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EXPERIMENTAL AND NUMERICAL INVESTIGATION OF INNOVATIVE DUCTILE AND REPLACEABLE ANCHORING SYSTEMS FOR WOOD SHEAR WALLS UNDER SEISMIC LOADS

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Abstract

Recently, timber has gained increasing importance as a structural material, as it offers competitive structural performance and efficient production combined with a significant reduction of the environmental impact. Light-weight timber frames provide an efficient structural solution for wooden multi-storey buildings, in which the wall elements - acting as diaphragms - provide lateral stiffness and resistance to wind and earthquake loads. Light-weight timber frame elements are built up of several components (sheeting, fasteners, frame, support and anchorages), each of them contributing to the total performance of the structure. The current paper presents the outcome of experimental and numerical investigations on a ductile and replaceable supporting and anchoring system for wood shear walls. The aim of the examined novel system is two-fold: (i) to limit the damage of the superstructure under seismic loads by concentrating plastic deformations on the anchoring elements, placed at the base of the building and (ii) to be replaced after getting damaged because of a significant earthquake. The supporting and anchoring detail is made of steel and designed appropriately to behave nonlinear in a ductile manner under severe seismic loads and thus to provide additional energy dissipation. The damage is concentrated on the steel part, which acts as a dissipative mechanism limiting the forces transferred into the timber building. Consequently, the timber members remain undamaged for moderate earthquakes and slightly damaged in case of strong seismic events. A parametric study, using FE models validated based on experimental results, was conducted with the objective to investigate the influence of the geometrical characteristics of the anchorage. Moreover, a built-up anchoring configuration optimized for replaceability was developed and analyzed. Finally, recommendations for applications are provided, as well as the main conclusions of the study.

Keywords: timber shear walls, ductile and replaceable anchoring system, experimental tests, FE-investigation

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1 INTRODUCTION

Recently, timber has gained increasing importance as a structural material, as it offers competitive structural performance and efficient production combined with a significant reduction of the environmental impact. Light-weight timber frames provide an efficient structural solution for wooden multi-storey buildings, in which the wall elements - acting as diaphragms - provide lateral stiffness and resistance to wind and earthquake loads. Light-weight timber frame elements are built up of several components (sheeting, fasteners, frame, support and anchorages); each of them contributes to the total performance of the structure.

Rising demand for multi-storey timber buildings leads to new challenges regarding stiffness, strength and ductility - properties responsible for the seismic performance. On the one hand, because of their relatively low structural mass, timber buildings are exposed to lower seismic loads compared to other structural materials. On the other hand, the anisotropic and naturally scattering characteristics of wood material settle the design of timber structures for dissipative behavior a challenging process. Fundamental for the dissipation is the capacity design, which guarantees a reliable and accurate prediction of the nonlinear response of the dissipative elements. Therefore, an innovative support and anchoring system acting as a dissipative element was developed, aiming at improving the seismic performance of light-weight timber frame structures.

Currently, tension ties made of galvanized flat steel represent the common solution for anchoring timber frames to the foundation. These ties are designed for uplift forces only; they are connected to the timber studs by means of screws or nails and to the foundations by anchor bolts. In [1][2], tension ties were investigated and it was found that, although the tested specimens can resist adequate tensile forces and provide ductility redundancies, they are susceptible to sudden failures. The limited predictability of the governing failure mode of the tension ties was confirmed by own tests (Figure 8 b & c). Tlustochowicz [3], compared two different alternative techniques for the transmission of shear and tension forces: either through nailing plates welded to a foundation steel plate or via a HEB profile, which was connected to the studs of the timber wall by glued rods and anchored in the foundation. The specimens with glued rods failed suddenly in the glued interface [4]. The alternative with nailing plates is widely used in timber structures. Sufficient ductility can be provided by adequate number and strength of nails, but sudden block failure of the connectors must be prevented.

In light-weight timber frames, the key component governing the ductility (stiffness and strength as well) is the shear connection between sheathing and frame. The investigated anchoring detail was developed to enhance the ductility and dissipation capability of the structure by adding a dissipative mechanism at the base of the timber wall. The development of the novel anchorage targeted at eliminating the drawbacks of existing solutions, by providing a simple, reliable and replaceable element which, to some extent, also prevents damages to the main timber structure. To this end, the anchorage must provide a stable hysteretic performance in the plastic domain. Additionally, the opportunity to strengthen or substitute the anchorage after a significant seismic event resulting in plastic deformations limited to the anchor system was also taken into account. This concept allows for a rapid restoration of the occupancy of the building after an earthquake.

The novel anchorage examined within the current study is made of steel and is integrated into the corner studs of a timber wall, as shown in Figure 1. It consists of a steel rectangular hollow section (RHS) and a steel plate, welded to the upper flange of the hollow section. The steel plate is used for transferring vertical forces from the slitted timber stud by means of a double shear connection with steel dowels or bolts. The anchoring device is designed to transfer vertical loads (tension and compression) only. Shear forces are transferred by separate shear keys or angle profiles to the foundation.

The response of the structure to seismic loads can be "controlled" through definition of different performance levels for the proposed anchorages. Following the principles of the modern codes [6], three main performance levels are considered: damage limitation, significant damage and near collapse. By adjusting the elastic limit of the examined anchorage, the performance of the total building can be subdivided into: (i) damage limitation performance level: both superstructure and anchorage respond elastic, (ii) the significant damage performance level (or ultimate limit state): the anchoring device behaves nonlinear and dissipates energy, the timber superstructure remains undamaged or slightly damaged and (iii) near collapse level, damages (nonlinear response) are expected both at the anchorage and at the superstructure.

