World Conference on Timber Engineering
Oslo 2023

NUMERICAL MODELLING OF LIGHT TIMBER FRAME WALLS – COMPARATIVE STUDY OF THREE FASTENER REPRESENTATIONS

Lukas Rauber[1], Benno Hoffmeister[2]

ABSTRACT: The lateral load-bearing capacity and stiffness of light timber frame walls (LTFW) depend on the mechanical properties and interaction of their components (frame, sheathing, fasteners and anchorage). The fasteners are decisive for the structural performance (resistance, stiffness) of LTFW. Therefore, numerical modelling focuses on the representation of the fasteners. In this paper, different modelling approaches for fasteners of LTFW are presented and compared with regard to applicability in research and practice – (i) an engineering model with line releases; (ii) a model with oriented springs and (iii) a model with beam elements. The models were validated and compared to experimental results from tests conducted at RWTH Aachen. All models represent the load-bearing capacity closer than the analytical approach of Eurocode 5. The model with line releases is recommended for engineering while the model with oriented springs is recommended for research. The modelling of GFB sheathing needs further investigation.

KEYWORDS: timber frame walls, shear walls, numerical modelling, push-over tests, load-bearing capacity, stiffness

1 INTRODUCTION

Light Timber Frame construction is a resource-efficient construction method with a high degree of prefabrication and short construction times. Light Timber Frame Walls (LTFW) consist of several components (frame, sheathing, fasteners). Frame and sheathing provide a load-bearing element when fastened in shear. The load-bearing behaviour is usually governed by the shear-connection between sheathing and frame as well as by the anchoring or the connection to adjacent building components.

In Europe, the horizontal load-bearing capacity and the displacement of LTFW are calculated according to the "pure shear" model ("Method A") of *Eurocode 5* [1]. This model is based on the lower bound theorem of plasticity theory and assumes that all fasteners are loaded with the same force and parallel to the sheathing edges. In addition, detailing provisions are given to ensure that yielding of the fasteners governs the failure mode of the diaphragm.
In *Eurocode 5* areas containing large openings, like windows or doors, are neglected in the determination of the load-bearing behaviour. It is assumed that wall sections with large openings do not contribute to the load-bearing capacity of the wall [1].

Eurocode 5 does not contain explicit provisions for the calculation of the horizontal stiffness of LTFW [1]. Engineers may calculate the deformation according to the "pure shear" model, by determining the contributions of the individual components – fastener deformation, shear deformation of the sheathing, axial displacement of the frame parts, deformation perpendicular to the grain of the bottom rail and deformation of the anchorage [2].

Test results [3–5] show that prediction of the load-bearing behaviour of LTFW using the "pure shear" model of *Eurocode 5* leads to very conservative results .
For seismic actions LTFW need to be verified according to seismic codes, e.g. *Eurocode 8* [6]. The seismic resistance may be determined assuming ductile and dissipative behaviour represented by a behaviour factor. The application of a behaviour factor of $q > 1.5$ assumes ductile fastener behaviour and requires prevention of brittle failure of other structural wall components. The condition for the achievement of ductile behaviour is the application of so-called capacity design rules. An accurate estimation of the load-bearing behaviour of all structural components is therefore essential for safe seismic design. Due to the lack of adequate provisions for calculating the realistic load-bearing behaviour of LTFW in *Eurocode 5*, non-linear numerical simulations are good alternatives for the calculation of load-bearing capacity and stiffness.

The objective of this contribution is to assess three promising numerical models regarding their applicability in research and engineering. The requirements according to which the models were preselected are the following:

- The model should be able to calculate the load-bearing capacity and the stiffness of LTFW and give more accurate results than the analytical model of *Eurocode 5*.
- The model output can be used as basis for the non-linear analyses of timber frame buildings.
- The model should have reasonable computing time for the application in engineering.

[1] Lukas Rauber, RWTH Aachen University, Germany, Institute of Steel Construction, l.rauber@stb.rwth-aachen.de
[2] Benno Hoffmeister, RWTH Aachen University, Germany, Institute of Steel Construction, hoff@stb.rwth-aachen.de

https://doi.org/10.52202/069179-0309

2 NUMERICAL MODELS

The reference structure used with the three presented numerical models is shown in Figure 1. For the frame all models use beam elements with linear-elastic material properties. The studs were hinged to the rails – this is common practice for numerical models of LTFW; whereas in the "pure shear" model of *Eurocode 5* studs and rails are considered contactless. The sheathing was modelled by means of linear-elastic shell elements. The anchorage was represented as a linear-elastic spring.

