Bending and Vibration Behaviour of CLT-Steel Composite Beams

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Abstract

A strengthening of cross-laminated timber (CLT) by a composite effect with steel girders can widen the application of CLT ceilings to spans over 8 m. Most possible shear connectors are not stiff enough to ensure a completely rigid composite. At present, it has not been sufficiently clarified how the elastic bending behaviour is affected by the influences of the flexibility of continuously and discontinuously shear connectors, the number of transverse layers of the CLT and the span width. Thus, 4-point bending tests and vibration tests were performed with different cross-section configurations and two different shear connectors in continuous and discontinuous spacing in spans of 8.10 and 10.80 m. To date, no comparable bending tests have been carried out in these spans, with more than five CLT-layers and discontinuously arranged shear connectors.

The composite beams deformed linear-elastically until the yield strength of the steel was reached. The composite effect increased the elastic bending stiffness up to twofold compared to no composite. Increasing the span resulted in a higher bending stiffnesses. The elastic bending stiffness of the composite beams with shear studs was significantly lower than with fully threaded screws.

For a worthwhile composite effect, both materials should contribute a balanced share of the stiffness. A larger share of the CLT in the bending stiffness compared to the steel girder created a higher elastic limit load capacity but an equivalent bending stiffness. It is necessary to discuss which cross-sectional configurations are appropriate in terms of load bearing capacity, economic efficiency and sustainability.

To assess the practical application potential in spans between 8 and 12 m, the tests were additionally evaluated for the equivalent load level for the serviceability limit state of office or industrial buildings. For spans of 8.10 m, the limits according to EC5 for the initial deflection of L/300 and fundamental frequency of 8 Hz can be met. For spans of 10.80 m, only less strict deflection limits are achieved. However, by increasing the degree of composite through more shear connectors, compliance with the limit values mentioned could already be possible with the cross-sections tested. In case of fire, it may be sufficient to consider only the CLT with the reduced cross-section method (EN 1995-1-2) for load transfer, even for longer fire durations.

Keywords: steel-timber-composite; cross-laminated timber; composite bending capacity; composite bending stiffness; vibration behaviour.

Introduction

Highly loaded ceilings of residential, office and industrial buildings are often designed as concrete systems or concrete-steel composite beams. The production of concrete is energy-intensive and requires exhaustible resources. The shear connection of composite beams is usually achieved by shear studs and enables a significantly higher bearing capacity than if the slab is only loosely placed on the girder.

Cross-Laminated Timber (CLT) is a sustainable material for ceilings but is limited in its application. Replacing the concrete slab of a composite beam with CLT would create a lighter alternative made from renewable or fully recyclable materials. A CLT-steel composite beam (Fig. 1) would extend the application range of CLT to areas with larger spans and higher loads. The efficien-

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Fig. 1
CLT-steel-composite beam

cy of the composite depends to a large extend on the bearing capacity and the flexibility of the shear connection. For this reason, Hassanieh et al. (Hassanieh, Valipour and Bradford, 2016a; Hassanieh, Valipour and Bradford, 2017a) and Loss et al. (Loss et al., 2016) developed various shear connecting systems using screws, bolts, tooth plates, nailing plates, steel components or glue and investigated them in push-out tests. Based on this, Hassanieh et al. studied the bending performance of composite beams of steel girders and CLT or laminated veneer lumber in 4-point bending tests with spans of 3 to 6 m with varying shear connectors. The ultimate bearing capacity was mainly limited by the failure of the shear connectors. The CLT-steel composite beams were able to achieve bending stiffnesses between 25,000 and 35,000 kNm^2 (Hassanieh et al., 2016b; Hassanieh et al., 2017b). Prior to the bending tests, vibration tests were carried out, which showed a fundamental frequency of 20 to 24 Hz for a span of 5.80 m (Chiniforush et al., 2017). In numerical investigations calibrated with this, a fundamental frequency of about 10 Hz was obtained for a span of 8 m (Chiniforush, 2018).

Loss et al. also carried out bending tests on CLT-steel composite beams with spans of about 6 m with cold formed steel profiles in U and Omega form and with glued-in perforated plates as well as vertically inclined fully threaded screws. U-sections with glued-in steel plates were able to achieve a share of the composite effect in the achieved bending stiffness of 71 %, omega-sections with fully threaded screws a share of 46.5 % (Loss et al., 2017; Owolabi et al., 2022).

