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TIMBER-FRAMED SHEAR WALLS WITH LARGE OPENINGS AS PART OF THE LATERAL FORCE-RESISTING SYSTEM - OPTIMIZATION OF THE SHEATHING-TO-FRAMING CONNECTION LAYOUT

Nadja Manser[1], René Steiger[2], Martin Geiser[3], Lukas Kramer[4], Andrea Frangi[5]

ABSTRACT: For lateral force-resisting systems of multi-storey timber-framed buildings, the usual policy of current standards is to only consider timber-framed shear wall segments ranging continuously from the ground floor to the top edge of the building and to neglect wall elements with openings. Developing a design method that allows taking wall elements with openings into account, would make the lateral force-resisting system more efficient and respective buildings more economic. This paper presents investigations of the combination of sheathing thickness and arrangement of the fasteners connecting the sheathing to the framing to maximize the load-carrying capacity of the wall while guaranteeing a ductile failure of the connection. The results of the study provide the basis for further experimental investigations on one- and two-story wall elements with large openings with the final goal of developing a design method for timber-framed shear walls with openings.

KEYWORDS: Timber-framed walls, shear walls, sheathing, OSB/3, openings, lateral stiffness, design

1 INTRODUCTION

Timber-framed shear walls are commonly used for the construction of buildings of low to medium heights. It is the usual policy in several current design standards, e.g. SIA 265, 2021 [1], DIN 1052, 2008-12 [2], EN 1995-1-1, 2008 [3], to adopt the so-called segmentation approach for the design of timber-framed shear walls contributing to the lateral force-resisting system (Figure 1, left). Thus, only wall segments, which continuously range from the ground floor to the top of the building can be considered to contribute to the lateral force-resisting system.

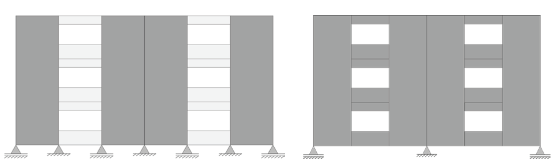

Figure 1: *Segmentation approach in the design of timber-framed shear walls with openings (left) versus considering all parts of wall elements with openings (right).*

However, modern architectural designs often demand flexible interior room arrangements, making exterior wall stiffening more important. Since modern architectural designs are characterized by numerous and in particular large openings (Figure 2), designing the lateral force-resisting system of a building using timber-framed shear walls gets very challenging, and sometimes even becomes impossible, since not enough continuous wall segments may be available to resist the lateral forces. The efficiency of the lateral force-resisting system could be enhanced by mobilizing wall segments with openings in the design (Figure 1, right). A beneficial side effect of this design approach is that the number of expensive anchorages is reduced.

Figure 2*: Buildings showing the recent trend in modern architectural design towards façade walls with large openings. [4]*

The objective of an ongoing research project in Switzerland, which forms the basis for the subsequent investigations and findings presented in this paper, is to develop a design method for timber-framed shear walls with openings. This will be accomplished by executing full-scale experiments on such walls and by an analysis of their behaviour under lateral loading. In order to come to an efficient design of wall specimens with openings, it is necessary to first of all define an appropriate and optimized sheathing-to-framing connection layout. This particular

[1] Nadja Manser, Empa – Swiss Federal Laboratories for Materials Science and Technology, Switzerland, nadja.manser@empa.ch
[2] René Steiger, Empa – Swiss Federal Laboratories for Materials Science and Technology, Switzerland, rene.steiger@empa.ch
[3] Martin Geiser, BFH – Bern University of Applied Sciences, Switzerland, martin.geiser@bfh.ch
[4] Lukas Kramer, BFH – Bern University of Applied Sciences, Switzerland, lukas.kramer@bfh.ch
[5] Andrea Frangi, ETH Zürich – Swiss Federal Institute of Technology, Zurich, Switzerland, frangi@ibk.baug.ethz.ch

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first-step investigations were performed on wall elements *without* openings.
In seismic active areas, timber-framed shear walls are typically designed to exhibit ductile failure of the sheathing-to-framing connection. This requires that buckling and shear failure of the sheathing and a failure of the framing elements is prevented in order to ensure that ductile failure of the sheathing-to-framing connection remains the governing failure mode. An optimal sheathing-to-framing connection layout allows for the maximization of the load-carrying capacity of the wall, while ensuring that the fasteners undergo ductile failure when the load-carrying capacity is reached and avoiding brittle failure of the sheathing.

2 STATE OF THE ART

2.1 TIMBER-FRAMED WALL CONSTRUCTION

The regional differences in the construction of timber-framed shear walls can be observed comparing the dimensions of the framing elements, the thickness and the materialization of the sheathing, and the type of fasteners utilized. In Europe, the dimensions of the framing elements are generally larger compared to e.g. North-America and the thickness of the OSB/3 sheathing is commonly 15 mm. The typical construction of timber-framed shear walls in Switzerland and in other European countries involves the application of staples for the sheathing-to-framing connection. In Switzerland, more than one row of fasteners is not an uncommon practice. Since the study presented in this paper is a part of a research project in Switzerland, the focus is on the type of timber-framed shear walls prevalent in Switzerland and Europe.

2.2 RULES FOR THE DESIGN OF WALLS WITHOUT OPENINGS

2.2.1 Swiss standard SIA 265, 2021

In the Swiss standard, SIA 265, 2021, some general specifications for the design of timber-framed shear walls without openings are provided but no specific design rules are available. Hence, design rules taken from either EN 1995-1-1, 2008 or from DIN 1052, 2008-12 are applied.