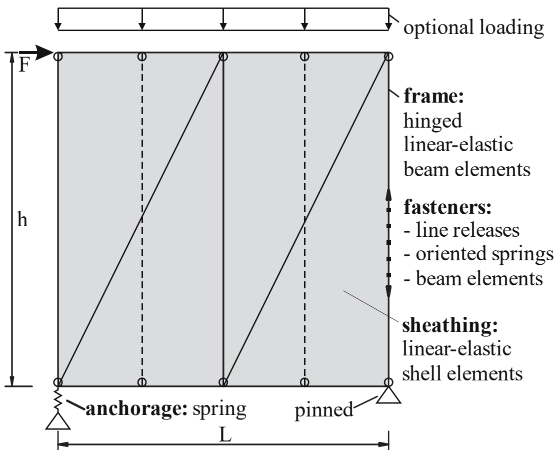


Figure 1: *Basic structure of the numerical models*

The three models differ in the representation of the non-linear fastener behaviour as seen in Figure 2. In model 1 (M1) the connection was modelled using "line releases". Model 2 (M2) used oriented springs to represent the connection between frame and sheathing. In model 3 (M3) flexible beam elements in bending were used according to *Vogt* [3].

M2 (oriented springs) and M3 (Vogt) were implemented in ABAQUS 2019 with "B31" beam elements for the frame and "S4R" shell elements for the sheathing. M1 (line releases) was realized in Dlubal RFEM 5.26 which uses automatic mesh generation.

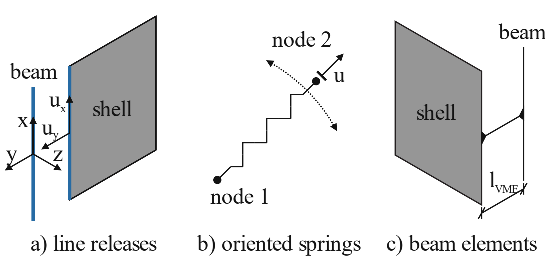


Figure 2: *Different fastener representations of the models*

2.1 LINE RELEASES (M1)

M1 used continuous cartesian spring elements for the representation of the connection between sheathing and frame. RFEM was chosen for the implementation of the model because of the availability and the simple applicability of continuous springs – called "line releases".

The input for line releases is defined as load-displacement per unit length. As shown in Figure 2 a, load-displacement behaviour is represented by continuous springs with degrees of freedom parallel and perpendicular to the sheathing edge. Since cartesian springs only act on their predefined axes, the resulting load-bearing capacity and stiffness are overestimated for oblique load directions [3]. Herein, both spring directions used non-linear curves from tests on connection units as input values as described in chapter 2.4 (0° for u_x and 90° for u_y).

The wall models were solved according to the theory of the third order with large displacements.

2.2 ORIENTED SPRINGS (M2)

M2 used non-linear oriented springs (Figure 2 b) for the representation of the individual fasteners. The model was implemented in ABAQUS.

For modelling the oriented springs "connectors" between sheathing and frame nodes were used with the translational type "axial". With this definition the springs acted in direction of their individual deformation. For modelling the non-linear fastener behaviour, input curves from tests on connection units were used (chapter 2.4).

"Static general" with large displacements (*NLgeom on*) was used as solving method.

2.3 BEAM ELEMENTS (M3)

M3 was also implemented in ABAQUS and used the same elements for frame and sheathing as M2. To model the fastener behaviour, beam elements as described by *Vogt* [3] were used (Figure 2 c).

The fastener elements (*B33*) were connected as "fixed" to frame and sheathing. They used elastic-plastic material behaviour and circular cross-sections to exhibit the same load-bearing behaviour in each load direction.

The fasteners remained elastic until plastic hinges occurred on both ends after the yield point of the material was reached.

To represent the load-bearing capacity of the connection F_f the length of the beam elements was calculated with:

$$l_{VME} = \frac{2 \cdot M_{pl}}{F_f} \tag{1}$$

To represent the stiffness of the connection *K*, a factor s_l was used for reducing *EI* in the numerical definition [3].

