16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago de Chile, January 9th to 13th 2017

Paper N° 3379

Registration Code: S-J1460625821

JOINT PROPERTIES AND EARTHQUAKE BEHAVIOUR OF BUILDINGS MADE FROM DOWEL-LAMINATED TIMBER

C. Sandhaas[(1)], P. Schädle[(2)]

[(1)] Researcher, Karlsruhe Institute of Technology, Timber Structures and Building Construction, sandhaas@kit.edu
[(2)] Structural Engineer, Ingenieurgruppe Bauen Karlsruhe, patrick.schaedle@ingenieurgruppe-bauen.de

Abstract

Dowel-laminated timber (DLT) elements consist of lamellae arranged side-by-side that are connected with beech dowels. Due to the glue-free DLT element layup, joints and shear walls potentially suffer from considerable reduction of stiffness and load carrying capacity as fasteners inserted perpendicular to the element plane may be placed in gaps between the single lamellae. Tests on joints showed that, depending on the fastener diameter, the remaining load carrying capacity of joints in DLT in comparison to joints in solid wood may be only 25%. Monotonic and quasi-static reversed cyclic tests on DLT shear walls demonstrated that the DLT construction typology has stiffness, load carrying capacities and energy dissipation properties similar to traditional timber frame constructions. Via lumped-mass models of a typical residential building whose hysteretic behaviour has been assigned to nonlinear hysteretic springs, a preliminary action reduction factor (ARF) has been derived. The springs have been calibrated using the test results of shear walls. The modelling outcomes have been compared to other timber construction typologies. Preliminary ARFs for DLT buildings resulted to 3 and are similar to those of the other typologies.

Keywords: Dowel-laminated timber, joints, shear walls, earthquake behaviour, nonlinear dynamic modelling

1 Introduction

In the last decennia, a large variety of timber building systems has been developed in Europe that complements traditional timber frame buildings. Dowel-laminated timber (DLT) belongs to these novel timber building systems. DLT elements consist of lamellae arranged side-by-side that are connected with beech dowels and are used as floor and wall elements in buildings. Considerable research has been carried out concerning DLT plates loaded out-of-plane, both timber-only [1] and timber-concrete [2] applications. No research could be identified related to joint and shear wall behaviour. Joints, e. g. joints fastening wall elements to the floor, and shear walls providing lateral stability are however essential for DLT buildings. More specifically, shear wall behaviour must be fully understood when buildings need to be assessed with regards to their earthquake resistance. Due to the glue-free layup of the DLT elements, gaps between the single lamellae are present (Fig. 1 on the left). Fasteners inserted perpendicular to the element plane could penetrate these gaps, and the resulting joint stiffness and load carrying capacity is expected to be significantly reduced which may subsequently also affect shear wall performance. As a result, a testing series using typical joint setups for DLT building typologies has been carried out together with reference series using solid timber in order to investigate the influence of the gaps on joint behaviour and thus also on shear wall behaviour. Furthermore, lateral stiffness, strength and energy dissipation capacity of DLT shear walls are difficult to assess as the lateral stiffness and capacity of DLT elements themselves are unknown. Current design handbooks do not contain information on joints or shear walls in DLT. In Eurocode 5, DLT is only considered when used as floor elements where an increase in strength of the single lamellae due to load distribution effects can be taken into account (system strength factor k_{sys}, Eurocode 5, section 6.6) [3]. According to Winter et al. [4], DLT walls currently require additional sheeting and can be designed similar to timber frame wall diaphragms (method A of Eurocode 5, section 9.2.4.2). This assumption could be confirmed with preliminary tests [5]. However, the stiffness and load carrying capacity of the entire DLT shear wall is unknown. Therefore, the influence of different joint layout on shear wall stiffness and load carrying capacity was examined by carrying out monotonic tests on shear wall specimens. Furthermore, as DLT buildings are also built in earthquake regions, quasi-static reversed cyclic tests have been carried out to assess energy dissipation capacity.

With the evaluated experimental results, important knowledge gaps concerning joint and shear wall properties can be filled and DLT buildings loaded statically can be designed by using e. g. reduction factors for load carrying capacities of joints. Dynamically loaded buildings however require more information. In current force-based seismic design methods, action reduction factors are "…used for design purposes to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures." (Eurocode 8 section 1.5.2, [6]). To derive such action reduction factors, costly shaking table tests or numerical simulations must be carried out with which a complete building can be subjected to real-time earthquake loading. In this article, the modelling approach proposed by Ceccotti and Sandhaas [7] has been used to evaluate action reduction factors for DLT buildings. In order to classify the DLT timber building system for which no seismic parameters have been known up to now, similar simulations have been executed considering other timber building systems. Three other building systems have been chosen, glued cross-laminated timber (CLT), prefabricated timber wall elements (PFTE) and, in order to compare the newer systems with a well-established system, timber frame systems were considered as well. The results of the cyclic shear wall tests have been used to calibrate nonlinear springs that represent the hysteretic shear wall behaviour. Subsequently, the derived preliminary action reduction factors for all investigated building systems are compared and critically discussed.

2 Investigated novel timber building systems

2.1 Dowel-laminated timber

The general layup of DLT elements studied here is shown in Fig. 1 on the left. The 20 mm beech dowels with a spacing of 300 mm are connecting the single lamellae of spruce (*Picea abies*). The lamellae themselves have a fixed cross-section of 24 x 100 mm^2 thus resulting in a gap perpendicular to the element plane every 24 mm. These standard DLT elements with a width of 625 mm can be assembled to typical shear walls as shown in Fig. 1 on the right. The single DLT elements are not fastened to each other. In building practise, DLT wall elements are pre-assembled in the factory including top and bottom plates for their out-of-plane stability during transportation on site. Usually, sheeting is applied as well which is fastened along the perimeter of every sheet using staples. On site, the wall element is fastened to the floor plate using standard angle brackets and, if necessary, hold-downs whose fasteners are inserted perpendicular to the element plane. The sheeting fulfils several functions. It supports the role of the top and bottom plate in terms of out-of-plane stability and it is necessary for a satisfactory building physics performance as the DLT elements have inherent gaps and thus cannot provide air tightness. Furthermore, sheeting is mechanically necessary as only with sheeting, a diaphragm action of the walls can be guaranteed [5].