2.2.2 European standard EN 1995-1-1, 2008

According to EN 1995-1-1, 2008, the design shear capacity of a timber-framed shear wall $F_{\mathrm{i,v,Rd}}$ can be determined from Formula (1).

$$F_{\mathrm{i,v,Rd}} = \frac{F_{\mathrm{f,Rd}} \cdot b_{\mathrm{i}} \cdot c_{\mathrm{i}}}{s} \tag{1}$$

where $F_{\mathrm{f,Rd}}$ = design load-carrying capacity of a single fastener, b_{i} = width of a single shear wall element, s = distance between fasteners, $c_{\mathrm{i}} = 1.0$ if $b_{\mathrm{i}} \geq b_0$ and $c_{\mathrm{i}} = b_{\mathrm{i}}/b_0$ if $b_{\mathrm{i}} < b_0$, where $b_0 = h/2$ with h = height of the shear wall element.

In EN 1995-1-1, 2008 it is assumed that the load-carrying capacity of the sheathing-to-framing connection is the governing factor for any wall geometry. This corresponds to the assumption that the resistance of the fasteners is always lower than the resistance of the sheathing. However, this assumption may not be valid when multiple rows of fasteners are used.

2.2.3 German standard DIN 1052, 2008-12

According to DIN 1052, 2008-12 the design shear strength of a timber-framed shear wall without opening can be calculated using Formula (2).

$$f_{\mathrm{v,0,d}} = \min \begin{cases} k_{\mathrm{v,1}} \cdot R_{\mathrm{d}}/a_{\mathrm{v}} \\ k_{\mathrm{v,1}} \cdot k_{\mathrm{v,2}} \cdot f_{\mathrm{v,d}} \cdot t \\ k_{\mathrm{v,1}} \cdot k_{\mathrm{v,2}} \cdot f_{\mathrm{v,d}} \cdot 35 \cdot t^2/a_{\mathrm{r}} \end{cases} \tag{2}$$

where $k_{\mathrm{v,1}}$ = factor to account for the arrangement and the way of connecting the panels ($k_{\mathrm{v,1}} = 1.0$ if the sheathing is connected to the framing on all edges in a sufficiently stiff manner in terms of transferring of shear stresses), $k_{\mathrm{v,2}}$ = factor to account for additional stresses (influences described in the next paragraph), R_{d} = design load-carrying capacity of a single fastener, a_{v} = distance between the fasteners, $f_{\mathrm{v,d}}$ = design shear strength of the panel, t = thickness of the sheathing, a_{r} = distance between the studs.

The first line in Formula (2) considers the limit given by the resistance of the sheathing-to-framing connection. The second line describes the limit given by the resistance of the panel and the third line the limit given by the buckling resistance of the panel. The $k_{\mathrm{v,2}}$ factor serves in accounting for additional stresses, which lead to a reduction of the shear strength of the sheathing. These stresses may originate from:

- the distance between the axis of the framing elements and the sheathing (Figure 3, a)),
- a discontinuous shear flow (Figure 3, b)),
- forces acting perpendicular to the axes of the framing elements (Figure 3, c)).

In DIN 1052, 2008-12 the $k_{\mathrm{v,2}}$ factor is set to 0.33 for the design of sheathing placed on only one side of the wall and to 0.50 for sheathing placed on both sides of the wall.

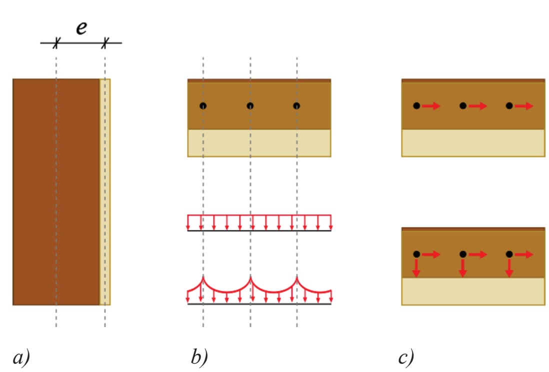

Figure 3*: Influences reducing the shear strength of the sheathing in timber-framed shear walls described in DIN 1052, 2008-12. a) Eccentricity between framing axis and sheathing axis, b) deviation from the model assumption of a continuous shear flow (top: model assumption, bottom reality), c) deviation from the model assumption of forces acting only parallel to the framing element axis (top: model assumption, bottom: reality).*

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2.2.4 Current working draft of Eurocode 5, prEN 1995-1-1, 2022

The rules for the design of wall diaphragms in the current working draft of Eurocode 5, prEN 1995-1-1, 2022 [5] are based on the design procedure specified in DIN 1052, 2008-12. The $k_{\mathrm{v},2}$ factor was renamed to $k_{\mathrm{p,model}}$ and increased to 0.5 for sheathing placed on one, and to 0.67 for sheathing placed on both sides of the wall. In addition to the three influences listed in DIN 1052, 2008-12, along with the design rules in prEN1995-1-1, 2022 two additional influences which need to be accounted in shear wall design (via the $k_{\mathrm{p,model}}$ factor) are mentioned:

- Model assumption of pinned connections between ribs while the actual connections are often weaker (Figure 4, a)),
- Eccentricity of the axes of the framing if the framing elements are of different depth in cross-section (Figure 4, b)).