The input values for the model presented by *Vogt* [3] used the shear strength $F_{f,Rk}$ and stiffness K_{ser} calculated according to *Eurocode 5*.

For this paper mean values derived from connection tests were used for F_f while *K* was calculated as K_{ser} according to *Eurocode 5* [1].

2.4 INPUT DATA

The input data for the modelling of the fasteners' plastic behaviour was derived from connection tests conducted at RWTH Aachen [4] and the Optimberquake project [7].

At RWTH a test program with a total of 78 connection units was conducted in accordance with *ISO 6891* [8] for the monotonic tests used in this contribution.

The test setup is shown in Figure 3. Structural timber grade C24 was used for the frame part.
Sheathing materials, staple diameters and force grain angles (0° and 90°) were varied. For the validation in this paper only monotonic tests with OSB/3 sheathing (t =15 mm) and staples of diameter 1.8 mm and a length of 65 mm were used.

Figure 3: *Test setup of connection units at RWTH with load acting 0° (left) and 90° (right) to the timber grain*

For the validation with walls tested in the Optimberquake [9] project, tests on connection units [7] performed in this project were used. Relevant for the validation were monotonic tests on OSB/3 sheathing with nails of diameter 2.8 mm and a length of 65 mm. Also relevant were monotonic tests on gypsum fibre board (GFB) with a thickness of 18 mm and staples (⌀ 1.53 mm and length 55 mm).

The non-linear input curves per fastener were derived directly from the test results and used in tabular form for defining fastener load-deformation properties of M1 (line releases) and M2 (oriented springs).
The input values used for the models and the calculation according to *Eurocode 5* are shown in Table 1. M1 and M2 used nonlinear input curves generated from tests on connection units (Figure 4). M3 used mean values of F_f derived from the connection tests. The analytical calculations according to *Eurocode 5* were performed with both mean (m) and characteristic (k) values of F_f.
The E-moduli were set to 11000 N/mm² for the frame components and 380 N/mm² for the sheathing.
In order to simulate the tests at RWTH Aachen the anchorage was simplified by pinned supports. For the Optimberquake walls the spring stiffness of the anchorage was set to 11150 N/mm (from *Vogt* [3]).
The process of the generation of input curves is described in Figure 4. First the mean curves of the monotonic tests were generated per connection variant. Afterwards the curve was scaled down to one fastener for the input of M2 (oriented springs). For M1 (line releases) the input curves still had to be divided by the fastener spacing of the modelled wall to obtain to the load-displacement curves per unit length.

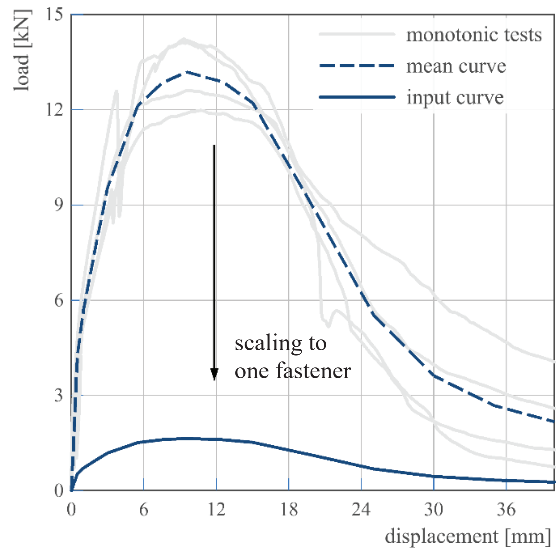


Figure 4: *Generating the fastener input curves from test results*

3 VALIDATION

3.1 TESTS ON WALL ELEMENTS

To validate the models, results of wall tests conducted at RWTH Aachen [4] and the Optimberquake project [9] were used. The tests for the validation are listed in Table 2. All tests were conducted with the monotonic loading protocol according to *ISO 21581* [10].

The experimental investigations conducted at RWTH include tests on standard reference walls as well as newly developed triple-sheathing walls. A total of 12 full scale wall tests were performed at RWTH. Four tests were conducted with and eight without additional vertical loading. In Figure 5 the test setup for a LTFW with vertical loading is shown. A vertical load of 37 kN was applied on each stud (111 kN in total) – representing loads of a four-story building.
All test specimens consisted of C24 frames and OSB/3 sheathing with a thickness of 15 mm which was fastened using resin coated staples (⌀ 1.80 mm and length 65 mm) with a spacing of 75 mm. Specimens with aspect ratios of 1.25 × 2.50 m and 2.50 × 2.50 m were tested.