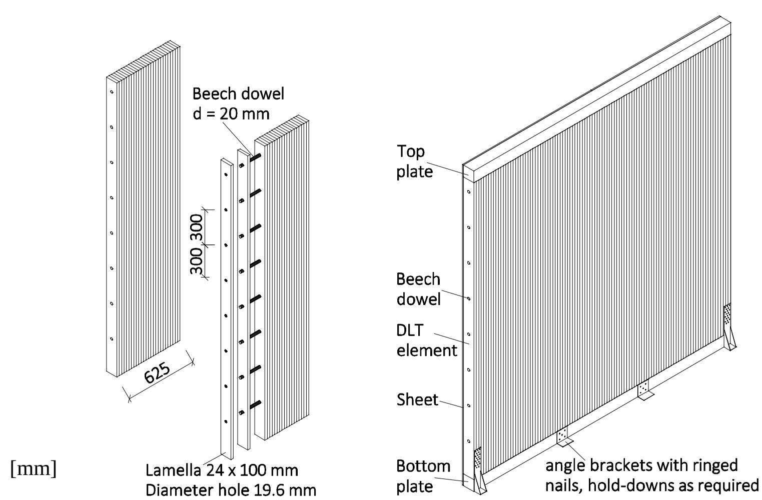

Fig. 1 – Left: DLT element. Right: DLT shear wall.

2.2 Glued cross-laminated timber

Glued cross-laminated timber (CLT) is, apart from timber framing, the product for timber construction typology that has been investigated most extensively in the last years (e. g. [8-11]). Thanks to its cross-wise fibre orientation in the single layers, CLT allows for versatile, two-dimensional, structural elements useable as wall or floor elements. CLT panels are rigid, dimensionally stable and allow for multi-storey buildings suited also to earthquake regions. Recent research carried out worldwide could confirm this [8-11]. The glued CLT panels used in this research consisted of panels with interspaces as shown in Fig. 2. Also non-glued CLT exists where wooden dowels or screws instead of glue are used to connect the single lamellae [12]. The production and erection of buildings is similar to those of DLT buildings. Wall elements are prefabricated in the factory, often already including thermal insulation and windows, transported on site and mounted. Differently to DLT, CLT shear walls do not need sheeting as the CLT elements themselves are rigid enough to provide lateral stability. Joinery used to connect the walls to the floors and the single shear walls include self-drilling screws, steel brackets with ringed nails and notched joints with LVL inlays [13]. Generally, joints in CLT are much weaker than CLT elements themselves. Innovative contact joints are developed in ongoing research projects that potentially overcome these issues without loosing energy dissipation capacity [14].

Fig. 2 – Glued CLT element with interspaces.

2.3 Prefabricated timber wall elements

The main feature of prefabricated timber wall elements (PFTE) is prefabricating wooden “brick” elements in small units primarily using sawmill residues. These elements represent a simple, sustainable construction system which is easy to handle on the building site (www.hib-system.be). The “wooden brick” in its basic dimension of 1,0 m x 0,5 m x 0,16 m (length x height x thickness) consists of four solid wood columns spaced at 250 mm and particle board sheeting, Fig. 3. The single elements are stuck together by overlapping/ shortened columns with dovetail geometry at the top/ bottom of the element. On both sides, a second layer is fixed to an inner sheeting layer. The second (outer) layer is fixed with a horizontal and vertical offset of 30 mm. When setting up the wall, the offset of the outer layers of lower and upper elements slide into the next one, so that the outer layer overlaps from one element to another. After finishing erection, the overlapping parts of the sheeting are connected by staples to create continuous shear walls. When erecting a wall with PFTE, first a bottom plate is fixed to the foundation. The next layers are simply laid by stacking the wooden “bricks”. When the planned wall height is reached, a continuous vertical stud is inserted from the top at least every 3 m of wall length. The vertical studs transfer the in-plane uplift forces to the foundation and they provide bending stiffness for loads perpendicular to the wall plane, e. g. wind loads. At the top of the wall, the top plate is added and the vertical studs as well as the top plate are connected to the elements via self-tapping screws. Further information is given in literature [15].

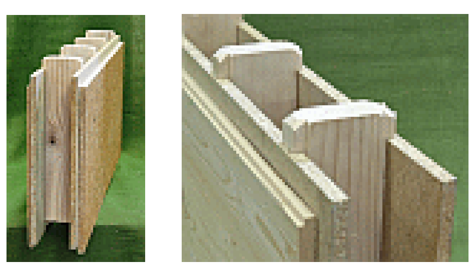

Fig. 3 – Glued PFTE. Left: Basic “brick”element. Right: Detail.