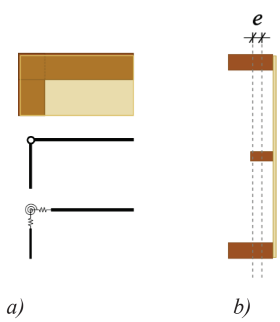

***Figure 4**: Influences reducing the shear strength of the sheathing in timber-framed shear walls compared to DIN 1052, 2008-12 additionally described in prEN 1995-1-1, 2022. a) Deviation from the model assumption of pinned connected framing elements (top: model assumption, bottom: reality), b) eccentricities within the framing elements.*

3 GOAL OF THE STUDY

The $k_{\mathrm{v},2}$ factor specified in DIN 1052, 2008-12 seems to have been introduced to overcome the problem of unaccounted additional stressed in the shear wall designs, which can reduce the shear strength of the sheathing. When looking at literature, it is unclear, to what extent these values are based on experimental research. Up to now, no results from experiments on timber-framed shear walls using OSB panels as sheathing material are available. This leads to the conclusion that the basis in terms of experimental data for the determination of the $k_{\mathrm{v},2}$ factor (or $k_{\mathrm{p,model}}$ factor, respectively) is still poor.

In Switzerland, it is common practice to design the fasteners (i.e. the staples) of the sheathing-to-framing connection in multiple rows. In consequence, the need for a more accurate quantification of the $k_{\mathrm{v},2}$ factor needed for an accurate calculation of the shear strength of the sheathing of timber-framed shear walls arises. The results of this study will help in defining the optimal combination of sheathing thickness and fastener layout in order to maximize the load-carrying capacity of the wall element, while avoiding brittle failure of the sheathing.

An experimental campaign was conducted to investigate the shear resistance of OSB/3 panels in timber-framed shear walls without openings. The resulting shear resistance of the OSB/3 panels was compared to analytic estimations based on the design rules available in the standards mentioned above. The ratios calculated from a comparison of experimental data and estimate from design code formulae correspond to the reduction of the shear strength of the OSB/3 sheathing for applications in timber-framed shear walls.

4 MATERIALS

4.1 SHEATHING PANELS

The panels investigated were of type OSB/3 with thicknesses of 12 mm, 15 mm, 18 mm and 25 mm, and dimensions (i.e. width and height) of 1250 mm and 2500 mm. The panels for all test specimens originated from one single production batch for each panel thickness. The moisture content was determined by using the oven-dry method in accordance with the test procedure specified in EN 322, 1993 [6] and was $7.2 \pm 0.4\%$, which fulfils the requirements of EN 300, 2006 [7].

4.2 FRAMING ELEMENTS

The framing elements were composed of Swiss-grown Norway spruce (*picea abies*) glulam GL24h (EN 14080, 2013 [8]). The moisture content was determined by measuring the electic resistance and was $10.3 \pm 0.7\%$.

4.3 SHEATHING-TO-FRAMING CONNECTION

For the sheathing-to-framing connection, two different types of fastener were used:

- resined staples with a diameter of 1.53 mm and a length of 50 mm (Haubold, KG 700),
- threaded nails with a diameter of 3.10 mm and a length of 90 mm. (Haubold 3.10 x 90 Ring Wire Weld).

5 METHODS

Experiments on wall elements, as well as on OSB/3 panels were carried out. The latter aimed at determining the mechanical properties of the OSB/3 panels.

5.1 TESTS ON OSB/3 PANELS

The shear strength of the OSB/3 panels was determined in accordance with the test procedure specified in the European standard EN 789, 2005 [9]. Five specimens were tested for each sheathing thickness for both parallel and perpendicular strand orientation of the top layer.

5.2 SHEAR WALL TESTS

In order to evaluate the shear resistance of OSB/3 panels when applied as sheathing in timber-framed shear walls, full-scale wall elements were tested. The study focused on wall configurations where the OSB/3 sheathing was placed on one side only. The test setup, including the terminology used for the framing elements and the definitions of fastener distances, is depicted in Figure 7.

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5.2.1 Design of the specimens

The specimens were designed to ensure that failure would occur in the sheathing, rather than in the sheathing-to-framing connection. To provoke this particular failure mode, sheathing-to-framing connection, anchoring, and framing elements were over-designed in comparison to the shear resistance of the sheathing. The shear resistance and the resistance against buckling of the panels (as described in the second and third line of Formula (2)) were evaluated on design level, using:

- $k_{\mathrm{v},1} = 1.0$,
- $k_{\mathrm{v},2} = 1.0$, assumption as starting point for the factor to be investigated,
- $f_{\mathrm{v,d}} = k_{\mathrm{mod}} \cdot f_{\mathrm{v,k}}/\gamma_{\mathrm{M}} = 6.23\ \mathrm{N/mm^2}$, where k_{mod}, γ_{M}, and $f_{v,k}$ are 1.1, 1.2, and 6.8 N/mm^2, respectively, according to EN 1995-1-1, 2008,
- t: Thickness of the sheathing (nominal values).

Due to lack of information the over-strength factor to design the sheathing-to-framing connection and the anchoring was conservatively selected 1.5. This resulted in a high number of fastener rows. The space needed to arrange the fasteners influenced the width of the edge studs and the top and the bottom rail. In consequence, the framing elements compared to the sheathing, have an over-strength higher than 1.5.