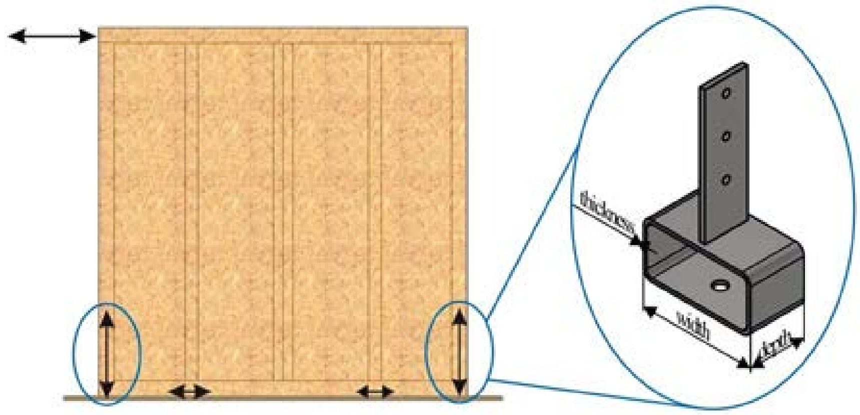


Figure 1: Innovative ductile and replaceable anchorage for timber shear walls

Crucial point of the developed concept is the capacity design of the detailing. The non–dissipative elements should be designed appropriately to remain elastic, permitting yielding and nonlinear response of the dissipative elements. Material hardening and overstrength of the dissipative elements must be considered. Concerning the tested subassemblies, fasteners, internal steel plate and timber studs were considered as non-dissipative elements. Regarding global systems, depending on the desired performance level, shear walls can also be considered as non-dissipative for specific scenarios.

The reliability of the novel ductile anchorage was investigated experimentally and validated numerically by the Institute of Steel Construction of RWTH University in Aachen, Germany. The investigation was part of a national funded research project (HOLZBEBEN), in which the seismic response of light-weight timber frames was holistically examined including tests on connections details, on wall elements and on a reference structure [7].

2 EXPERIMENTAL INVESTIGATIONS

Two series of experimental tests both monotonic and cyclic were carried out. The aim of the first series (S1) was the investigation of the principle of the novel anchorage and the validation of the numerical models, whereas the target of the following tests (S2) was to examine the performance of the anchorages as parts of a realistic building, including the interaction with the timber members. For comparison reasons also tests on commercial tension ties were executed. Table 1 provides an overview of the tested specimens and their characteristics.

Series	Specimen	Anchorage	Dimensions (b x t x h x d) in mm	Material	Connectors Ø in mm	Loading
S1	M01	novel anch.	200x100x100x6.3	S235	3 x threaded rods 12-8.8	monotonic tension
S1	M01*	novel anch.	200x100x100x6.3	S235	3 x threaded rods 12-8.8	monotonic compression
S1	Z01	novel anch.	200x100x100x6.3	S235	3 x threaded rods 12-8.8	cyclic (ECCS)
S1	Z02	novel anch.	200x100x100x6.3	S235	3 x threaded rods 12-8.8	cyclic (ECCS)
S1	Z03	novel anch.	200x100x100x6.3	S235	3 x threaded rods 12-8.8	cyclic (ECCS)
S2	A01	novel anch.	200x100x100x12.5	S355	14 x dowels 12-S355	monotonic tension
S2	A02	tension tie	HTT31 [9]	S350GD	49 screws 5.0x80 + 1 bolt 24-8.8	monotonic tension
S2	A03	tension tie	HTT31 [9]	S350GD	49 screws 5.0x80 + 1 bolt 24-8.8	monotonic tension

Table 1: Overview of experimental tests

2.1 Investigation of Series 1

2.1.1. Test set-up and instrumentation

The tests of this series were conducted on a ZwickRoell Z100 testing machine. The specimens were fixed through a bolt on a steel plate, which was rigidly clamped at the testing machine. The loads were transmitted from the machine jack through the timber element using a steel to timber connection.

Firstly, a monotonic tensile test (M01) was performed, in order to define the yield displacement, according to ECCS testing procedure [8]. The determination of the yield displacement is necessary to define the amplitudes for the following cyclic tests (Z01-Z03).

During the tests, the machine jack displacement and the applied load were recorded by the testing machine. In addition, the vertical displacement in the middle of the hollow cross-section (W1), the vertical displacement at the upper corner of the hollow profile (W2) and the relative displacement between the inner steel plate and the timber member (W3) were measured by displacement transducers. During the cyclic tests (Z01-Z03), the inclination of the right and left part of the hollow steel member flange were recorded by inclinometers (N1 and N2). The monotonic and the cyclic tests were conducted in displacement control with a constant speed rate of 0.1 mm/s. The instrumentation configuration is presented in Figure 2.

2.1.1. Results

Regarding the monotonic (M01 and M01*) tests, it was observed, that: (i) the novel anchorage presented significantly different stiffness in tension $K_t = 2.69\ kN/mm$ and in compression $K_c = 23.92\ kN/mm$, (ii) in tension the behavior was ductile and a remarkable amount of energy could be dissipated, (iii) no brittle failure occurred despite extreme tensile displacement, and (iv) the high compression stiffness is present, when the system anchorage-stud returns to the initial position and the compression load was transferred by contact between the timber stud and the flange of the RHS. The response of the anchorage is displayed in Figure 3, whereas the initial stiffness, the yield point (according to ECCS [8]), the dissipated energy and the damping ratio are depicted in Table 2.

Concerning the cyclic tests, it was observed, that the envelope curve is very close to the monotonic curve. Moreover, the anchorage was proved to be capable of dissipating energy and resisting a large number of loading cycles, without significant strength reduction (less than 7.5% as illustrated in Figure 4). In addition, the measurements of inclinometers confirmed, that the anchoring device responded in a symmetric way.