Table 1: *Input values of the parameters used for the validation of the numerical models for the different tests (see Table 2)*

Test	Fastener parameter		M1 (LR)	M2 (OS)	M3 (Vogt)	EC 5 (m)	EC 5 (k)
h-m-xx	K	[N/mm]	non-linear curve from		450	450	450
	F_f	[N]	test series OSB15-st1.80		1609.5	1609.5	908.5
WL-1.1	K	[N/mm]	non-linear curve from		859.5	859.5	859.5
	F_f	[N]	test series na2.8-o18-m		1071	1071	819
WL-2.1	K	[N/mm]	non-linear curve from		643.7	643.7	643.7
	F_f	[N]	test st1.53-g18-m-1		1225	1225	682.7

https://doi.org/10.52202/069179-0309

Table 2: Test data of timber frame walls used for the validation of the numerical models

Test	L × h [m]	Sheathing t [mm]	Fasteners ⌀ - length - spacing [mm]	Loading	Project
h-m-02/03	1.25 × 2.50	2 × OSB 15	Staples 1.80 - 65 - 75	horizontal	RWTH [4]
h-m-04	1.25 ×2.50	2 × OSB 15	Staples 1.80 - 65 - 75	horizontal + vertical	
h-m-11	1.25 × 2.50	3 × OSB 15	Staples 1.80 - 65 - 75	horizontal	
h-m-12	2.50 × 2.50	3 × OSB 15	Staples 1.80 - 65 - 75	horizontal	
h-m-13	1.25 × 2.50	3 × OSB 15	Staples 1.80 - 65 - 75	horizontal + vertical	
WL-1.1	2.50 × 2.50	2 × OSB 18	Nails 2.80 - 65 - 75	horizontal + vertical	Optimber-quake [9]
WL-2.1	2.50 × 2.50	2 × GFB 18	Staples 1.53 - 55 - 75	horizontal + vertical	

The anchorage was designed as non-dissipative including sufficient overstrength. The “shear response” according to *Dujic* [11] was ensured for all tests. For this purpose, two anchorages of the type HTT31 of Simpson Strong-Tie as well as horizontal cleats were used at each support.

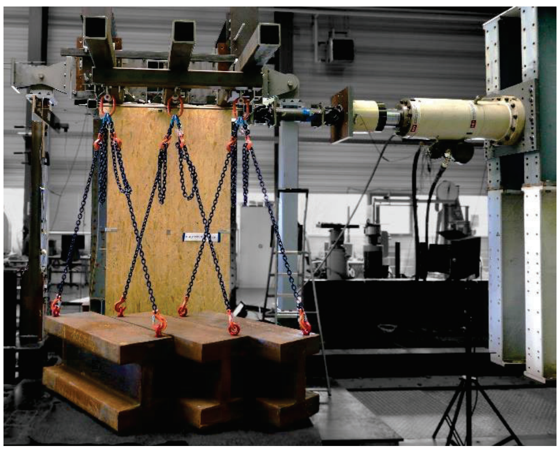


Figure 5: *Test setup for LTFW with vertical loading*

Further test results on LTFW elements from the Optimberquake program [9] were used for validation. Two test setups were selected: (i) specimens with 18 mm OSB sheathing and nails (⌀ 2.80 mm and length 65 mm) and (ii) specimens with 18 mm GFB sheathing and staples (⌀ 1.53 mm and length 55 mm).

3.2 NUMERICAL RESULTS

Figure 6 shows the comparison of the load-bearing capacities (F_{max}) from tests, by numerical models and according to *Eurocode 5* calculated with characteristic (k) and mean test (m) input values.

For M1 and M2 maximum values were used to determine the load-bearing capacity while M3 (Vogt) was evaluated at 60 mm displacement in this paper, since this model does not include decreasing of resistance (Figure 7).

The load-bearing capacity was underestimated for all OSB sheathed test specimens (*h-m-xx* and *WL-1.1*), by the models and by *Eurocode 5* design rules.