The panels with thicknesses 12 mm, 15 mm, and 18 mm, were connected to the framing elements by means of resined staples. For the 25 mm thick panels, threaded nails were used, since the resined staples used in practice are technically difficult to insert in the 25 mm thick OSB/3 panel and the penetration depth in the framing members would be critically low.

The size of the specimen was governed by the dimensions of the panels (2.50 m and 1.25 m) and an offset of 20 mm (Figure 7 d_1) on all edges to prevent local crushing of the sheathing due to increasing deformations during the tests. The width and the height of the specimen were 1.29 m and 2.54 m respectively. To ensure comparability, the geometric configuration of the three stapled wall types was kept identical. The layout of the sheathing-to-framing connection and the dimensions of the framing elements as well as of the anchoring were taken based on the test specimens with the 18 mm thick sheathing, while the number of inner rails was taken based on the test specimens with the 12 mm thick panel since the resistance against buckling failure is minimal for the thinnest panel. Three specimens were tested for each wall type. In Table 1 the materials used and the geometrical properties are listed for all the four wall types investigated.

In order to prevent compression failure parallel to the grain of the edge studs and the top and the bottom rail, the corners of the walls were designed in such way that the rails and the edge studs both ranged continuously to the edges of the wall elements (mortise and tenon joints). The top and the bottom rails were constructed overlapping in order to simplify the installation of the test specimens in the test rig as well as the force transfer from the hydraulic jack to the specimen. (Figure 5)

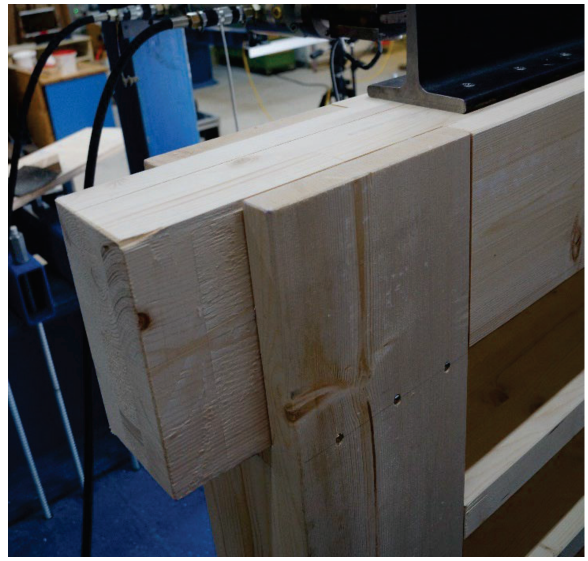

Figure 5: *Construction detail of mortise and tenon joints at the corners of the tested timber-framed shear wall elements.*

5.2.2 Loading protocol

The walls were tested according to the testing standard ISO 21581, 2010 [10], using the force controlled testing protocol shown in Figure 6. $F_{\mathrm{max,est}}$ was determined by estimating the shear capacity of the wall element resulting from the shear resistance of the panel (as described in the second line of Formula (2)) taking the parameters as follows:

- $\mathrm{k}_{\mathrm{v},1} = 1.0$,
- $k_{\mathrm{v},2} = 1.0$, assumption as starting point for the factor to be investigated,
- $f_{\mathrm{v,mean,est}} = 9.4\ \mathrm{N/mm^2}$ [1],
- t: Thickness of the sheathing (nominal values)

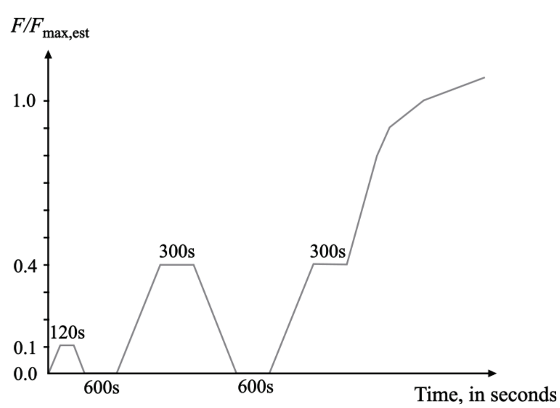

Figure 6: *ISO 21581 loading protocol applied in the timber-framed shear wall tests.*

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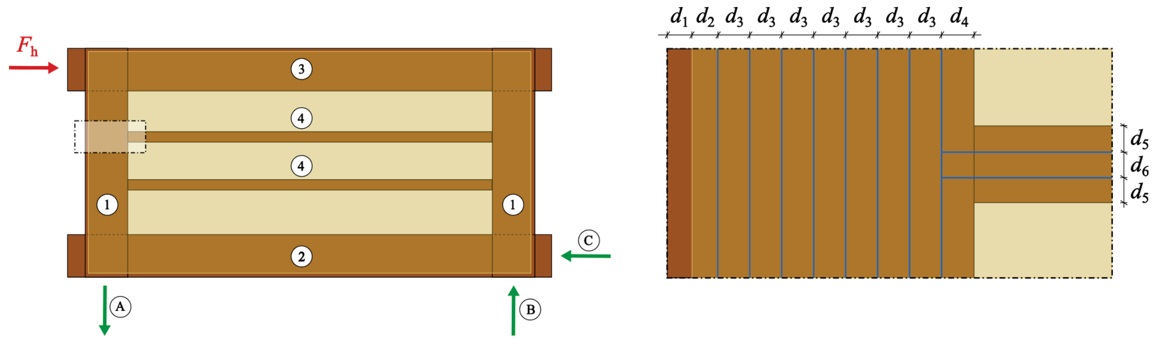

a) *b)*

***Figure** 7: a) Overview of the tested specimen with framing elements labelled with numbers (1: edge studs, 2: bottom rail, 3: top rail, 4: inner rails) and with reaction forces A, B, C induced by the applied force F_h. The dashed-dotted marked area is shown in Figure b). The blue lines indicate the rows, where the fasteners connecting the sheathing to the framing elements are placed.*