Figure 2: Series 1 test set-up

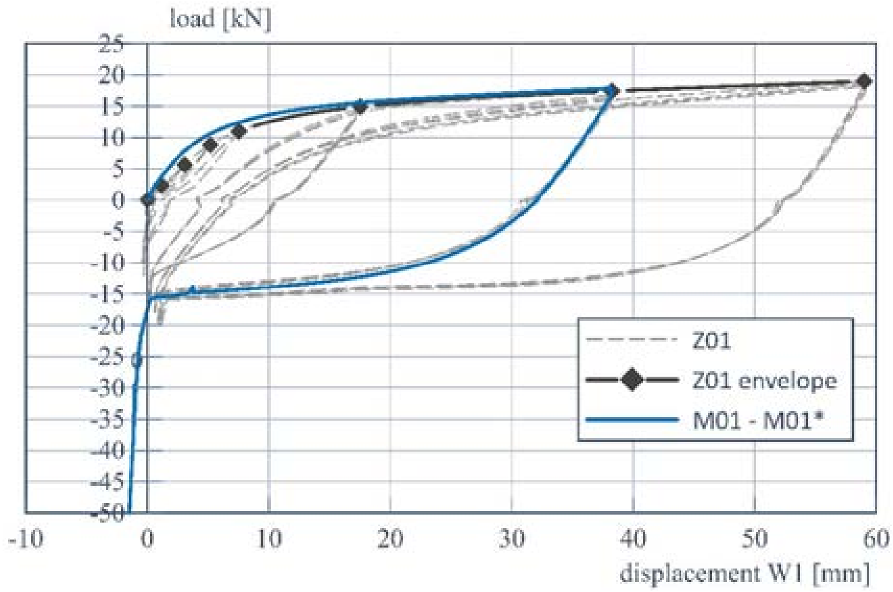


Figure 3: Load displacement curves M01 - M01* and Z01

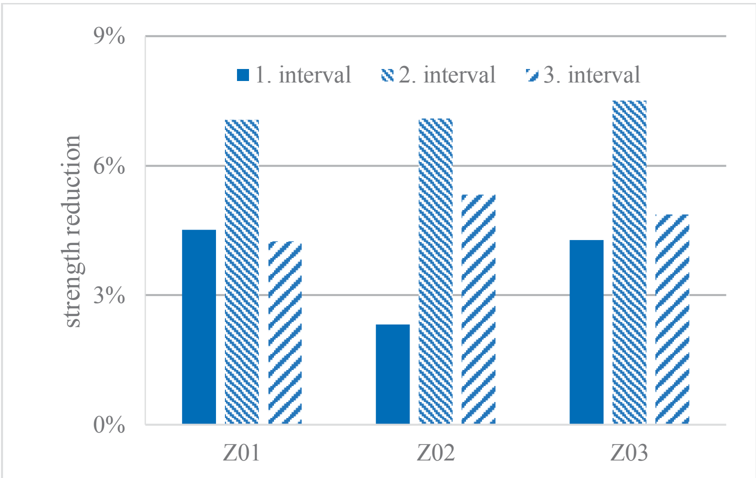


Figure 4: Strength reduction between 1. & 3. cycle of each interval

Figure 5: Deformed shapes of the specimen

Test results	Values
Initial stiffness in tension	2.69 kN/mm
Stiffness in compression	23.92 kN/mm
Yield force	12.30 kN
Yield displacement	4.48 mm
Dissipated energy	865 J
Equiv. damping ratio	7 %

Table 2: Series 1; results M01-M01*

2.2 Investigation of Series 2

2.2.1. Test set-up and instrumentation

After the applicability of the novel anchoring devices was experimentally proven in test-series S1, the anchorages were designed for the shear walls of a multi-storey timber building. In addition, commercial tension ties (HTT31, Simpson Strong Tie [9]), which could be used as a conventional solution for the same building, were selected as reference anchorages. Tensile tests were carried out on one specimen of the innovative anchorage and on two HTT31 specimens. The innovative anchorage was made of steel S355 and its thickness was *t=12.5 mm*. HTT31 anchoring elements are made of galvanized steel S350GD. According to [9], they can resist $R_{1,c} = 85.1$ kN characteristic tensile load. They are considered to be very ductile and capable of dissipating energy and are assumed to provide sufficient dissipation capability for ductility classes DCM or even DCH.

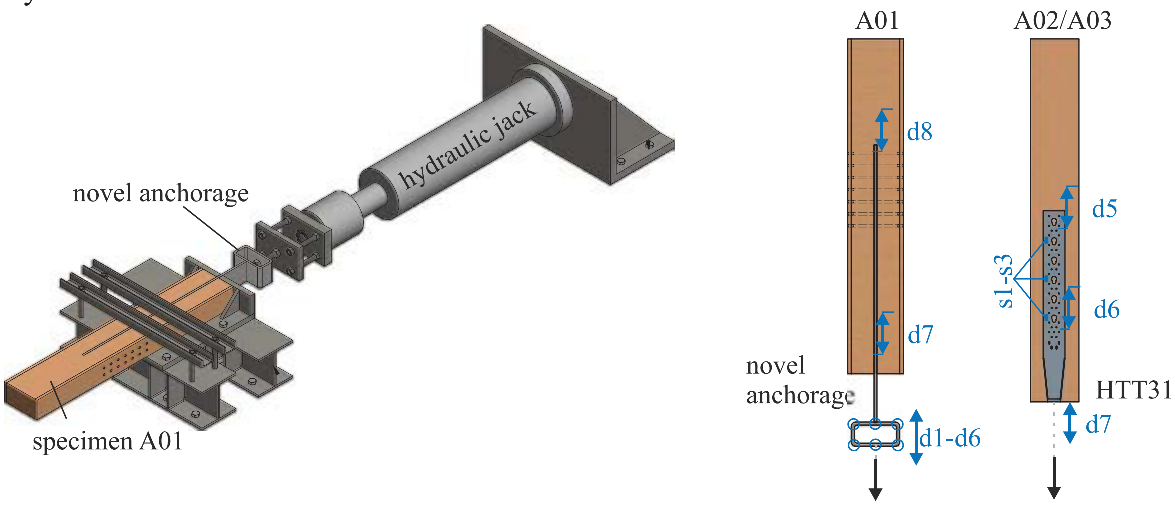


Figure 6: Series 2, test set-up and instrumentation configuration

For the test-series S2 a new test set-up was used, accounting for the increased dimensions and expected loads. The test configuration, the specimens and the instrumentation system are depicted in Figure 6. The instrumentation of the novel anchorage consisted of eight displacement transducers, two of them (d7 and d8) measuring the relative displacement between the internal steel plate and the timber stud. The other sensors (d1÷ d6) were measuring the response of the anchorage in terms of displacement at three points (left, right, middle) at the upper and at the lower flange of the hollow profile. The instrumentation of the tension ties included measurement of the relative displacement between steel plate and timber member by displacement transducers d5 and d6 and measuring of the displacement in the middle of the bottom angle part by the transducer d7. In addition, the strains at three characteristic positions along flat steel part of the ties were measured (s1÷ s3), in order to determine, the load distribution to the connectors. In all tests the hydraulic jack load and displacement were measured too.