3 Properties of joints and shear walls using DLT elements

3.1 Monotonic tests on joints

An experimental programme covering different typical DLT joint configurations has been carried out to assess the influence of gaps between lamellae. As can be seen in the sketch of a typical shear wall example, Fig. 1 on the right, fasteners connecting angle brackets, hold-downs and oriented strand boards (OSB) sheeting to the shear wall are inserted perpendicular to the element plane and may thus be inserted in the gaps potentially leading to a lower stiffness and strength. Fig. 4 on the left shows the test setup with a symmetrical specimen where four joints with five fasteners each were tested. The used materials were DLT elements with a width of about b = 310 mm (about 13 lamellae), a width of about b = 160 mm (about 7 lamellae), 0.75 mm steel plates with 31 x 80 smooth nails, 1.0 mm steel plates with 40 x 50 ringed nails, 12 mm thick OSB/4 with 1.53 x 55 staples and 25 x 55 coil nails. The DLT elements were produced with lamellae of a mean moisture content of u = 12.8% and a mean density of ρ_u = 486 kg/m^3. The end distance of the beech dowel differed as shown in Fig. 4 on the left. The five fasteners of a joint were inserted in the gaps and centrally in the lamellae. For each tested configuration, the DLT elements were replaced with solid wood (*Picea abies*, u = 13.0%, ρ_u = 483 kg/m^3) to establish the influence of the built-up of DLT elements. Four transducers were used that measure the relative slip between wood and OSB or steel plate. Due to the symmetrical setup, one (conservative) value per test for the load carrying capacity F_{max} and four values per test for the slip modulus K_{ser} were derived. The used test protocol corresponded to EN 26891 [16]. More information on the experimental programme and results are given in [5]. The test results in terms of load carrying capacity are shown in Fig. 4 on the right. The mean value for the load carrying capacity of a joint with five 4 mm ringed nails is 15.8 kN if solid wood is used and only 4.3 kN if DLT is used and the nails are inserted in the gaps. Even if the ringed nails are all inserted centrally in the lamellae, the mean load carrying capacity is with 9.0 kN nearly half the value for solid wood. This can be explained with the thickness of the lamellae of only 24 mm leading to premature splitting already when inserting the nails. Also the position of the beech dowel influences the load carrying capacity. If the dowel is placed "underneath" the joint with an end distance of 30 mm, the mean value is with 7.4 kN a bit higher than the 4.3 kN with the dowel "above" the joint. Similar trends can be observed for the slip modulus K_{ser}. This trend of decreasing stiffness values and load carrying capacities is less significant the thinner the fasteners are. Staples, for instance, show much smaller load drops due to the thin shanks that do not lead to premature splitting and due to the fact, that not both shanks are inserted in the gaps, but only one. Whereas a ductile failure mode with embedment and one yield moment could be observed for all joints in solid wood, especially the fasteners with larger diameters in joints with DLT simply rotated in the gaps resp. the lamellae not showing significant plastic deformation.

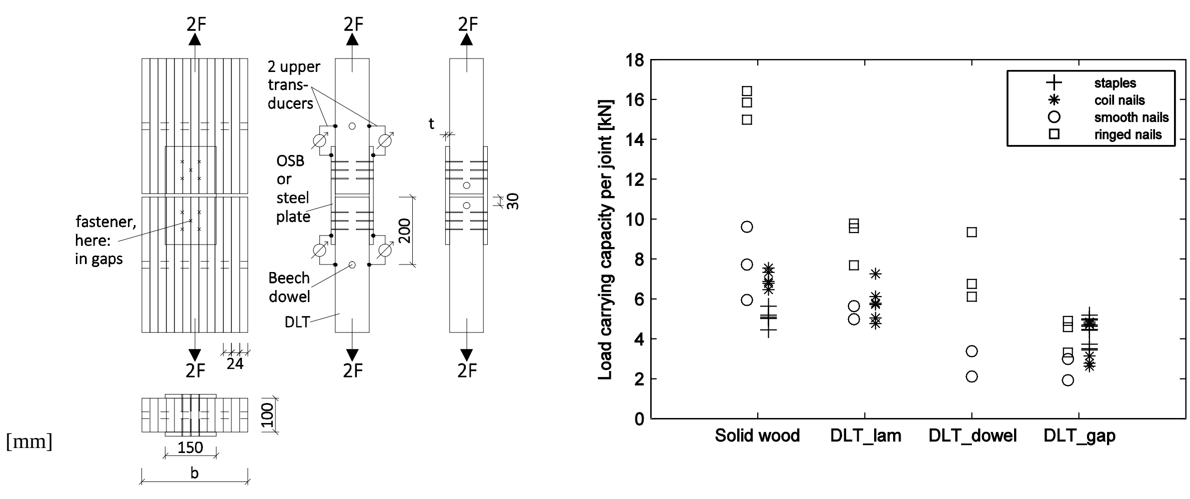

Fig. 4 – Left: Setup for joint tests. Here: fasteners inserted in gaps between lamellae. Right: Test results for joints with five fasteners. DLT_lam = fasteners in lamellae and dowel distance = 200 mm, DLT_dowel = fasteners in gaps and dowel distance = 30 mm, DLT_gap = fasteners in gaps and dowel distance = 200 mm.

3.2 Monotonic and cyclic tests on shear walls

A typical layout of shear walls in DLT buildings is shown in Fig. 1 on the right. The shear walls consist of several DLT elements with top and bottom plate and additional sheeting. As already discussed, the influence of different sheeting material (OSB and gypsum plasterboard (GPB)) and arrangement has been assessed in a preliminary testing series [5]. These tests showed clearly that DLT shear wall elements need additional sheeting in order to be considered as lateral load carrying elements. In this contribution, the influence of different joint solutions to fasten prefabricated DLT shear wall elements to the flooring is discussed. Three different fastening solutions, series G to K, have been selected which are shown in Fig. 6. In series G to K the sheeting consisted of one-sided OSB panels, the layout is given in Fig. 5 on the left. The fastening detail of series G consisted of a bottom plate that was mounted on-site. The wall element with longer OSB panels on both sides was inserted and fastened with staples and, in the case of series K, with additional hold-downs. In series J instead, the bottom plate is part of the prefabricated wall. On site, the wall element is fastened with steel brackets and hold-downs which are both fastened with ringed nails. Besides assessing the lateral stiffness and capacity of DLT walls with different fastening solutions via monotonic shear wall tests according to EN 26891 [16], also quasi-static cyclic tests according to EN 12512 [17] have been carried out. The cyclic tests served to evaluate the energy dissipation capacities of DLT shear wall systems and as input for a numerical model with which important data related to seismic design, i. e. sustainable peak ground accelerations and action reduction factors, have been derived. The arrangement of the measuring equipment is shown in Fig. 5 on the right. All displacement data used here is the horizontal displacement at wall top. Table 1 summarises all monotonic and Table 2 all cyclic tests carried out. More detailed information, e. g. densities and moisture content, quantity of fasteners and load-slip curves, is given in literature [5].

Results of monotonic tests

The results of the monotonic tests are given in Table 1. The results for the different fastening solutions, series G to K, do not show large differences among each other except for the deformations U_{Fmax}. In series K, the influence of the additional hold-down can be seen in the higher value for the ultimate load F_{max} in comparison to series G without hold-down. However, the values for f_{5mm} and stiffness K_{ser} do not differ considerably as the hold-down starts contributing only at larger deformations. Generally, the test results are on the safe side as the lower horizontal movement of the entire wall element has not been blocked. Therefore, the influence of perpendicular walls blocking horizontal movement has not been taken into account. All wall elements showed ductile failure with significant inclination of the elements and hence significant rotation of the sheeting. Subsequently, the staples fastening the sheeting to the DLT elements developed plastic hinges. The executed shear wall tests for DLT walls did not show significantly better results than shear wall tests on timber frame structures (same test setup and same dimensions) taken from Blaß and Schädle [15] which are also given in Table 1. It must be noted though that the horizontal movement of the tested timber frame walls had been blocked leading to lower ultimate deformation and probably also to higher stiffness.