***Table 1**: Geometry of the four tested wall configurations. The distances d_1 to d_6 are illustrated in Figure 7, a_v describes the distance between two fasteners in the same row.*

			Wall configuration			
			12-S	**15-S**	**18-S**	**25-N**
Sheathing	Thickness	[mm]	12	15	18	25
Framing	Bottom and top rail (width x height)	[mm]	240 x 200	240 x 200	240 x 200	280 x 200
	Edge studs (width x height)	[mm]	240 x 200	240 x 200	240 x 200	280 x 200
	Inner rails (width x height)	[mm]	60 x 200	60 x 200	60 x 200	100 x 200
	Number of inner rails	[-]	2	2	2	1
Fasteners	Type	[-]	Staples	Staples	Staples	Nails
	Rows at the edges of the sheathing panels	[-]	8	8	8	10
	Rows on inner rails	[-]	2	2	2	2
Distances	d_1	[mm]	20	20	20	20
	d_2	[mm]	20	20	20	30
	d_3	[mm]	25	25	25	20
	d_4	[mm]	25	25	25	50
	d_5	[mm]	20	20	20	35
	d_6	[mm]	20	20	20	30
	a_v	[mm]	23	23	23	40

5.2.3 Realization in the lab

The force was applied centrically on the top rail of the specimen using a 1000 kN hydraulic jack (Figure 8 and Figure 9). Due to the non-perfect stiffness of the test rig, small test rig deformations in direction of the applied force could not be prevented. Since the effective deformation ($d_{\mathrm{max,rel}}$) of the timber-framed shear wall was determined in accordance with the procedure given in the standard ISO 21581 as the difference between the deformation in direction of the applied force on the top rail and the bottom rail, these small test rig deformations were neglected. The horizontal displacement in the centre of the bottom edge of the wall was measured using a mechanical displacement sensor with a measuring range of 10 mm and a precision of 0.04 mm, the horizontal displacement in the centre of the top edge of the wall was measured using a laser distance sensor with precision of 0.11 mm.

The anchoring of the vertical tensile and compressive forces and the resulting horizontal force is shown in Figure 9 and in Figure 10. Due to the asymmetry of the wall elements with sheathing on one side, out-of-plane forces developed during loading. To prevent tilting of the wall, a steel support was attached to the top rail of the wall and a 50 kN hydraulic jack was used to stabilize the wall (Figure 11). By forcing the stroke of this hydraulic jack to a displacement of 0 mm, tilting of the wall was avoided.

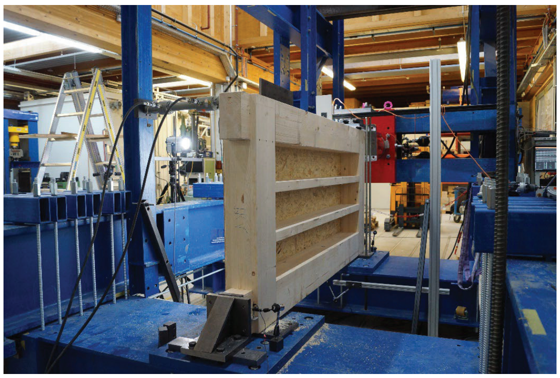

Figure 8: *Timber-framed shear wall specimen mounted in the test rig and ready for investigation of the shear resistance of the OSB/3 panels applied as sheathing material.*

Figure 10: *The horizontal reaction force was transferred to the test rig via a massive steel angle bracket. The vertical compressive force was transmitted through contact.*

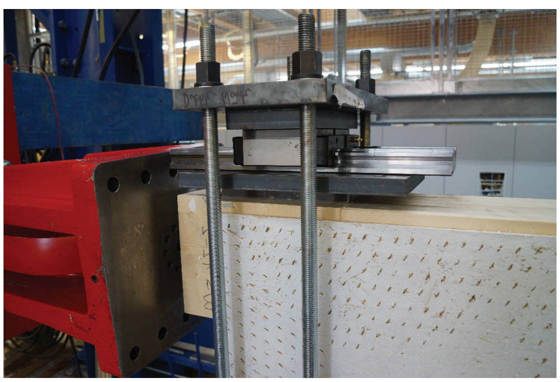

Figure 9: *The force was applied by the red hydraulic jack. The rail construction on top of the specimen is freely movable in horizontal direction. The tensile reaction forces were transmitted through the edge studs to the top edge of the specimen, where they were then transferred to a steel plate and brought back to the specimen via four steel rods.*

Figure 11: *Tilting of the specimen was prevented by means of a 50 kN hydraulic jack maintaining the out-of-plane displacement on the top rail at a value of 0 mm.*

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6 RESULTS AND DISCUSSION

6.1 TESTS ON OSB/3 PANELS

The results of the shear tests performed on the OSB/3 panels are presented in Table 2. The coefficients of variation (CoV) of the shear strength were in the range of 3.6% to 10.8%. The mean value of the shear strength of OSB/3 panels reported in the literature (9.4 $\mathrm{N/mm^2}$, [11]) was not confirmed by the results obtained from the tests conducted in this study. The 5% fractile values calculated from the test data according to the procedure in the European Standard EN 14358, 2016 [12] were lower than the value specified in EN 1995-1-1, 2008 and in the declaration of performance (6.8 N/mm^2) for all test series except for the 12 mm and 15 mm thick panels tested perpendicular to the strand orientation of the top layer. Investigations are planned to identify the causes of the insufficient shear strengths of the panels.