2.2.2. Results

In Figure 7, the experimental results are presented. The novel device displays no failure until a load of *230 kN* (max. load capacity of the hydraulic jack) with a corresponding displacement of about *90 mm*. It is obvious, that such displacements are not considered as accepted under seismic loads. The maximum accepted displacement depends on the requirements of each building. Regarding the specimen, it is to underline, that after a displacement of about *40 mm*, membrane effects appear and are responsible for the change of inclination. Concerning the HTT31 tension ties, it is to be noted, that their stiffness, calculated according to [14], was close to the suggested values and their capacity about 27% higher than characteristic strength provided in the approval [9]. The results of the experiments are included in Table 3. In addition, it is worth mentioning, that the two HTT31 specimens presented different failure modes (Figure 8: b & c). The first one exposed net cross-section failure in tension, while the second one failed in the bottom part responsible for the connection to the foundation. Both failure mechanisms provided some plastic ductility followed by a sudden drop of resistance. Measurements by strain gauges showed a non-uniform force distribution along the HTT31 and the corresponding fasteners. The measured force decreases from *1 kN* (point s3) to *0.67 kN* (point s2) and 0.*25 kN* (point s3) (the values are normalized). Comparing the test configuration of the novel anchorage with the reference anchorage, the main conclusions are: (i) the novel anchorage is slightly stiffer and presents higher strength, (ii) the yield point is clearly identifiable and is about 50% higher (*96.01* kN to *62.65* kN), (iii) both anchors are very ductile in monotonic tension however, the novel anchorage has the potential to dissipate significantly more seismic energy by developing stable

Test results	Novel A01	HTT31 A03
Initial stiffness	29.63 kN/mm	27.39 kN/mm
Yield load	96.01 kN/mm	62.65 kN
Yield displacement	3.14 mm	2.42 mm
Max. load	>230 kN	109.17 kN
Max. displacement	>90 mm	19.62 mm
Ductility factor	-	8.1

Table 3: Series 2; results A01, A03

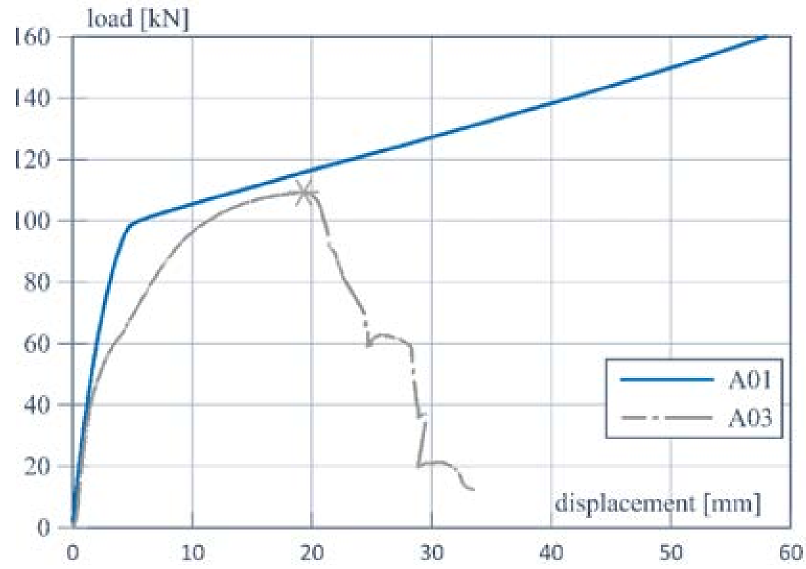


Figure 7: Test results A01 (novel anchorage) and A03 (HTT31)

hysteresis in tension and compression. In general, the examined novel anchorage shows an excellent performance, regarding the decisive properties for the seismic design – stiffness, strength and ductility - compared with a very effective reference tension tie (HTT31).

(a) (b) (c)

Figure 8: (a) Novel anchorage after loading with 230 kN and 94 mm deformation, (b) Failure mode of HTT31- (A2) specimen, (c) Failure mode of HTT31 - (A3) specimen

3 NUMERICAL AND ANALYTICAL INVESTIGATION

Numerical simulations were used to verify and to predict the behavior (load transfer, plastic hinges) of the examined specimens and to extend the study to other parameters (geometry, material properties) allowing for an adjustment of the anchorage to the design situation. The numerical investigations in this study were developed using the FEM software Abaqus Simulia 2018 [12].

3.1 FEM modeling procedure

The numerical models consisted of solid elements and included the rectangular hollow section (RHS), the steel plate, a foundation plate and the connectors, as depicted in Figure 9. The RHS, the steel plate and the foundation plate were modeled with a simplified isotropic bilinear material law for steel S235 and S355, according to the minimum requirements for steel as proposed by Eurocode 3[11]. The material law for the steel to timber connectors is taken from experimental results published in [10]. The timber element was modeled assuming elastic behavior, based on observations from the tests. Regarding the interactions and boundary conditions, the following options were applied: (i) “tie constraint” – for the welded connection between internal steel plate and hollow profile, (ii) “contact interaction” – for the interaction of different model components (bolts-timber, timber - RHS, RHS – foundation plate) in contact. The ‘normal hard contact’ used in the model, allows for separation between the parts and the ‘friction/penalty contact’ for considering friction effects. Sensitivity study for penalty values between 0.01 and 0.25 exposed that the penalty value does not influence the results. Regarding the boundary conditions, the foundation plate was fixed on both transverse edges, which approaches very close the real boundary conditions of the testing machine. The external loading was applied as imposed vertical displacement on the upper surface of the timber stud. For convergence reasons ‘Dynamic Explicit Analysis’, with a smooth step amplitude was selected, considering geometrical and material nonlinearities. The kinetic energy at all stages of the analyses (monotonic and cyclic) remained negligible. Regarding the discretization method, the mesh was adapted to the different components (*4 mm* for the timber stud and the hollow profile, *1.5 mm* for the connectors and *1 mm* at the areas of the RHS, where plastic strains were expected). The element type was linear hexahedral elements type C3D8R. The size of the mesh elements was adapted until the results were independent of the mesh.