Merryday et al. (Merryday et al., 2023) carried out a larger-format 4-point bending test at a span of 9.14 m with a narrow CLT element of a width of 0.483 m. Analogous to their push-out tests on vertically arranged screws, the slip in the composite joint was so small in the first 10% of the test load that a rigid composite was achieved. Under a further load, the expected non-rigid composite effect occurred.

The previously published work on the bending behaviour of CLT-steel-composite beams was mainly limited to shorter spans up to about 6 m, thinner CLT with a maximum of five layers, a continuous arrangement of shear connectors and a smaller scope of testing. The influence of the flexibility of continuously and discontinuously arranged shear connectors and of the shear deformation of the transverse layers on the elastic bearing capacity for spans over 8 m needs to be evaluated. 15 large-scale laboratory tests were conducted. The tests are based on previous studies on inclined fully threaded screws and shear studs casted-in CLT-openings as shear connectors (Böhm et al., 2023).

This work presents the results of the short-term elastic bearing capacity, the elastic bending stiffness and the vibration behaviour of CLT-steel composite beams in application scenarios of spans between 8 and 11 m. The mechanical influences of the previous mentioned parameters on the bearing behaviour are quantified. The various design options for CLT steel composite beams in terms of cross-section setups and shear connectors are to be demonstrated and evaluated.

Based on the test results, the application potential for a load level occurring in office or industrial buildings with spans of 8.10 m and 10.80 m is discussed. In timber construction, the serviceability is mostly decisive for the design. The limit values for deflection and vibration are often reached at significantly lower loads than the mechanical failure. Therefore, the load-bearing behaviour is evaluated particularly for the serviceability limit state.

Methods

Bending test programme

To achieve the objectives formulated above, five different test series were conducted. The first test series (C1.1) represents the basic configuration. A composite beam consisting of a five-layered CLT200 element and a hot rolled steel section HEA 200 was tested in a span of 8.10 m. Fully threaded screws were used as shear connectors. In the subsequent test series, the initial configuration was basically retained. One parameter (span width, cross-sectional structure and shear connectors) was varied in each case. Each configuration was tested three times identically. The test programme is shown below.

Table 1

Test programme of the bending tests

Test series	Quantity	Span	CLT	steel	Shear connectors
C1.1	3	8.1 m	CLT200 L5s 1.50 x 8.90 m	HEA200 S355	Inclined fully threaded screws Ø10x155 mm
C1.2	3	10.8 m	CLT200 L5s 1.50 x 11.6 m	HEA200 S355	Inclined fully threaded screws Ø10x155 mm
C1.3	3	8.1 m	CLT240 L7s 1.5 x 8.9 m	HEA160 S355	Inclined fully threaded screws Ø10x155 mm
C1.4	3	10.8 m	CLT240 L7s 1.5 x 11.6 m	HEA160 S355	Inclined fully threaded screws Ø10x155 mm
C2.1	3	8.1 m	CLT200 L5s 1.5 x 8.9 m	HEA200 S355	Cast-in shear studs Ø22x90 mm

Bending test setup and testing method

Due to the lack of guidelines for CLT-steel composite beams, the tests were based on the specifications for glulam and CLT according to EN 408 (European Committee for Standardization, 2012) and EN 16351 (European Committee for Standardization, 2021). The preloading procedure was adapted from previous bending tests of Hassanieh et al. (Hassanieh et al., 2017b). The tests were conducted as 4-point bending tests with an excess end on both hinged supports of 40 cm. The two load introduction points were aligned centrally with two metres distance between each other. Directly under the load application points, 10 mm thick steel plates with a base area of 10 cm x 150 cm were screwed onto the CLT surface. The load was initially force-controlled up to a preload of 40 % of the estimated maximum load. After a holding time of 30 seconds, unloading to 10 % of the estimated maximum load, the main load was applied until failure or a deformation path of 210 mm. The main load was applied with a displacement control at 5 mm/min. As shown in Fig. 2, the vertical deflections at midspan and under the load application points, as well as the horizontal slip in the composite joint at supports and quarter points were documented with path transducers. The strains at the lower flange, upper flange and web of the steel girder, as well as the lower and upper side of the CLT were recorded in each case at mid and quarter span with strain gauges. Additional vibration tests were carried out in the unloaded state. Analogue to the vibration tests on timber slabs in (Winter et al., 2010), the vibrations were induced in various ways. In addition to human walking, the excitation was also achieved by hitting the CLT surface with a rubber hammer or a fist and dropping a 5 kg medicine ball. The excitation at mid and quarter span was investigated. The vibration acceleration was recorded at mid and quarter span.