***Table 2**: Mean values, standard deviations and coefficients of variation (*CoV*) of the shear strength obtained from the tests on the OSB/3 panels. The 5% fractile values were calculated in accordance with the procedure given in EN 14358, 2016.*

	n	Mean	Stdev	CoV	5% fractile
	[-]	[N/mm^2]		[%]	[N/mm^2]
12, ⊥	5	7.69	0.28	3.6	6.79
15, ⊥	5	7.84	0.36	4.6	6.92
18, ⊥	5	7.61	0.41	5.4	6.63
25, ⊥	5	6.36	0.69	10.8	4.86
12, ǁ	5	6.97	0.38	5.5	6.08
15, ǁ	5	7.40	0.42	5.6	6.41
18, ǁ	5	7.28	0.46	6.3	6.21
25, ǁ	5	6.37	0.27	4.3	5.62

6.2 SHEAR WALL TESTS

In all the 12 tests performed on timber-framed shear wall elements, the sheathing failed in a brittle manner, in direction of the force flow, connecting the point of the applied force and the point where the horizontal force is transferred to the test rig. The initial failure occurred either along one of the fastener rows (Figure 12), or in diagonal direction in the field of the sheathing (Figure 13).

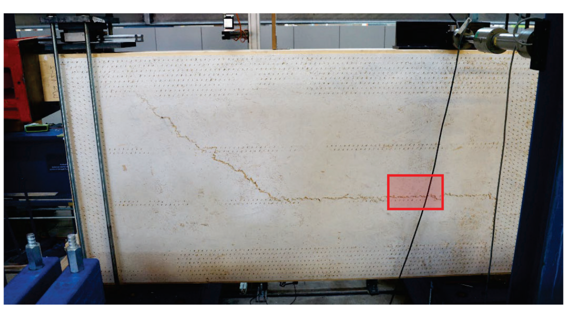

***Figure 12**: The initial failure in the 12-S-2 shear wall test occurred along the fastener row on the lower inner rail (marked in red).*

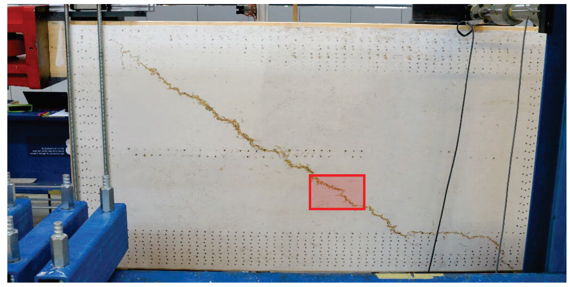

***Figure 13**: The initial failure of the 25-N-3 shear wall test occurred in diagonal direction in the field of the sheathing (marked in red).*

The applied maximum force and the respective maximum of the relative horizontal displacement ($d_{\mathrm{max,rel}}$, see 5.2.3) are listed in Table 3 for each of the specimens tested. Since the displacement and the resulting stiffness of the tested wall elements are important for the comparison of numerical calculations with test results, the displacements are given for the sake of completeness, without being further discussed in this paper.

***Table 3**: Applied maximal horizontal force $F_{\max}$ and respective maximal relative displacement $d_{\mathrm{max,rel}}$ at failure for each of the timber-framed shear wall specimens tested.*

	Specimen	$F_{\max}$ [kN]	$d_{\mathrm{max,rel}}$ [mm]
12-S	12-S-1	181	9.5
	12-S-2	186	10.1
	12-S-3	178	9.5
15-S	15-S-1	241	12.5
	15-S-2	221	10.6
	15-S-3	229	10.6
18-S	18-S-1	299	11.9
	18-S-2	290	11.7
	18-S-3	305	12.9
25-N	25-S-1	394	16.6
	25-S-2	405	16.3
	25-S-3	379	14.5

The $k_{\mathrm{v,2}}$ factor can be determined by equating the shear resistance of the OSB/3 panel in timber-framed shear walls estimated using the second line in Formula (2) multiplied by the length of the wall, and the maximum force $F_{\max}$ at failure determined in the experiments. Solving for $k_{\mathrm{v,2}}$ leads to Formula (3).

$$k_{\mathrm{v,2}} = \frac{F_{\max}}{F_{\mathrm{max,est}}} = \frac{F_{\max}}{k_{\mathrm{v,1}} \cdot f_{\mathrm{v,est,mean}} \cdot t \cdot l} \tag{3}$$

where $F_{\max}$ = applied force at failure, $k_{\mathrm{v,1}} = 1.0$, $f_{\mathrm{v,est,mean}}$ = estimated shear strength of the panel on mean level, t = nominal thickness of the sheathing, l = length of the wall.