F_{max} of the GFB sheathed specimen (*WL-2.1*) was overestimated by all models and *Eurocode 5* when using test data from connection units (*m*).

Comparable tests without (*h-m-02* and *h-m-11*) and tests with vertical loads (*h-m-04* and *h-m-13*) showed no significant difference for F_{max}. The same was the case for the numerical models and the analytical calculations.

The load-displacement curves of the models are compared to the test results of *h-m-02* and *h-m-03* and shown in Figure 7 (left).

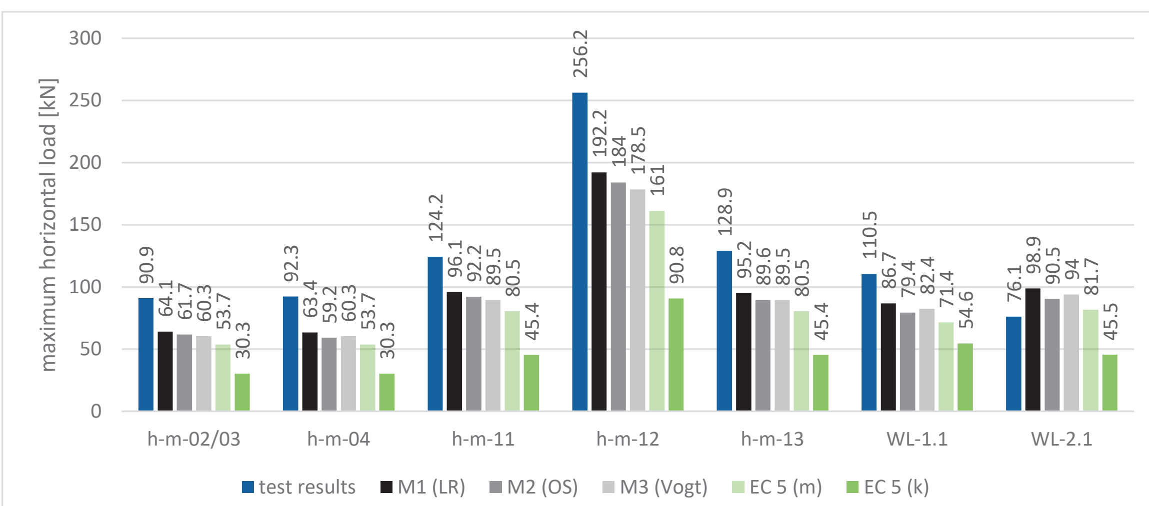


Figure 6: *Bar chart comparison of the load-bearing capacity between mean test results, numerical models and analytical calculation*

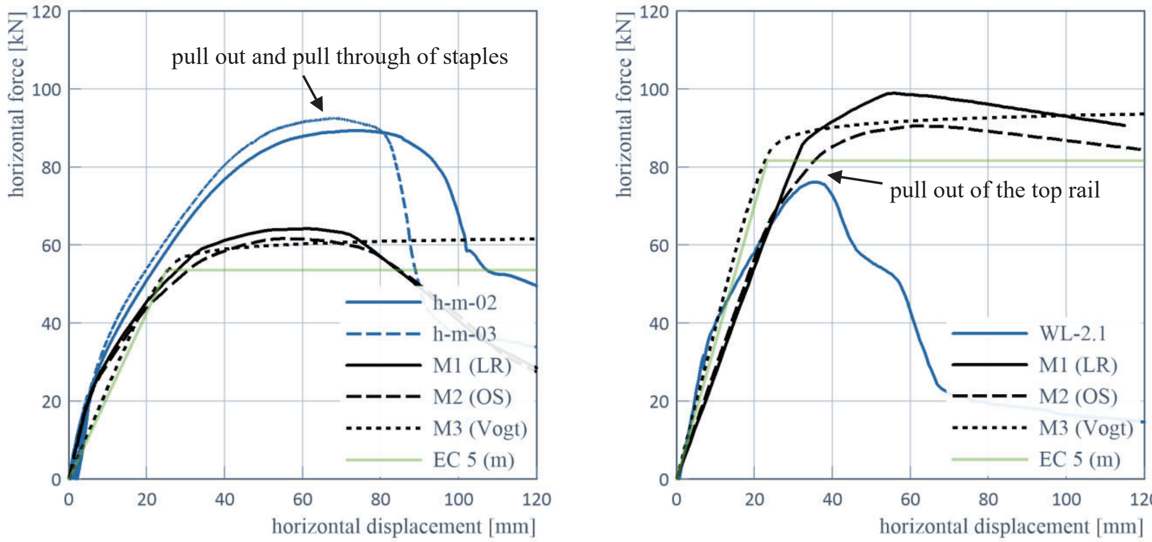


Figure 7: *Load displacement comparison of tests vs models for walls made of OSB with staples (left) and of GFB with staples (right)*