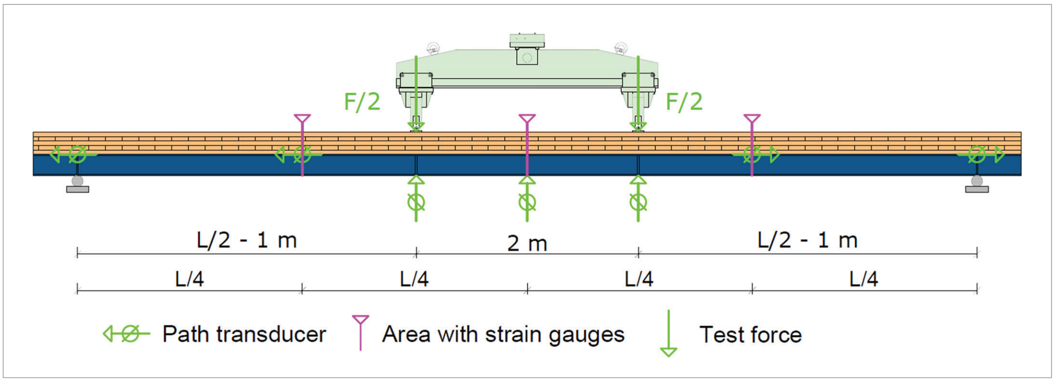


Fig. 2
Test set-up and measuring technique of the 4-point bending tests

Shear connectors

The test programme includes two different shear connectors. Since the shear connectors are crucial for the bending behaviour of the composite beams, the investigations of the used shear connectors from (Böhm et al., 2023) are briefly described hereinafter. Both shear connectors were investigated in double push-out tests. The test setup was based on the Standard Push Out Tests for shear studs of EN 1994-1-1 Annex B.2 (European Committee for Standardization, 2010a) and modified for CLT and screws.

The first system consists of self-drilling fully threaded screws, that are screwed upwards into the CLT through pre-drilled holes in the steel girder top flange. The arrangement of the screws is shown in Fig. 3 (left) in the setup of the double push out-tests. The screws are inclined in the vertical plane by the angle α, so that the screws are axially stressed. Consequently, the load-bearing capacity and the stiffness of the screws increase in contrast to a right-angled arrangement. A further improvement can be achieved by increasing the screw length. This is prevented for ordinary I-sections by the limited space for screw and drill between top and bottom flange. To solve this, the inclined fully threaded screws were rotated in the drilled hole by an angle β in the horizontal plane. Thus, each screw can be screwed in past the bottom flange. The composite force therefore stresses the screw at angles in the horizontal and vertical planes so that a multidimensional loading situation occurs. In previous investigations (Böhm et al., 2023), the influence of the screw length and different rotation angles β in the horizontal plane could be evaluated for a constant angle α. Up to an angle of rotation in the horizontal plane of about β = 30°, the use of longer screws resulted in higher load capacities and stiffnesses compared to shorter screws at lower angles β. At larger angles β, the stiffness of the screws decreased significantly, so that the use of longer screws was not worthwhile. In the bending tests, fully threaded screws Ø10 x 155 mm, type KonstruX by Eurotec, with angle α of 45° in the vertical plane and an angle β of 30° in the horizontal plane were used. Using wedge-shape washer, type Taurus 45°, ensured that the shear studs are fully supported in the girder hole despite the inclination.

As a second shear connection, shear studs were adapted from the concrete composite beam. As it is shown on **Fig. 3** (right) circular openings with a diameter of 15 cm are milled into the CLT, placed centrally around the shear stud on the steel girder. The opening is then filled with a high-strength mortar of type V1/50 from Pagel with compression strength class C60/75. Shear studs from Köster & Co. with a yield strength of 375 N/mm² and a tensile strength of 470 N/mm² were used. The shear studs showed comparable load-bearing mechanisms as in continuous concrete slabs. The connection remains load-bearing even after the mortar has cracked, as the CLT edge keeps the mortar cylinder in shape. There is no significant deformation of the CLT edge.

Further test results of fully threaded screws and shear studs are described in (Böhm et al., 2023).