The mean value of the shear strength of OSB/3 panels $f_{\mathrm{v,est,mean}}$ was estimated starting from the characteristic shear strength specified in EN 1995-1-1, 2008 (i.e. $f_{\mathrm{v,k}} = 6.8\ \mathrm{N/mm^2}$). Based on the assumption of a log-normal distribution, the mean value was calculated using the definition of the moments and the cumulative distribution function of the log-normal distribution (Formulae (4) - (6)).

$$\mu_{\mathrm{X}} = e^{\lambda+\zeta^2/2} \tag{4}$$

$$\mathrm{CoV_X} = \sqrt{e^{\zeta^2}-1} \tag{5}$$

$$F_{\mathrm{X}}(x) = \Phi\left(\frac{\ln(\mathrm{x})-\lambda}{\zeta}\right) \tag{6}$$

where ζ and λ = distribution parameters, μ_{X} = mean value of the population, $\mathrm{CoV_X}$ = coefficient of variation of the population.

In the JCSS Probabilistic Model Code [13], there is no information about the CoV of the shear strength of OSB/3 panels. In the present study, it was decided to use a CoV of 15% as the upper bound for the investigations, a value equal to the one specified in the JCSS Probabilistic Model Code for the shear strength of glued laminated timber (GLT). The mean CoV determined from the tests carried out to assess the shear strength of the OSB/3 panels was 6%. This low value can be explained by the fact that all panels tested were part of one single production batch. Hence, in the investigations, the 6% CoV was chosen as the lower bound. Additionally, calculations were also performed using a CoV of 11%. The $k_{\mathrm{v,2}}$ factors resulting from these calculations (i.e. from the comparison between estimates and experimental data) are listed in Table 4. In the presented study, the $k_{\mathrm{v,2}}$ factors for timber-framed shear walls sheathed with OSB/3 panels on one side were found to range between 0.69 and 0.88, depending on the thickness of the panels and the CoV outlined above accounted for in the calculation. The results show that the $k_{\mathrm{v,2}}$ factor of 0.33 given in DIN 1995-1-1, 2004-08 for sheathing placed on one side is very conservative. However, the specified value of 0.5 specified in the working draft of prEN 1995-1-1, 2022 matches the experimental results of this study better.

***Table 4**: $k_{\mathrm{v,2}}$ factors determined by applying Formula (3) and using the estimated shear strength $f_{\mathrm{v,est,mean}}$ evaluated by assuming different CoV's for the distribution of the shear strength of the OSB/3 panels.*

CoV*	$f_{\mathrm{v,est,mean}}$	$k_{\mathrm{v,2}}$			
		12-S	15-S	18-S	25-N
6%	7.52 N/mm^2	0.81	0.82	**0.88**	0.84
11%	8.19 N/mm^2	0.74	0.75	0.81	0.77
15%	8.78 N/mm^2	**0.69**	0.70	0.75	0.72

* CoV 6%: Lower bound value based on the mean value of the shear strength measured in the experiments on OSB/3 panels, CoV 15%: Upper bound value based on the CoV given for GLT in JCSS.

In addition to the influencing factors, which induce additional stresses as listed in DIN 1052, 2008-12 (see 2.2.3) and in prEN 1995-1-1, 2022 (see 2.2.4), there are additional influences, which may reduce the shear resistance of OSB/3 panels when applied as sheathing material in timber-framed shear walls. These factors include:

- Additional stresses resulting from eccentricities in the connection of the ceiling to the wall.
- Weakened sheathing due to the attachment of steel bracket angles as anchoring.

These influences were not considered in the experiments, and hence, also not considered in the calculated $k_{\mathrm{v,2}}$ factors in Table 4.

When investigating the maximum number of fasteners that can be applied while still ensuring a ductile failure of the connection, an over-strength factor of 1.6 was assumed [14]. By comparing the resistance of the sheathing-to-framing connection and the shear resistance of the panel (first and second line in Formula (2)) and using the over-strength factor 1.6, the minimum distance $a_{\mathrm{v,min}}$ between the fasteners can be calculated with different assumptions in terms of $k_{\mathrm{v,2}}$ factor with Formula (7):

$$a_{\mathrm{v.min}} = \frac{1.6 \cdot R_{\mathrm{d}}}{k_{\mathrm{v,2}} \cdot f_{\mathrm{v,d}} \cdot t} \tag{7}$$

where $k_{\mathrm{v,2}}$ = factor to account for additional stresses, $f_{\mathrm{v,d}}$ = design shear strength of the panel, t = thickness of the sheathing, R_{d} = design load-carrying capacity of one single fastener.

The characteristic load-carrying capacity $F_{\mathrm{v,Rk}}$ of a single nail and a single staple was calculated according to the standard EN 1995-1-1, 2008 (Clause 8.2.2, Equation 8.6). The design load-carrying capacity R_{d} according to EN 1995-1-1, 2008 is $R_d = k_{\mathrm{mod}} \cdot F_{\mathrm{v,Rk}}/\gamma_{\mathrm{M}}$, where k_{mod} and γ_{M} are 1.1 and 1.2, respectively, according to EN 1995-1-1, 2008. For a single nail, the design load-carrying capacity R_{d} is equal to 0.66 kN and for a single staple the respective value is 0.52 kN.

Taking the data and $k_{\mathrm{v,2}}$ factors listed in Table 4 and applying Formula (7), the minimum fastener distances were calculated for $k_{\mathrm{v,2}}$ values between 0.55 and 0.75. The resulting minimum distances $a_{\mathrm{v,min}}$ are listed in Table 5.