Results of cyclic tests

Table 2 gives the results for the cyclic tests on DLT wall elements with three different fastening solutions and results for timber frame walls of the same dimensions taken again from Blaß and Schädle [15]. Equivalent viscous damping υ per half cycle has been calculated according to EN 12512 [17]. Both υ and strength degradation ΔF have been evaluated for the cycles at F_{max} (F_{max} corresponds thus to the maximum force of the first of three cycles). Evaluated values the equivalent viscous damping of the first cycle of DLT wall elements are similar to those of timber frame wall elements and lie between 13% and 16%. Also the reduction of equivalent viscous damping between first and third cycle is similar for both wall systems. The strength degradation between first and third cycle is with 20% to 30% largest for series J and, in terms of seismic design, even too large as Eurocode 8 sets a limit of 20% for strength degradation [6]. Series G, where no hold-down was used, performed unexpectedly well. In comparison to the monotonic test results, Table 1, the ultimate load carrying capacity F_{max} is higher and more similar to F_{max} of series K and J, which may be due to natural scatter of results (only 1 wall tested). The importance of hold-downs in DLT structures may thus be overrated if they are applied on the wall sides. However, only few tests were carried out, therefore no conclusive statements are possible.

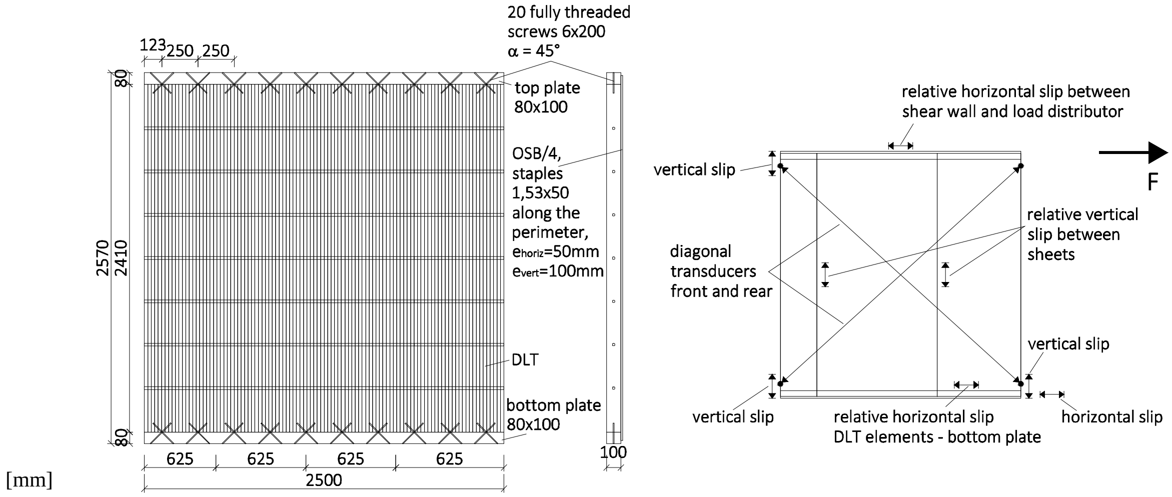

Fig. 5 – Left: Standard shear wall. Right: Arrangement of measuring equipment.

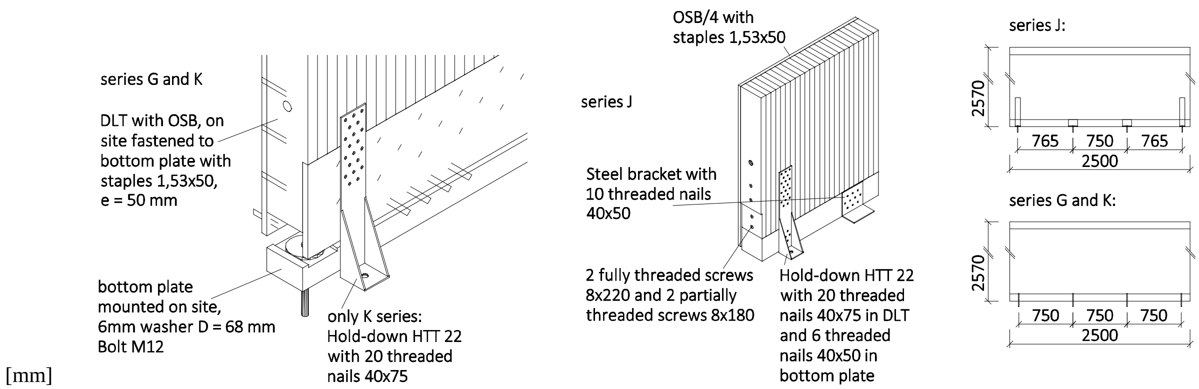

Fig. 6 – Joint details series G, K and J.

Table 1 – Monotonic shear wall tests. F_{max} = ultimate load carrying capacity. U_{Fmax} = horizontal displacement at F_{max}. f_{5mm} = lateral load carrying capacity at 5 mm horizontal displacement at top of wall per meter wall. K_{ser} = lateral stiffness. Designation: "0" = no vertical load, "10" = 10 kN/m vertical load.

Series	n	F_{max} [kN]	U_{Fmax} [mm]	f_{5mm} [kN/m]	K_{ser} [kN/mm]
G – bottom plate mounted on site					
G_10	1	35	61	4	1.4
K – same as G, but with hold-downs					
K_10	1	49	89	4	1.6
J – steel brackets and hold-downs					
J_0	1	50	101	4	1.2
J_10	1	50	146	4	1.4
Timber frame wall 2.55 m x 2.5 m with one-sided OSB/3 fastened with staples 1.53 x 64, e = 50 mm*					
PO_10	1	50	51	5	1.9

* taken from Blaß and Schädle [15]. Lower horizontal movement has been blocked.