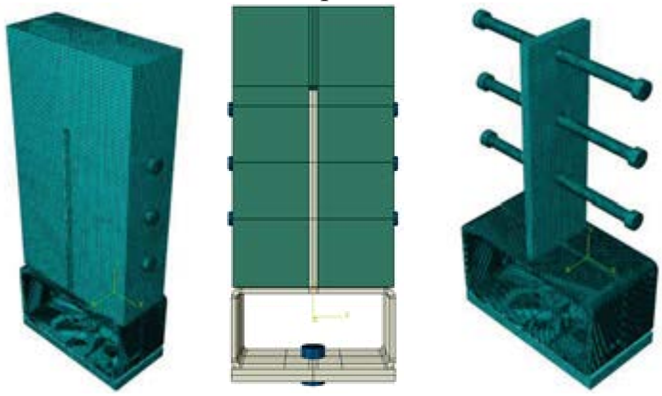


Figure 9: Assembled finite element model and discretization

3.2 Calibration of the model

The numerical model was developed with the characteristics described above and was calibrated with the monotonic and pseudo cyclic load – displacement curve from the tests A01 and M01 - M01*. In Figure 10, the results of the test and of the numerical simulation both in tension and compression are compared.

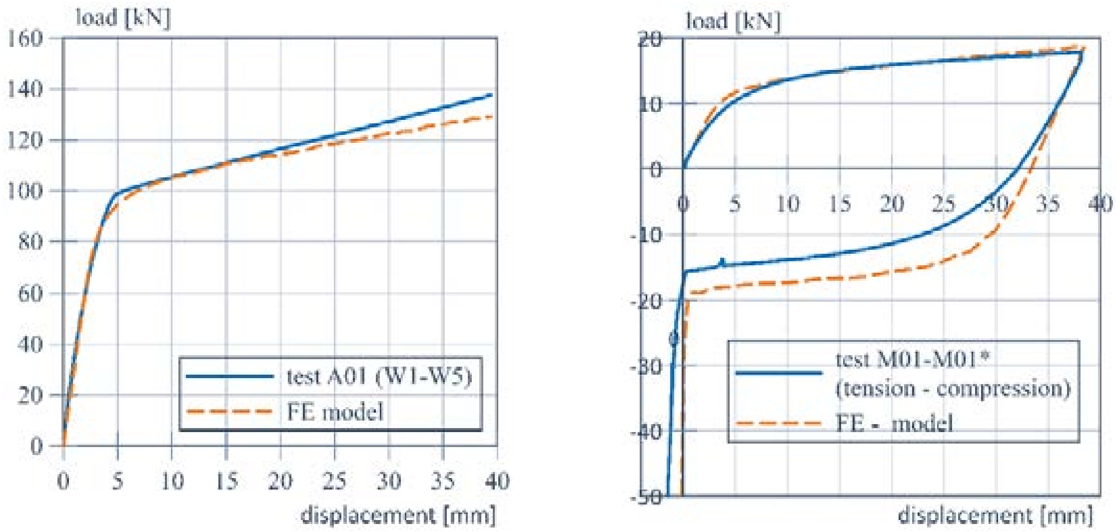


Figure 10: Comparison of response curves: test versus numerical reference model

Taking into consideration the simplified material law, the numerical and experimental results are in good agreement. The differences between the numerical and the experimental results are within an acceptable range for the intended application. In detail, the following deviations between A1 test and simulation are observed: (i) 1.0 % at yield force, (ii) 1.8 % at initial stiffness. Between M01-M01* test and FE model the deviations are: (i) 5.6 % at yield, (ii) 9.49 % at initial stiffness, (iii) 2.9 % at maximum load and (iii) 13.2 % at dissipated energy. The stress and stain distribution of the numerical model, that represented the specimen M01-M01* are displayed at the image series of Figure 11. The distribution of stresses and equivalent strain (PEEQ) illustrates the symmetric response of the anchoring and the sequential formation of plastic zones in the RHS, initially, near the foundation connector and at the sides of the vertical plate and afterwards at both ends of the webs.