M1 (line releases) and M2 (oriented springs) resembled the scaled non-linear input curves for fasteners (Figure 4) while M3 (beam elements) showed almost ideal elastic-plastic behaviour.

As seen before, all models underestimated the load-bearing capacity of the tests. This model behaviour looked similar for all comparisons to tests *h-m-xx* (OSB and staples) and *WL-1.1* (OSB and nails).

In Figure 7 (right) the load-displacement diagram of specimen *WL-2.1* is shown – the only test with GFB sheathing. F_{max} was overestimated by all models compared to the test result. The initial stiffness was underestimated with M1 and M2 while M3 was closer to the initial stiffness of the test curve.

For the evaluation of the initial stiffness of the tests as well as the model results the calculation of K according to *ISO 21581* [10] was carried out with:

$$K = \frac{0.3\, F_{max}}{u_{40\%F_{max}} - u_{10\%F_{max}}} \tag{2}$$

The comparison of the evaluated initial stiffnesses of wall tests, models and analytical calculations (according to [2] but also named EC 5) is shown in Figure 8.

For the test series *h-m-xx* (OSB and staples) the following order applies to the stiffness: M1 is the stiffest, followed by M2, M3 and *Eurocode 5*.

Tests on nominally equal walls without (*h-m-02* and *h-m-11*) and tests with vertical loads (*h-m-04* and *h-m-13*) showed no significant difference regarding the initial stiffness. This was also observed for the numerical models and the analytical calculations.

The mean average percentage error (*MAPE*) was used for the comparison between models and test results and was calculated for F_{max} and the initial stiffness by:

$$MAPE = \frac{100\%}{n} \sum_{i=1}^{n} \left| \frac{A_i - F_i}{A_i} \right| \tag{3}$$

with A_i as the test value and F_i as the model value.

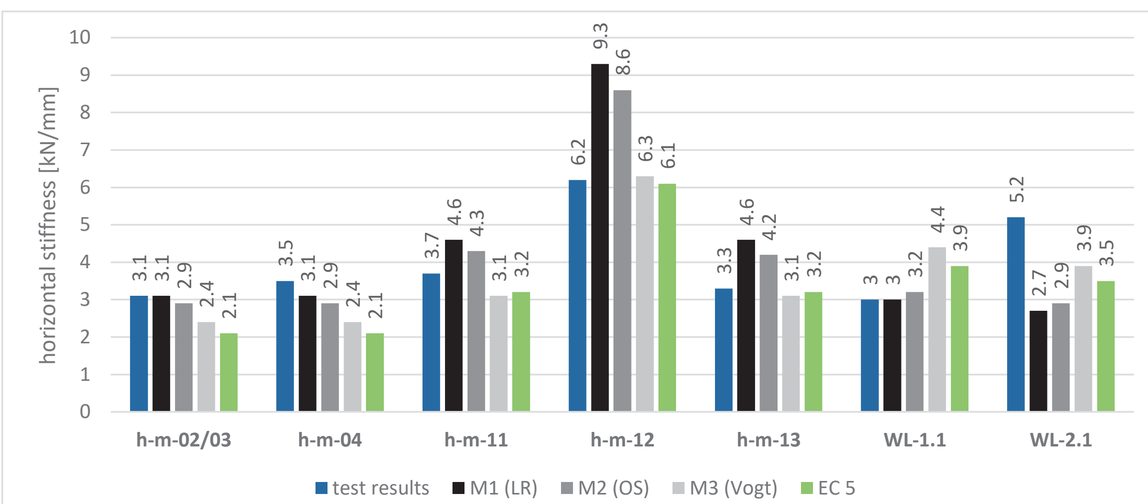
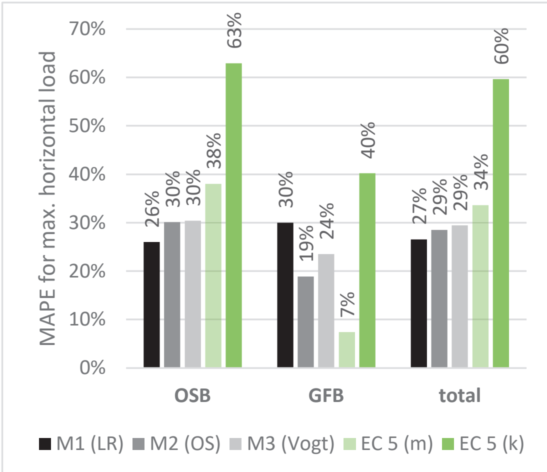