Table 2 – Cyclic shear wall tests. F_{max} = ultimate load carrying capacity. U_{Fmax} = horizontal displacement at F_{max}. Equivalent viscous damping over the first half cycle, $\upsilon_{Ed,1}$, and over the third half cycle, $\upsilon_{Ed,3}$. ΔF_3 = strength degradation between first and third cycle.All calculated for the cycles at F_{max}.
Designation: "10" = 10 kN/m vertical load, "20" = 20 kN/m vertical load.

Series	n	F_{max} [kN]	U_{Fmax} [mm]	$\upsilon_{Ed,1}$ [%]	$\upsilon_{Ed,3}$ [%]	ΔF_3 [%]
G – bottom plate mounted on site						
G_cyc_10	1	46	59	16.0	10.3	9.2
G_cyc_20	1	48	59	16.2	11.5	8.2
K – same as G, but with hold-downs						
K_cyc_10	1	56	60	16.4	12.6	13.5
J – steel brackets and hold-downs						
J_cyc_10	1	50	89	13.6	11.2	20.8
J_cyc_20	1	50	88	16.6	15.6	30.2
Timber frame wall 2.55 m x 2.5 m with one-sided OSB/3 fastened with staples 1.53 x 64, e = 50 mm*						
ZYK_10	1	51	60	13.0	9.0	17.1

* taken from Blaß and Schädle [15]. Lower horizontal movement has been blocked.

4 Nonlinear dynamic modelling

4.1 Methodology

Testing on joints and substructures such as shear walls is not sufficient to exhaustively assess earthquake performance of buildings. By means of cyclic tests on shear walls, relevant information about envelope curves, energy dissipation capacities, stiffness and strength degradation and failure modes can be gained. However, information concerning sustainable peak ground acceleration (PGA) values or action reduction factors, called behaviour factor q in Eurocode 8, cannot be gained with these tests. As an alternative to costly shaking table testing, numerical models can be used to derive action reduction factors for buildings as other, existing methods are not able to provide reliable values. Eurocode 8 [6] for instance allows to choose the behaviour factor according to ductility class M if the static ductility ratio of the dissipative zone is 4 where the "static ductility ratio" is the ratio of ultimate displacement over yield displacement derived from monotonic test results. Such a regulation is critical when dealing with timber structures for two main reasons. Firstly, a "yield displacement" is difficult to define as usually, there is no clear limit between elastic and plastic range. Secondly, the typical pinching behaviour of timber structures (see also Fig. 7 on the right) due to embedment deformations cannot be captured via monotonic tests. However, this pinching behaviour can potentially lead to a significant loss of energy dissipation capacity in repetitive cycles.

In order to establish behaviour factors for DLT buildings via numerical models, the chosen modelling approach has to correctly simulate the characteristics relevant for earthquake behaviour, building stiffness and load carrying capacity, damping, ductility, hysteresis and thus energy dissipation capacity. Furthermore, a valid near collapse criterion, i. e. a maximum interstorey drift, must be chosen. The chosen modelling approach has been described by Ceccotti and Sandhaas [7] and is based on the "Florence pinching hysteresis model" [18]. The chosen approach consists of the following steps:

1. Calculation of seismic base shear for the case study building with a chosen PGA_{design} value and assuming linear-elastic building behaviour (behaviour factor q = 1).
2. Design of case study building with seismic base shear of previous step → determination of necessary shear wall length, i. e. wall length is just enough to withstand earthquake.
3. Quasi-static cyclic testing of shear walls that mirror exactly the shear walls used in the case study building and transfer of their hysteretic behaviour into models (if final building model is 3D, also information on floor diaphragm behaviour is necessary). All hysteretic behaviour of shear walls is assigned to nonlinear springs ("Florence pinching hysteresis model") that connect rigid bars.

4. Generation of a 2D or 3D building model of case study building using the calibrated hysteretic springs from the previous step, building masses are applied as lumped masses. The calibrated spring model based on shear wall tests on short walls must be transformed to reflect the necessary larger shear wall length as determined in step 2.
5. Execution of nonlinear dynamic calculations in the time domain by applying different accelerograms (the chosen accelerograms, 10 natural and 10 synthetic earthquakes, are listed in Table 5). The respective peak ground acceleration values are increased until a previously defined near collapse state is reached → PGA_u.
6. Determination of the behaviour factor q for the modelled case study building and the single earthquakes as ratio of PGA_u over PGA_{design}.

With this methodology, the energy dissipation capacity of buildings is explicitly considered when determining the behaviour factor. The determined behaviour factor is valid for force-based design methods and is not representing the "true" behaviour factor, as the necessary design (steps 1 and 2) is based on linear-elastic calculations [19]. However, the chosen approach is reliable and precise enough as could be shown by Pozza et al. [19] who established the "true" behaviour factor of a CLT building (q = 2.7-3.5) which has not been significantly different from the behaviour factor evaluated with the above method on the same CLT building (q = 2.5-4.6, considering different earthquakes however) [20]. Also a benchmark study carried out on a two-storey US-American timber frame building could confirm the validity of the chosen approach [21].

The failure modes observed in the cyclic tests with predominant horizontal shear deformation lead to the model approach shown in Fig. 7 on the left. A shear wall is modelled with rigid bars forming a frame and all constitutional behaviour is assigned to rotational springs. The model shown in Fig. 7 on the left cannot model rocking behaviour, only horizontal deformations can be modelled. The "Florence pinching hysteresis model" shown in Fig. 7 in the centre [18] has been chosen to model the hysteresis curves of the rotational springs which is already implemented in the DRAIN software used to carry out the nonlinear dynamic analyses [22]. The spring model is able to correctly simulate the pinching behaviour of typical timber joints. Furthermore, only nine clearly identifiable parameters (U1, U2, K1 to K6, F0, see Fig. 7 in the centre) are needed to define the hysteretic behaviour. The model however cannot model the strength degradation in repeating cycles. The springs were iteratively calibrated by "repeating" the cyclic tests, until envelope curve and dissipated energy between model and test did not differ considerably (difference in dissipated energy < 6%). Fig. 7 on the right shows the calibration result for the rotational springs used to model the hysteretic behaviour of series J.