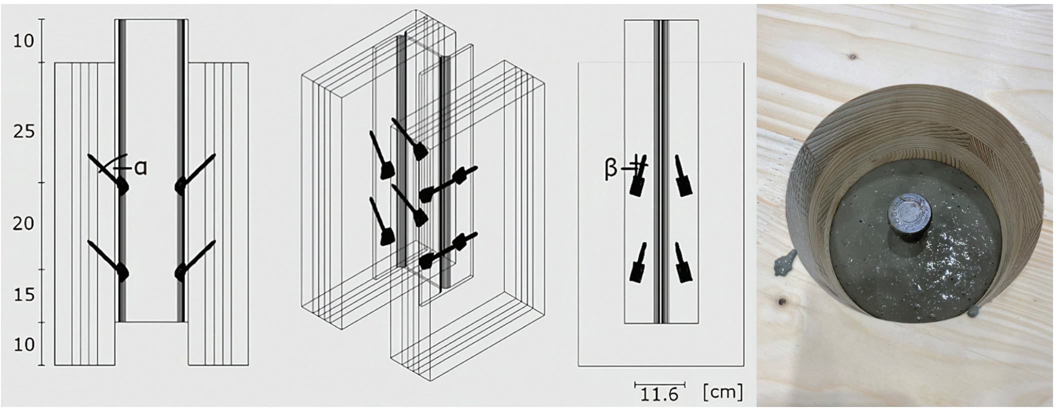


Fig. 3
Multi-dimensionally inclined fully threaded screws in double push-out tests (left), grouting process of the shear studs in CLT openings (right)

Material properties

CLT elements from the manufacturer HBS Berga with material properties according to ETA-20/0860 were used. The elements were made of European spruce wood of strength class C24. The CLT elements in test series C1.1, C1.2 and C2.1 had the layer-structure 40l-40q-40l-40q-40l, while those in series C1.3 and C1.4 had the layer-structure 40l-30q-40l-20q-40l-30q-40l. A moisture content of 9,8 % ± 0.5 % was measured for all CLT elements, which is within the usual range for CLT in service class 1. For the steel girders HEA160 and HEA200 hot rolled sections, of steel grade S355JR, were chosen. The material properties of CLT and steel were investigated in additional tests and are presented afterwards. For the CLT, the mean values and the characteristic values (5%-quantile) from material tests of the test specimens are given in Table 2. The compression, tensile and bending tests were conducted on individual lamellas. The mean values of the modulus of elasticity, the tensile strength and the yield strength of the steel are shown in Table 3.

Table 2
Material properties of test specimens for CLT

	f_m	$f_{c,0}$	$f_{t,0}$	E_0	
Mean value	36.6	28.3	20.0	12,774	N/mm²
5% quantile	26.0	20.2	16.1	10,443	N/mm²