Figure 8: *Bar chart comparison of the stiffness between mean test results, numerical models and analytical calculation*

https://doi.org/10.52202/069179-0309



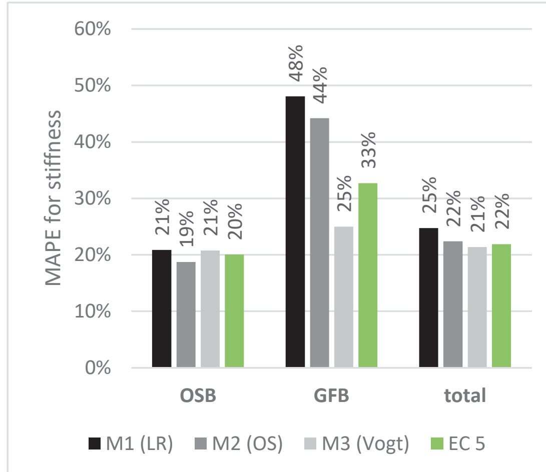


Figure 9: *Comparison of the MAPE of models and Eurocode 5 to test results for F_{max} (left) and for stiffness (right)*

The comparison of *MAPE* between the different models and *Eurocode 5* was evaluated for OSB sheathing, GFB sheathing and the total of all tests. The results are shown in Figure 9 for F_{max} (left) and stiffness (right).
M1 showed the least total error with regard to F_{max}, followed by M2, M3 and EC 5 (mean). For the test with GFB sheathing M1 had the biggest model error, followed by M3, M2 and EC 5 (mean).
With regard to stiffness M1 showed the greatest total error, followed by M2, EC 5 and M3. For OSB sheathing the error is approximately 20% for all models.

3.3 DISCUSSION

The presented numerical models showed a better representation of the load-bearing capacity than the analytical "pure shear" model of *Eurocode 5*.
However, all models show large deviations from the load-bearing capacity as obtained by tests. The differences were significantly higher than expected.

For OSB sheathing the load-bearing capacity was strongly underestimated by all models. This might be due to friction effects between sheathing and frame and/or scale effects of the fastener behaviour – in particular the behaviour of large groups compared to few fasteners in the connection tests – which both might not be represented by the conducted tests on connection units (Figure 3).
For GFB sheathing the load-bearing capacity was overestimated by all numerical models. This might be due to the limited test data from GFB tests – only one connection test and one wall test were used here. Nevertheless, the models' overestimation of load-bearing capacity for GFB is a safety risk for the calculation with these methods. More tests with GFB are needed to calibrate and validate the models.
With regard to stiffness estimation all model results are only as good as the calculation with the model in *Eurocode 5* (calculated with [2]) for OSB test specimens.

In the following the known abilities and limitations of the models are listed and related to the models (named in parentheses).

Abilities

- Calculate the load-bearing capacity and stiffness more accurately than *Eurocode 5* for OSB (all).
- Failure of other wall components (studs, anchorage, sheathing) may also be assessed (all).
- Resulting load-displacement curves may be used as basis for simplified full scale building models (all).
- Wall parts with large openings due to windows and doors can be included in the calculation (all).

Limitations

- Model relies on test results – ideally non-linear test curves – of connection units for best results (all).
- Not applicable for huge and complex engineering models of buildings due to time consuming modelling and long computation time (M2, M3).
- Overestimation of oblique fastener loads and stiffness by using cartesian spring pairs (M1).
- Sheathing failure (e.g. shear and tension) cannot be represented by the models if using linear-elastic material representation. This can lead to overestimation of the load-bearing capacity when sheathing failure occurs for the LTFW (all).
- Seismic behaviour cannot be derived directly because the models' lack the representation of damage (strength reduction) due to cyclic loading (all).