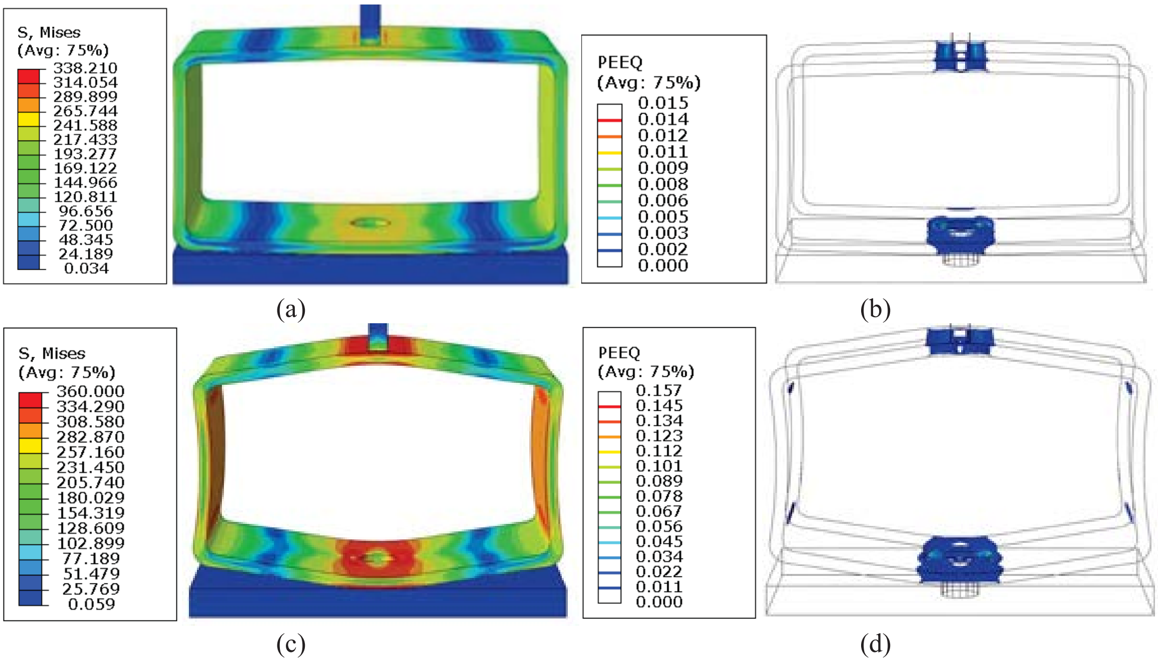


Figure 11: (a) stress distribution U2 = 5 mm, (b) plastic strains at U2 = 5 mm, (c) stress distribution U2 = 25 mm, (d) plastic strains at U2 = 25 mm

3.3 Parametric Study

After the calibration of the numerical models based on the experimental results, a parameter study was performed, in order to investigate the influence of the geometrical characteristics of the novel anchorage. The influence of wall thickness (*t*), width (*w*) and depth (*d*) of RHS was examined by varying separately these parameters in numerical models. As reference the anchorage with the properties as tested in first experimental series is used. The load-displacement curves in tension for various values of (*t*), (*w*) and (*d*) determined by FEM are presented in Figure 13. Furthermore, a simple analytical approach for the determination of the

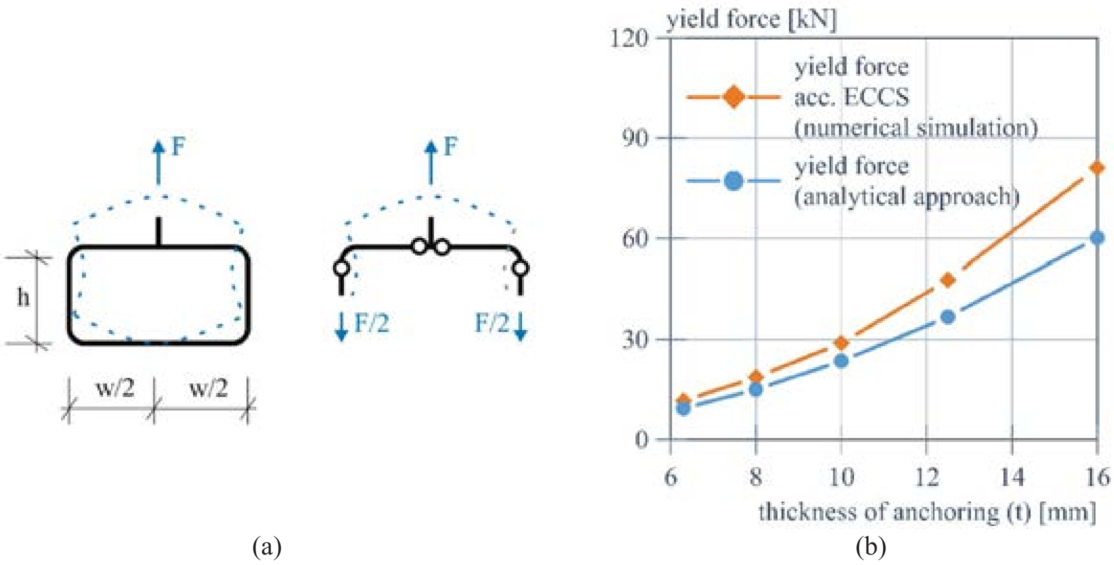


Figure 12: (a) Simplified model for analytical investigation, (b) Influence of thickness on yield force according to numerical and analytical approach

yield force was developed. For the analytical approach, the plastic zones are reduced to plastic hinges Figure 12a.The influence of the thickness on the yield force by the analytical and the numerical approach is illustrated in the Figure 12b. For the upper part of the plastic mechanism with four plastic hinges, the following equations apply:

$$M_{ex} = F_y \cdot \frac{w}{2} \tag{1}$$

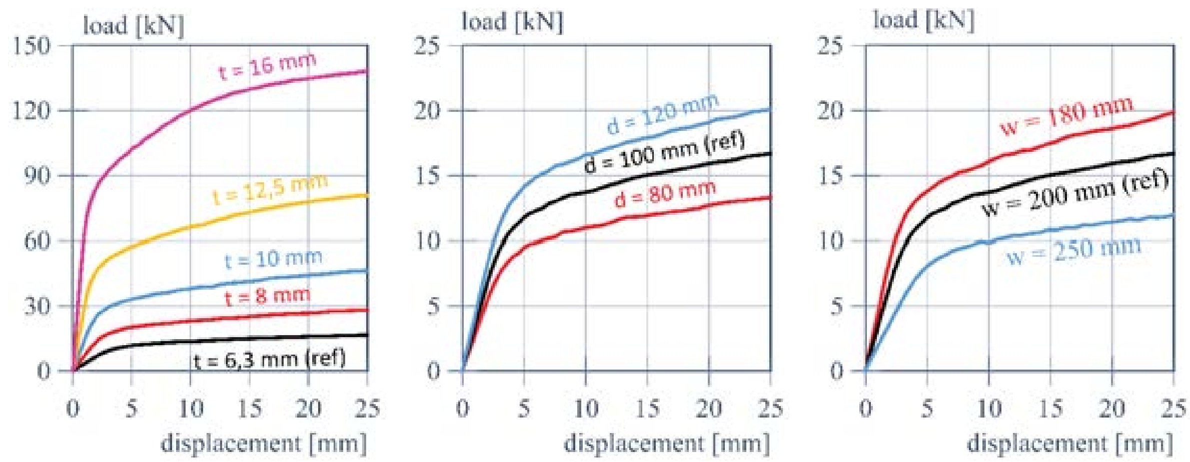
$$M_{ex} = 4 \cdot M_{pl} \tag{2}$$

$$M_{pl} = \left(d \cdot \frac{t}{2} \cdot f_y\right) \cdot \frac{t}{2} \tag{3}$$