***Table 5**: Resulting minimal fastener distances $a_{\mathrm{v,min}}$ for the four investigated sheathing thicknesses (12 mm, 15 mm, 18 mm and 25 mm) assuming different $k_{\mathrm{v,2}}$ factors.*

$k_{\mathrm{v,2}}$	$a_{\mathrm{v,min}}$			
	12 mm	**15 mm**	18 mm	25 mm
0.60	18.6	**14.8**	12.4	11.2
0.65	17.1	13.7	11.4	10.4
0.70	15.9	12.7	10.6	9.6
0.75	14.8	11.9	9.9	9.0
0.80	13.9	11.1	9.3	8.4

For example, for a 15 mm thick sheathing and assuming a conservative value of 0.6 for the $k_{\mathrm{v,2}}$ factor, based on the results listed in Table 4, the minimum $a_{\mathrm{v,min}}$ distance between the staples was calculated to be 14.8 mm (→ 15 mm). When arranging two rows of staples, the minimum distance between the staples would be 29.6 mm (→ 30 mm). According to the Swiss standard SIA 265/1,

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2018 [15] the minimum distances of the connection need to be increased by a factor of 1.5 to ensure a ductile behavior. The minimum distance between the staples in Switzerland would be 34.4 mm (→ 35 mm)

7 CONCLUSIONS AND OUTLOOK

The factor $k_{v,2}$ specified in DIN 1052, 2008-12 (0.33 for sheathings placed on one side), which accounts for additional stresses that are reducing the shear resistance of the sheathing in timber-framed shear walls seems to be quite conservative, compared to the results from the experiments conducted in the presented study. The values for $k_{v,2}$ resulting from the experiments range between 0.69 and 0.88 depending on the estimated mean value $f_{v,est,mean}$ of the shear strength of the panels. The value for $k_{v,2}$ (renamed to $k_{p,model}$ in prEN 1995-1-1, 2022) matches the experimental results of this study better.

The test results indicate that by specifying reasonable minimum distances a_v between the fasteners (i.e. for instance 30 mm for two rows of staples in a 15 mm thick OSB/3 sheathing and choosing a $k_{v,2}$ factor of 0.60), multiple rows of fasteners can be planned while ensuring a ductile failure of the fasteners and avoiding a brittle failure of the sheathing.

Further investigations (i.e. additional experiments accompanied by numerical analyses) are required to determine the accurate value of the $k_{v,2}$ factor used in the calculation of the shear resistance of OSB/3 sheathings in timber-framed shear walls. The study presented here only investigated wall elements with OSB/3 sheathing placed on one side of the wall. Additional experiments recently performed on OSB/3 sheathed timber-framed shear walls can be taken into consideration to determine the values of the $k_{v,2}$ factor more reliably. An increase of this factor compared to the values currently specified in prEN 1995-1-1, 2022 (i.e. 0.50 for sheathings on one side and 0.67 for sheathings on both sides) would lead to an increased load-carrying capacity of timber-framed shear walls.

This study represents the first step of a series of investigations in a research project where further experiments will be performed on one- and two-storey wall elements with large openings, with the ultimate goal of developing a design method for timber-framed shear walls with openings as part of the lateral force-resisting system in timber buildings. The use of timber-framed shear walls with openings in the lateral force-resisting system has the potential to provide more economical design solutions in timber construction.

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REFERENCES

[1] SIA 265: Holzbau, Swiss Society of Engineers and Architects SIA, Zurich, Switzerland, 2021.*

[2] DIN 1052: Design of timber structures. General rules and rules for buildings, Germany, 2008-12.

[3] EN 1995-1-1: Eurocode 5 – Design of timber structures - Part 1-1: General - Common rules and rules for buildings, CEN, Brussels, Belgium, 2008.

[4] https://www.schaerholzbau.ch/

[5] prEN 1995-1-1: CEN/TC 250/SC 5 N 1650, Eurocode 5 – Design of timber structures, CEN, Brussels, Belgium, 2022.

[6] EN 322: Wood-based panels - Determination of moisture content, CEN, Brussels, Belgium, 1993.

[7] EN 300: Oriented Strand Boards (OSB) - Definitions, classification and specifications, CEN, Brussels, Belgium, 2006.

[8] EN 14080: Timber structures - Glued laminated timber and glued laminated solid timber - Requirements, CEN, Brussels, Belgium, 2013.

[9] EN 789: Timber structures - Test methods - Determination of mechanical properties of wood based panels, CEN, Brussels, Belgium, 2005.

[10] ISO 21581: Timber structures - Static and cyclic lateral load test methods for shear walls, ISO, Geneva, Switzerland, 2010.

[11] Schick M., Probabilistic investigations on overstrengths of nailed timber-framed wall elements, Kassel, Germany, 2017. *

[12] EN 14358: Timber structures - Calculation and verification of characteristic values, CEN, Brussels, Belgium, 2016.

[13] JCSS 3.05: Probabilistic Model Code - Part 3.05: Resistance Models: Timber, 2006.

[14] Ratsch G., Geiser M., Erdbebenbemessung von Holztragwerken nach dem duktilen Tragwerksverhalten – Die schwierige Frage der Überfestigkeit, In: 16th *D-A-CH Tagung Erdbebeningenieurwesen & Baudynamik*, 2019. *

[15] SIA 265/1: Holzbau – Ergänzende Festlegungen, Swiss Society of Engineers and Architects SIA, Zurich, Switzerland, 2018.*

* Only available in German