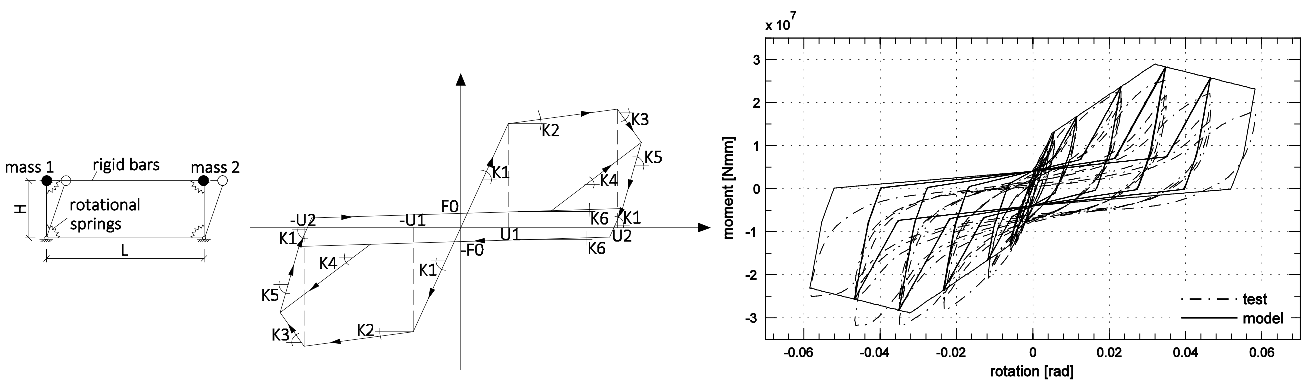

Fig. 7 – Left: Model approach for shear walls with predominant horizontal shear deformation.
Centre: "Florence pinching hysteresis model", spring law with 6 inclinations [18].
Right: Superposition of test and model for series J.

The calibration procedure is described in Ceccotti and Sandhaas [7] and has been carried out for all three tested DLT shear walls with different fastening solutions (series G to K) and with 10 kN/m additional vertical load. Furthermore, as not only DLT buildings were investigated, typical shear walls of three other timber building systems, CLT with interspaces and stapled and nailed connections, PFTE and timber frame, all with 10 kN/m additional vertical load, have been transferred into the same shear wall model as also their load-slip behaviour has been governed by horizontal shear deformations (for test specimens and results see Schädle [23]). In order to investigate the seismic performance of the seven building typologies (three varieties of DLT, two varieties of CLT, PFTE, timber frame), a case study building has been modelled with, consequently, lumped masses and rigid bars connected with hysteretic rotational springs, see Fig. 8 on the right. The calibrated wall models were transferred into the building model which was then subjected to increasing PGAs until the near collapse has been reached, PGA_u. The investigated DLT typologies were presented in section 3, the two varieties of CLT buildings, one with nailed and one with stapled connections, PFTE and timber frame are exhaustively presented in Schädle [23].

4.2 Case study building

The three-storey case study building is shown in Fig. 8 on the left. This building is transferred into the building model shown in Fig. 8 on the right where the following assumptions are taken:

- Only one-directional, horizontal shaking is investigated, shaking direction see Fig. 8 on the left.
- The case study building is modelled in 2D, any torsional effects and influence of floor diaphragms are neglected. The total building mass and the seismic forces per storey are distributed on the three walls acting in shaking direction.
- Similar to the shear walls, only horizontal deformations are possible, any uplift is neglected, see also deformed model in Fig. 8 on the right.

The masses for the case study buildings have been evaluated assuming timber beam floors with an additional load of 2 kN/m^2 and are given in Table 3. The seismic base shear has been calculated with the lateral force method given in Eurocode 8 [6] for the constant ordinate of the design spectrum (conservative) and with a soil parameter of S = 1, PGA_{design} of 3.5 m/s^2 for the (most severe) earthquake zone 1 in Italy, an importance factor $\gamma_I = 1$ and a purely linear-elastic behaviour factor of q = 1. The seismic base shear has then been distributed on all storeys depending on storey height and mass, leading to the horizontal forces given in Table 3.

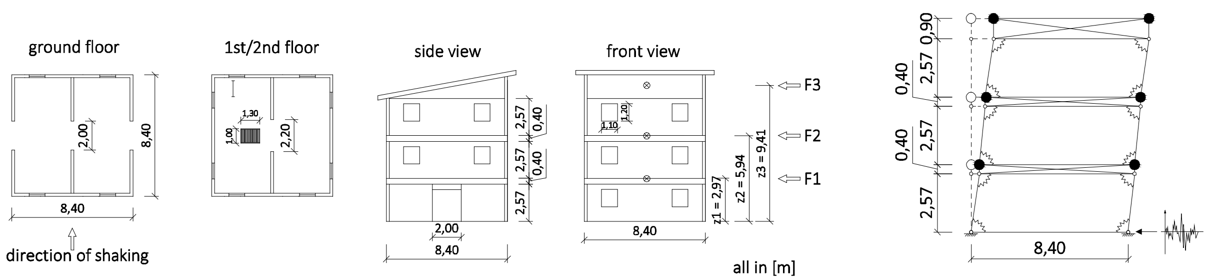

Fig. 8 – Left: Case study building. Right: 2D model of case study building.

Table 3 – Building mass and seismic force per storey.

