	71.84	65.09	73.78
	F_R	74.60	73.74	67.07	75.70
A	F_x	7.39	16.04	6.03	15.30
	F_y	71.68	62.23	78.98	62.10
	F_R	72.06	64.26	79.21	63.96
G	F_x	0.07	0.07	0.07	0.07
	F_y	3.98	3.01	18.11	12.48
	F_R	3.98	3.01	18.11	12.48
Axis 1:	F_1	23.52	62.23	24.08	62.10

The results presented show that in the case of load-bearing DSF elements, the force F_1 in the X-direction was reduced by 1.8 % (Case 1) and by 2.3 % (Case 2), while in the Y-direction the force remained approximately the same in both cases. This is quite logical, as Table 3 shows that the stiffness of the CLT beams is significantly higher than of the LTF wall elements. This means that the contribution of considering DSF as load-bearing elements in LTF structures is much more important. In addition, the reduction of the acting force in the X-direction F_x on the CLT member is lower than in the similar study [14], in which a three-storey building with the same floor plan built exclusively in a Lightweight Timber-Framed (LTF) system was analysed.

5. Conclusion

The numerical study carried out on the selected 6-storey prefabricated timber structure clearly demonstrated the importance of considering the previously developed and pa-tended timber DSF elements [23] as additional bracing envelope wall components of the structure. In the analysis, we have deliberately chosen two computational cases, where first the importance of the DSF load-bearing elements has been analysed in the case of a hybrid structural system (CLT+LTF), and secondly in the case of a generally more rigid non-hybrid full CLT system. Also, all DSF elements were deliberately placed rather asymmetrically around the building envelope, entirely on the south façade (in the X-direction) and partially also on the east façade (in the Y-direction), as is also the norm in contemporary multi-storey timber buildings.

In both cases, the results of the studies carried out showed a significant increase in the horizontal stiffness of the whole building when DSF elements are considered as load-bearing. In Case 1, this increase was 23 % in the X-direction and 10.4 % in the Y-direction, while in Case 2 it was 17.3 % and 16.3 % respectively, i.e. much more symmetric. Consequently, the increased horizontal stiffness of the whole building also makes it much easier to meet the prescribed Eurocode conditions for maximum displacements in the case of using load-bearing DSF elements.

However, as already analysed in detail in [17], hybrid structural systems are usually the most optimal solution in multi-storey prefabricated timber buildings when several different aspects, both structural, structural-physical and environmental, are considered. Particularly important in this respect is the combination of Light Timber-Framed (LTF) and Solid-timber system in the form of Cross-laminated Timber (CLT), which was basically considered in the design of our structure in Case 1. With Case 2, where all the internal load-bearing wall elements constructed in LTF system were replaced by the more rigid CLT, we only followed the basic design objective of increasing the horizontal stiffness of the whole building envelope, but not in some other also very important respects. However, the analysis carried out has shown that a very similar increase in the horizontal stiffness of the whole building can be achieved in fact by considering the pa-tented DSF elements as additional load-bearing elements. In this case, of course, there is further no need to design the building exclusively in CLT structural wall system, which may be inferior in some respects to a hybrid (CLT+LTF) design.

The results of the study highlight the importance of carefully considering the load-bearing DSF elements in seismic design, as their incorporation can have a significant impact on the overall behaviour of the multi-storey timber structure through their influence on the displacements and stiffness properties. In a sense of European standards, it exist currently only some guidance for European structural design of timber-glass components [31]. Taking these considerations into account is crucial to ensure compliance with seismic design codes [25] and to improve the structural resilience of multi-storey prefabricated timber buildings in a sense to be further implemented also in Eurocode 8 standard.

Acknowledgement

Funding for this research was provided by the Slovenian Research Agency, National research programme P2-0129.

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