The limitations might be overcome by the following modifications:

- Instead of test data, characteristic values could also be used as model input for safe-sided estimates leading to less model error than *Eurocode 5*.
- M1 could be used in engineering applications, as it requires low computing times and gives reasonable results. The overestimation due to the cartesian spring pair could be overcome by using just the main load direction of the fasteners (u_x) or reducing the second input values (u_y).
- Sheathing stress could be assessed continuously in the models to determine premature sheathing failure.

4 CONCLUSIONS

Three different numerical approaches of fastener representation for LTFW were presented – (i) an engineering model with line releases; (ii) a model with oriented springs and (iii) a model with beam elements.

The comparison of the test results with numerical models demonstrates the difficult venture of modelling load-bearing behaviour of LTFW. The transfer of experimental data from connections to full-scale walls does not lead to the expected (realistic) results.

The results indicate that a scale effect between connection tests to LTFW tests exists. Further research is needed in this regard.

Nevertheless, all models represent the load-bearing capacity of OSB sheathing better than the "pure shear" model given in *Eurocode 5*.

The following conclusions can be drawn with regard to load-bearing capacity:

- All models underestimated the maximum load for the OSB sheathed specimens.
- All models overestimate the maximum load for the GFB specimen. As long as the models cannot represent the load-bearing capacity of GFB sheathed walls, wall tests are recommended.
- Tests and models with and without vertical loading show no significant difference regarding lateral performance.

The following conclusions can be drawn with regard to stiffness:

- The numerical models represent the stiffness of OSB specimens as good as the "pure shear" model.
- No influence from vertical loading is observed for both tests and numerical models.

Model 1 (line releases) shall be particularly interesting for engineers, as it is simple, quick to apply and contains all wall components required for capacity design. However, the effects due to usage of the cartesian spring pair need to be considered.

Model 2 (oriented springs) is suggested for the application in research as it gives valuable output about the individual fastener forces and displacements. Model 3 leads to better estimations than the analytical model of *Eurocode 5* but showed high computational demand and modelling times.

Model 1 (line releases) and model 2 (oriented springs) might be especially interesting for nonlinear pushover analyses of timber frame buildings. Although results are not as good as individual wall tests the models are a less expensive alternative to testing. Both models might also be used to estimate the ductility of LTFW with data from tests on connection units only, but this still needs to be investigated. If the shown model limitations could be overcome, the number of wall tests required to validate the performance would become significantly less in the future.

ACKNOWLEDGEMENT

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.

REFERENCES

[1] EN 1995-1-1: Eurocode 5: Design of timber structures - Part 1-1: General – Common rules and rules for buildings, 2004.

[2] Blaß H. J., Ehlbeck J., Kreuzinger H., Steck G.: Erläuterungen zu DIN 1052: 2004-08: Entwurf, Berechnung und Bemessung von Holzbauwerken. 1st ed. München, 2004.

[3] Vogt T., Seim W.: Advanced modelling of timber-framed wall elements for application in engineering practice. INTER, Bath, 2014.

[4] Wilden V., Hoffmeister B.: Experimental analyses of innovative wood-shear walls under seismic loads. IABSE Congress, 2021.

[5] Schick M., Vogt T., Seim W.: Connections and anchoring for wall and slab elements in seismic design. CIB-W18/46-15-4, 2013.

[6] EN 1998-1: Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, 2009.

[7] Seim W., Vogt T.: OptimberQuake - Deliverable 2A: Connection units – monotonic and cyclic testing. Kassel, 2013.

[8] ISO 6891: Timber structures - Joints made with mechanical fasteners -· General principles for the determination of strength and deformation characteristics, 1983.

[9] Seim W., Vogt T.: OptimberQuake - Deliverable 2B: Timber framed wall elements – monotonic and cyclic testing. Kassel, 2013.

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

[11] Dujic B., Aicher S., Zarnič R.: Investigations on in-plane loaded wooden elements: influence of loading and boundary conditions. Otto-Graf-Journal 200516:259–72.

https://doi.org/10.52202/069179-0309