Timber building system	Building masses [t]			Horizontal seismic forces [kN]		
	GF	1st	2nd	GF	1st	2nd
DLT	34	34	32	124	248	370
CLT with interspaces (stapled and nailed)	32	32	30	119	237	345
PFTE	32	32	30	117	234	344
Timber frame technique	30	30	28	112	225	322

Conversion of spring properties

In the chosen approach, all hysteretic behaviour of the modelled building has been assigned solely to the rotational springs. All other structural elements have been programmed to be infinitely rigid. The springs have been calibrated by means of test results on 2.50 m long shear walls. In buildings however, the shear walls are longer and therefore, the calibrated spring properties must be converted as longer shear walls will have different stiffness and strength properties. Such a conversion represents a difficult step that is based either on experimental evidence or on assumptions. In particular, three issues are of interest:

- Determination of necessary shear wall length during the design of the case study building:
  The horizontal load at a horizontal displacement on top of wall of H/150 reached during the shear wall tests has been taken as "design load carrying capacity" of the walls at which the wall just withstands any lateral load. With this value the necessary length of a shear wall loaded with a horizontal seismic force can be calculated. H/150 is the recommended maximum deformation of a cantilever beam.
- Conversion from 2.50 m shear walls to longer shear walls:
  A linear-proportional relationship between shear wall length and stiffness is assumed, i. e. a shear wall of 2 m length has two times the stiffness of a shear wall of 1 m length. Such an assumption is realistic for systems whose lateral stiffness and strength is mainly due to continuous staples or nails along the perimeter of the sheeting and which deform rhombically.
- Influence of additional vertical load:
  As no clear trends on the influence of vertical loads could be observed during the tests and in real buildings, some vertical load will always be present, the calibration has been carried out for the wall tests with 10 kN/m additional vertical load.

Damping and friction

Contribution of friction to energy dissipation is implicitly included in the hysteretic spring model as the springs represent the overall load-slip behaviour of the shear walls which is based on both frictional effects and deformations of the joints. Damping however, the third important dynamic parameter besides mass and stiffness, is difficult to assess. A value of 5% has been assumed here in order to obtain conservative values.

Near collapse criterion

A maximum interstorey drift of $u_{max} = 0.02 \cdot h$ has been considered as near collapse criterion of the case study building. With a storey height of h = 2570 m, this criterion results in a near collapse criterion of 51 mm interstorey drift. The chosen near collapse criterion has a strong influence on the derived behaviour factors. If larger interstorey drifts are accepted, PGA_u and thus behaviour factor increase accordingly.

First Eigenperiod T_1

The Eigenperiods of the six case study buildings have been calculated and are given in Table 4. The elastic stiffness of timber structures, K1 in Fig. 7 on the right, is subject to natural scatter; slight changes in K1 are reflected in changing Eigenperiods. Therefore, not only the simplification of the case study building in a 2D building model where the modelling input is derived from shear wall tests is not considered when calculating the Eigenperiods, but also the uncertainties when establishing the value for K1. Furthermore, the derived first Eigenperiod T_1 is needed to calculate the stiffness-proportional Rayleigh damping used in the FE package DRAIN [22] and thus influences also the nonlinear dynamic modelling results.

Table 4 – First Eigenperiod T_1 for case study buildings.

Timber building system	Eigenperiod T_1	Timber building system	Eigenperiod T_1
DLT with joint series G	0.294 s	CLT with interspaces and nails	0.404 s
DLT with joint series K	0.351 s	CLT with interspaces and staples	0.399 s
DLT with joint series J	0.415 s	PFTE	0.328 s
Timber frame technique	0.325 s		

5 Behaviour factors

Table 5 lists all derived behaviour factors q as the ratio of PGA_u reached during dynamic analyses over $PGA_{design} = 3.5\ m/s^2$. The first conclusion is that the evaluated behaviour factors do not differ considerably between the six analysed building systems, but rather differ considerably between used accelerograms. Especially the high values for the Friuli earthquake are remarkable which can be explained with the earthquake's response spectra [23]. The spectra exhibit the highest accelerations for periods below 0.2 s which decrease rapidly for longer periods in the range of the Eigenperiods of the investigated buildings. Generally, the behaviour factors are lower for synthetic earthquakes.

Table 5 – Behaviour factors q for 5% damping.

Earthquake (Station), Date	DLT Series G	DLT Series K	DLT Series J	CLT Nails	CLT Staples	PFTE	Timber frame
Natural earthquakes							
Roermond (Bergheim), 13.04.1992	4.3	4.3	2.9	3.6	3.9	4.5	4.0
L'Aquila NS (FA030), 06.04.2009	6.0	5.7	4.7	5.4^+	5.7^+	5.7	4.7
L'Aquila EW (FA030) , 06.04.2009	4.4	5.0	3.4	4.3	4.5	5.4	5.1
L'Aquila NS (GX066) , 06.04.2009	5.5	4.9	4.4	4.5	4.9	5.3	4.7
L'Aquila EW (GX066) , 06.04.2009	4.6	4.5	5.0	4.2	4.4	4.4	4.2
L'Aquila NS (AM043) , 06.04.2009	5.5	5.6	3.9	5.2	5.5	6.0	4.8
L'Aquila EW (AM043) , 06.04.2009	4.4	4.6	3.6	4.3	4.6	5.0	4.4
Friuli NS (Feltre), 06.05.1976	9.2	9.7	7.6	9.7^+	8.9^+	10.8	8.6^+
Friuli EW (Feltre), 06.05.1976	9.1	9.8	7.8	9.5	9.8	10.5	9.9
Lazio Abruzzo (Atina), 07.05.1984	3.5	3.6	2.5	3.0	3.2	3.7	3.3
Synthetic earthquakes (derived for response spectrum according to Eurocode 8 [6])							
SYNT_1	4.3	4.5	3.6	3.5	3.8	4.1	3.9
SYNT_2	4.3	4.4	3.5	4.3	4.4	4.3	4.0
SYNT_3	4.3	4.5	3.5	3.5^+	3.9^+	4.5	3.7
SYNT_4	3.6	3.7	3.5	3.5	3.5	3.7	3.5
SYNT_5	4.1	3.9	3.9	3.1	3.4	4.2	3.0
SYNT_6	4.6	4.4	4.2^+	3.0	3.3	3.3	2.6
SYNT_7	4.1	4.2	3.6	4.0	4.3	4.5	4.0
SYNT_8	5.9	5.5	4.6^+	3.3	3.5	4.3	3.7
SYNT_9	4.1	4.1	3.5	4.1	4.2	4.3	3.7
SYNT_10	3.6	4.0	2.7	3.3	3.5	4.3	3.2^+

* Failure on 1st floor, $^+$ failure on 2nd floor, all others failure on groundfloor.