Table 3
Material properties of test specimens for steel

	f_u	f_y	E	
Mean value	557.2	395.3	209,585	N/mm²

Test Results and Discussion

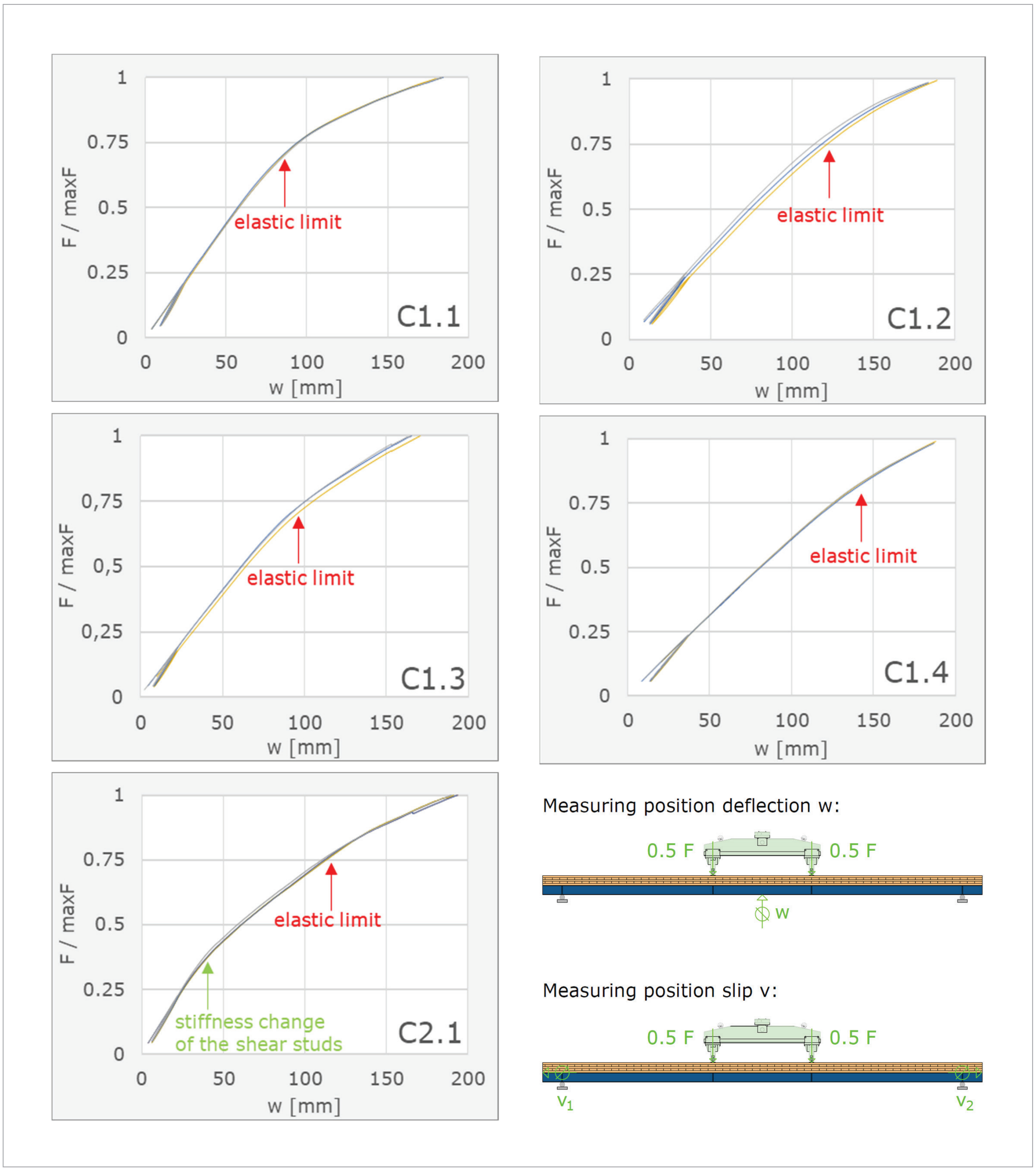
Failure modes and deformation behaviour

In all test series, the load-bearing behaviour of the composite beams was distinctly linear elastic until the yield strength of the steel was achieved. The elastic limit load-bearing capacity occurs at about 70 to 76 % of the maximum test load, as depicted on the load-displacement diagrams below. With a further load increase, the steel girder deforms significantly plastically until a tensile failure on the underside of the CLT occurs.

The fully threaded screws used in the test series C1.1 to C1.4 as shear connectors deformed exclusively elastically and could be simply unscrewed after the end of the test. Due to the constant composite joint stiffness, the composite force was transferred constantly between the materials during the elastic deformation of the composite beam. As verifiable on **Fig. 4** and **Fig. 5**, the vertical deflection w and the horizontal slip v can be described as a bilinear curve with a transition point at the yield point of the steel. Different to that, a trilinear load-bearing behaviour can be revealed in the diagram for the vertical deflection of test series C2.1 (**Fig. 4**). This is caused by the deviating deformation behaviour of the shear studs used as shear connectors in these tests. At about 30 % of the maximum test load, the mortar around the shear studs cracked and the stiffness of the connection was thus significantly reduced. However, under further loading the cracked mortar cylinder is held in shape by the surrounding CLT. Consequently, a constant stiffness can still be achieved again but on a lower level. This ensures a further shear force transmission. The change in stiffness is clearly visible in the diagram (**Fig. 5**, right) of the horizontal slip in the composite joint for test series C2.1. This drop in stiffness does not occur in the composite joint of the remaining tests, shown in **Fig. 5** on the left as an example for C1.1 with fully threaded screws.