Combining the equations (1÷3) allows the calculation of the yield force F_y as function of thickness t, depth d, width w and material (f_y):

$$F_y = \frac{2 \cdot f_y \cdot d \cdot t^2}{w} \tag{4}$$

Parametric studies have shown, that the thickness t has the strongest influence on the initial stiffness, the yield load and the dissipation capacity. The influence of the depth and width variation is less significant, but changes of these parameters allow for a fine-tuning of the anchors, according to the specific structure and loading situation. Finally, the dependency of the dissipated energy on thickness, depth and width of the RHS was examined by FE-analysis. To this end, each of these values was increased by 20% compared to the reference dimensions. The dissipated energy was determined for a loading cycle with a deformation amplitude of 30 mm in tension. It was found that 20% thickness increase (*t = 7.5mm*) leads to 53% more energy dissipation, 20% increase depth (*d = 120 mm***)** absorbs 19.8% more energy and a 20% wider RHS dissipated 29.3% less energy.



(a) (b) (c)

Figure 13: Load-displacement parametric curves of the anchorage (a) wall thickness varying between 6.3 mm to 16 mm, (b) depth varying between 80 mm to 120 mm and (c) width varying between 180 mm to 250 mm

3.4 Optimization for replaceability

One of major aims of the development of the novel anchorage was its replaceability after a significant earthquake. The substitution of dissipating structural parts is always a great challenge, as the existing structure should be temporarily supported and residual forces in the members or details are expected [15][16]. On the other hand, considering the concept of replaceability in the design and detailing of a structure inherently includes the option of recentering the main structure, under condition that the residual (plastic) deformations are limited to the specific dissipative objects. The introduced innovative anchoring solution can be also designed for disassembling and replacing, as long as the connection of the anchor to the timber studs remains elastic. Also, the main timber structure shall not suffer significant damages. Using the validated numerical models, different alternatives following this concept were investigated. The research focused on built-up members, in which one part only (the lower one in particular) is considered to be dissipative and needs to be replaced. The upper (non-dissipative) part must remain robust and undamaged. After investigating several alternatives, the configuration illustrated in Figure 14a, was selected as the most efficient one. It enables the transfer of gravity forces by contact and simultaneously the lower part (dissipative part) is activated only when uplift forces are present. The investigated hybrid built-up member consists of an outer profile with thickness *t =16 mm* and width *w = 200 mm*. Regarding the internal profile, the dissipative flange was selected as *t = 6.3 mm*; the webs are *t = 16 mm* thick as well. The total height of the anchorage remains 100 mm, the steel material is S235 for both the lower and the upper part. The connecting bolts are M16-10.9.

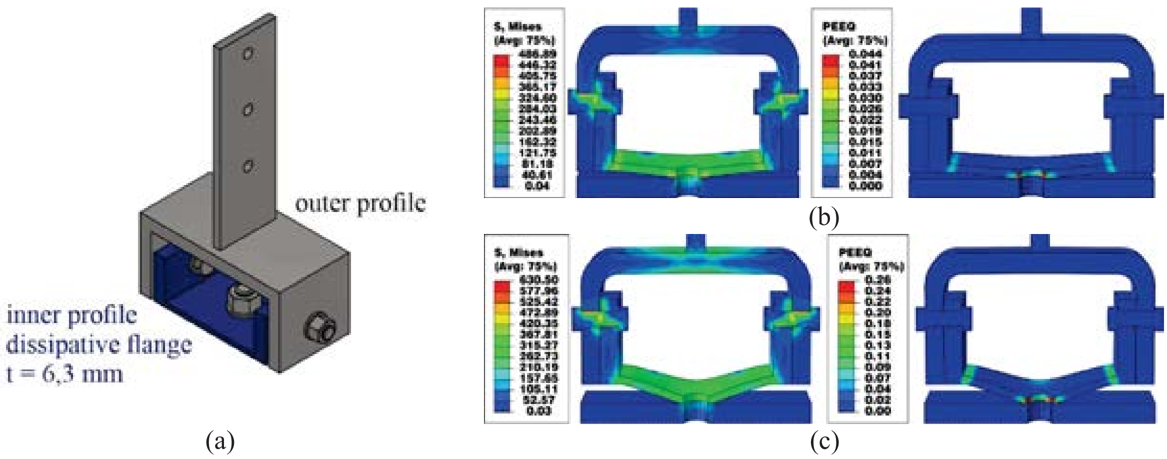


Figure 14: (a) Built up optimized anchoring device, (b) Stress and plastic strain distribution, U2 = 5 mm, (c) Stress and plastic strain distribution, U2 = 15 mm

The results of the numerical simulation, plotted in Figure 15, are very encouraging, as the optimized configuration presents a similar initial stiffness to the RHS anchoring configuration with *t = 10 mm* thickness ($K_{t=10} = 14.57\ kN/mm$) and a performance (yield force and plastic behaviour), which corresponds to the novel RHS anchorage with a thickness between *t = 8 mm* and *t = 10 mm*. The plastic hinges form in three regions (around the foundation bolt and at both ends of the dissipative flange), as illustrated in Figure 14 (b & c: plastic strains). The performance limit was estimated to be at an uplift deformation of about 26 mm, where the concentration of strains is significantly beyond those in the reference configuration (RHS). Nevertheless, the ratio between ultimate displacement ($d_u = 26\ mm$) and yield displacement ($d_y = 1.4\ mm$) combined with the stable cyclic response without pitching effects (Figure 15b) indicate that the optimized configuration is also capable of dissipating considerable amount of seismic energy. In order to keep the lower part replaceable, the webs of the lower profile must remain elastic, to enable unscrewing the connecting bolts.