6 Conclusions

The general behaviour of joints and shear walls used in the DLT building system has been investigated and the influence of inherent gaps between lamellae und different sheeting material on the load carrying capacity of both joints and shear walls has been assessed. Therefore, relevant knowledge gaps hampering the application of DLT building systems could be closed. The load carrying capacity of joints in DLT needs to be reduced in dependence of the fastener diameter. The case of earthquake loading on timber building systems has been investigated as well. Behaviour factors needed in force-based design methods have been derived obtaining values ranging from 2.5 to 10.8 for natural earthquakes and considerably lower values between 2.6 and 5.9 for synthetic earthquakes. No significant differences could be observed between the six discussed timber building systems. The behaviour factors are conservative, a severe near collapse criterion, low damping and only the constant branch of the design spectrum when designing the case study buildings were considered. The evaluated behaviour factors are valid only for force-based approaches which however will remain important also in future as force-based design keeps design effort small, no extensive database is needed and the discussed timber building systems are mostly used for regular residential buildings of up to 3 to 4 storeys.

7 Acknowledgements

The DLT project has been financed by the German Federal Ministry for Economic Affairs and Energy within the "ZIM"-programme under grant ID KF2007011US2. Cooperating SME partner has been Kaufmann GmbH.

8 References

[1] Krämer V (2003) *Trag- und Verformungsverhalten genagelter Brettstapelelemente unter Querlast*. Dissertation. Universität Karlsruhe (TH), Germany.

[2] Schänzlin J (2003) *Zum Langzeitverhalten von Brettstapel-Beton-Verbunddecken*. Dissertation. Universität Stuttgart, Germany.

[3] EN 1995 1-1 (2010) *Eurocode 5. Design of timber structures - Part 1-1: General -Common rules and rules for buildings*. Comité Européen de Normalisation (CEN), Brussels, Belgium.

[4] Winter S, Kreuzinger H and Mestek P (n. d.) *Flächen aus Brettstapeln, Brettsperrholz und Verbundkonstruktionen*. Holzbau der Zukunft, Teilprojekt 15. Technische Universität München, Germany.

[5] Sandhaas C and Blaß HJ (2016) *Statisches und dynamisches Verhalten von aussteifenden Wandscheiben in Brettstapelbauweise*. Karlsruher Berichte zum Ingenieurholzbau Band 30, ISBN 978-3-7315-0459-7. Karlsruhe Institute of Technology, Germany.

[6] EN 1998-1 (2010) *Eurocode 8. Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings*. Comité Européen de Normalisation (CEN), Brussels, Belgium.

[7] Ceccotti A and Sandhaas C (2010) A proposal for a standard procedure to establish the seismic behaviour factor q of timber buildings. *11th World Conference of Timber Engineering WCTE*. Riva del Garda, Italy.

[8] Ceccotti A, Sandhaas C, Okabe M, Yasumura M, Minowa C and Kawai N (2013) SOFIE project - 3D shaking table test on a seven-storey full-scale cross-laminated timber building. *Earthquake Engineering and Structural Dynamics*, **42**(13)**:**2003-2021. DOI 10.1002/eqe.2309.

[9] Pozza L (2013) *Ductility and behaviour factor of wood structural systems*. Dissertation. Università degli Studi di Padova, Padua, Italy.

[10] Pei S, Popovski M and van de Lindt JW (2012) Seismic design of a multi-storey cross laminated timber building based on component level testing. *12th World Conference of Timber Engineering WCTE*. Auckland, New Zealand.

[11] *CLT handbook* (2011) Gagnon S and Pirvu C, eds. FP Innovations, Québec, Canada.

[12] Görlacher R (2012) Berechnung von massiven Decken- und Wandelementen aus nachgiebig miteinander verbundenen Brettern. *Karlsruher Tage Holzbau - Forschung für die Praxis*. Karlsruhe, Germany.

[13] Sandhaas C, Boukes J, Van de Kuilen JWG and Ceccotti A (2009) Analysis of X-lam panel-to panel connections under monotonic and cyclic loading. *CIB-W18 Meeting 42 Paper 42-12-2*. Dübendorf, Switzerland.

[14] Schmidt T (2016) Kontaktverbindungen zur Längsschubübertragung in den Schmalseiten von Brettsperrholzelementen. *Doktorandenkolloquium*. Stuttgart, Germany.

[15] Blaß HJ and Schädle P (2009) *Aussteifende Wände in Einzelelement-Bauweise*. Karlsruher Berichte zum Ingenieurholzbau Band 13, ISBN 978-3-86644-334-1. University of Karlsruhe, Germany.

[16] EN 26891 (1991) *Timber structures - Joints made with mechanical fasteners - General principles for the determination of strength and deformation characteristics (ISO 6891)*. Comité Européen de Normalisation (CEN), Brussels, Belgium.

[17] EN 12512 (2001) *Timber structures - Test methods - Cyclic testing of joints made with mechanical fasteners*. Comité Européen de Normalisation (CEN), Brussels, Belgium.

[18] Ceccotti A and Vignoli A (1989) A hysteretic behavioural model for semi-rigid joints. *European Earthquake Engineering*, **3:**3-9.

[19] Pozza L, Scotta R and Vitaliani R (2009) A non-linear numerical model for the assessment of the seismic behaviour and ductility factor of X-Lam timber structures. *International Symposium on Timber Structures*. Istanbul, Turkey. pp. 151-162.

[20] Ceccotti A (2008) New technologies for construction of medium-rise buildings in seismic regions: The X-lam case. *Structural Engineering International*, **18**(2)**:**156-165. DOI 10.2749/101686608784218680.

[21] Folz B and Filiatrault A (2004) Blind predictions of the seismic response of a woodframe house: an international benchmark study. *Earthquake Spectra*, **20**(3)**:**825-851. DOI 10.1193/1.1774989.

[22] Prakash V and Powell GH (1993) *DRAIN-2DX, DRAIN-3DX and DRAIN BUILDING: Base program design documentation*. Department of Civil Engineering, University of California, Berkeley, USA.

[23] Schädle P (2012) *Innovative Wandbausysteme aus Holz unter Erdbebeneinwirkungen*. Dissertation, ISBN 978-3-86644-832-2. Karlsruhe Institute of Technology, Germany.