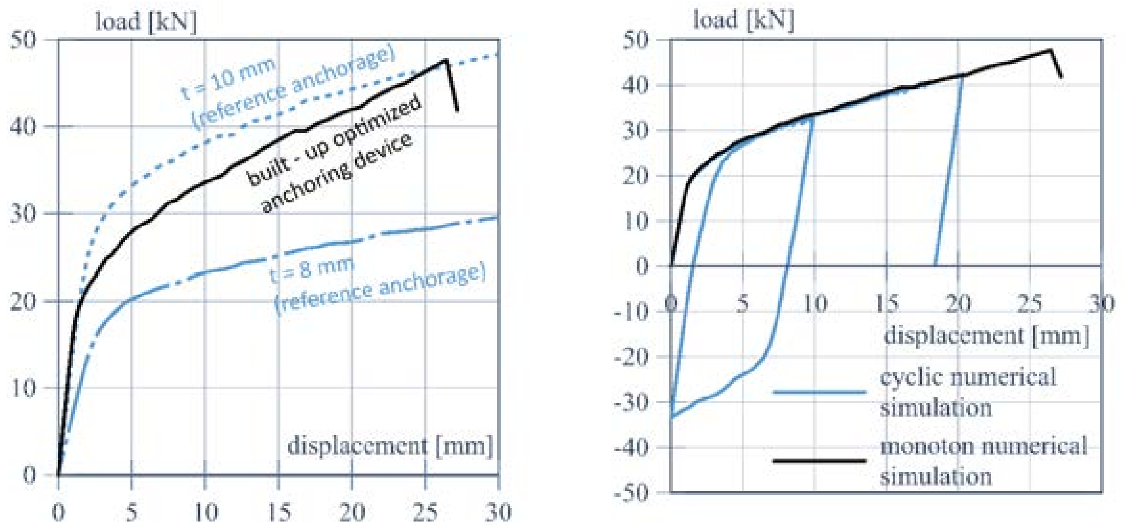


Figure 15: Load – displacement curves of the optimized anchoring configuration

4 APPLICATION

For the application of the novel anchors in timber buildings, the stiffness and strength characteristics and relations of the timber superstructure and the anchors need to be considered. The anchors have different stiffness properties in pure compression (e.g., gravity loads govern the force) and if uplift forces (overturning moment governs) are present. In pure compression the elastic stiffness of the anchors is very high, thus having limited influence on the stiffness of the building. The plastic deformations and cyclic energy dissipation are activated as soon as uplift forces appear. In order to avoid damages to the timber building, it needs to be designed using the lateral seismic actions corresponding to the yield force of the anchors. This shall comprise material overstrength and hardening effects of the dissipative part. Also, the transfer of the base shear force to the foundation and of the shear force between the storeys must follow this principle. If designing for deformations on top of the building, the whole horizontal displacement shall be determined considering two components:

(i) the displacement Δ_1, which is the sum of the horizontal head displacement of the wall on each floor due to shear loads and depends on the applied shear force on each floor and on the stiffness of the wall and

(ii) the displacement Δ_2, which is the result of the rocking effect due to usage of the novel ductile anchorage and depends on the presence of uplift forces and the corresponding stiffness of the anchoring

In order to achieve an optimum seismic performance in combination with limited damage to the building, the following aspects shall be considered:

- anchorage should be equipped with sufficient strength to remain elastic under moderate earthquakes (local damage limitation);
- light timber shear walls should be designed appropriately according to capacity design as non–dissipative elements for the design earthquake, in order to concentrate the damages in the ductile anchoring device;
- the inherently available ductility of the light timber shear walls should be activated only for a near collapse seismic scenario. In such case, lateral deformations may become critical (inter-storey drift, theory 2[nd] order effects). This can be controlled either by selecting stiffer profiles or by using additional tension ties, which get activated after a certain, pre-defined uplifting displacement.

As horizontal displacements and drifts are decisive for the design of the ductile anchorage, a pushover analysis is recommended for the evaluation of the different performance levels. Finally, the novel anchorage is designed to transfer vertical loads only (uplifting and compression). In order to prevent unintended transfer of base shear forces, slotted holes for the innovative anchorage should be used. Additional shear keys preferably at the middle of the wall, are suggested.

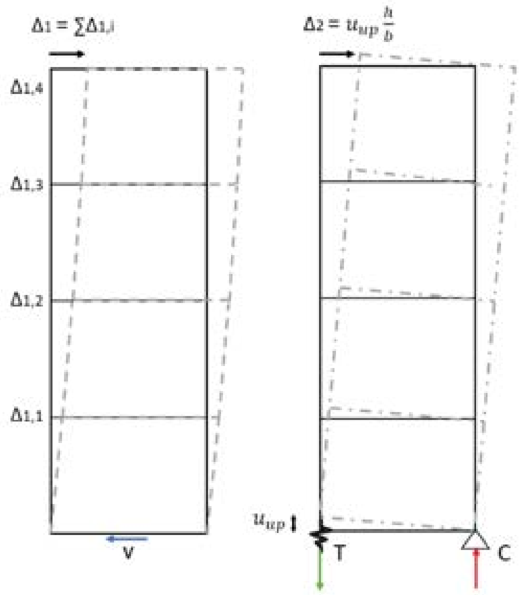


Figure 16: Shear displacements (left), displacement due to rocking effects at the base (right)

5 CONCLUSIONS

The current paper described the results of experimental and numerical investigations of an innovative, ductile and replaceable anchorage for timber shear walls. Aim of the developed device is the protection of the timber superstructure. One additional advantage of the investigated system is its replaceability. The monotonic experimental tests have shown that the anchoring device behaves in a very ductile manner, when uplift forces are present; in pure compression the loads are transferred by contact and the anchor behaves elastically with a high stiffness. Buckling effects under compressive loads should also be taken into consideration. Cyclic tests have proved, that the investigated anchoring can withstand many loading cycles even of high amplitude, without any significant reduction of its strength. Simultaneously a reasonable amount of seismic energy can be absorbed. After validating the numerical models based on experimental results, a parametric study was conducted and proved that the governing parameter for the behavior of the introduced anchorage is the thickness of the profile. Thicker profiles have an increased stiffness, yield force and dissipation capacity. Finally, the examined anchorage was optimized aiming at its replaceability. The most effective solution among several investigated cases was a built-up member, in which only the lower part is dedicated to dissipate energy and must be replaced after a significant earthquake. The achievement of the intended plastic mechanism requires the application of capacity design rules, in order to prevent plastic deformations or damages in the connections to the timber studs and to the parts of the anchorage, which are supposed to remain in the structure.

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