Fig. 4

Load displacement curve for the deflection w at midspan and the measuring positions for the load displacement curves



Hassanieh et al. (Hassanieh et al., 2017b) observed five distinctive failure modes for composite beams made of steel and CLT in spans up to 6 m. The occurrence of the individual modes depended on the shear connectors used. Either the shear connectors failed, or there was a brittle tension failure of the underside of the timber. In all their tests, a brittle rolling shear failure occurred in the CLT, but had no direct influence on the load-bearing capacity. The failure of the shear connectors was ductile for screws and bolts and brittle for mortar and glue. In addition, the cold-formed profiles of Loss et al. (Loss et al., 2017; Owolabi et al., 2022) failed in part due to local buckling.

The test results thus confirm the finding of Hassanieh et al. that the failure modes depend significantly on the shear connectors. Even with spans of up to 11 m, brittle tensile failure of the timber occurs if the limit load of the shear connector is not reached. Rolling shear failure in CLT, as well as local buckling of the steel could not be confirmed. In principle, a ductile failure should be aimed for, as this heralds for a loss of bearing capacity. Here, the brittle timber failure is preceded by the plasticising of the steel girder and can therefore be recognized beforehand by a very ductile bearing phase. To completely rule out brittle, unannounced failure, the shear connectors must either not fail or should do so in a ductile manner. The fully threaded screws did not reach their maximum forces in the tests and would then fail ductilely. The maximum forces of the cast-in shear studs were achieved. However, unlike Hassanieh et al., the cracking in the mortar did not lead to brittle failure.

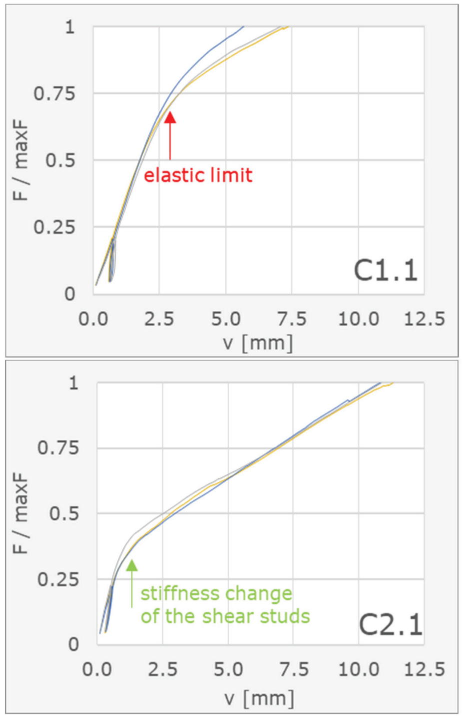


Fig. 5

Load deformation curve for the slip v as the average of v_1 and v_2 in the composite joint

Elastic limit bearing capacity

As previously described, the failure of the composite beams is characterised by the steel exceeding its yield strength before a fracture occurs at the bottom edge of the CLT. The elastic limit load-bearing capacity of the test specimens is thus defined by the steel achieving its yield strength. In Table 4, the elastic maximum load and the corresponding deflection w at midspan and the slip v in the composite joint at the supports are listed. These are average values for each test series. The results of test series C1.1 are set as a reference configuration and are compared to the other test series.

Table 4

Elastic maximum load F and corresponding midspan deflection w and slip v

Test series	Span	Configuration	F [kN]	w [mm]	v [mm]
C1.1	8.1 m	CLT200 L5s + HEA200	214.48	85.78	2.97
C1.2	10.8 m	CLT200 L5s + HEA200	149.36	125.24	2.01
C1.3	8.1 m	CLT240 L7s + HEA160	258.33	100.68	3.42
C1.4	10.8 m	CLT240 L7s + HEA160	159.41	138.67	2.44
C2.1	8.1 m	CLT200 L5s + HEA200	209.64	105.35	5.63

The use of a more massive CLT element and a smaller steel girder with the same overall construction height leads to an increase of the elastic limit load by about 20 %. Even at the load level of the elastic limit of test series C1.1 (F = 215 kN), the deviating cross-section configuration of C1.3 leads to an approximately 10 % lower vertical deflection at midspan (w = 79.03 mm). The comparison of the stress diagrams of both test series in **Fig. 6** shows clearly that the flexibility of the shear connectors leads to a non-continuous normal stress curve. The significantly higher stiffness share of the CLT in test series C1.3 put the steel girder almost completely under tensile stress. The zero-stress line of the girder shifted nearly in the composite joint. The higher stiffness share of the steel girder in test series C1.1 created a clearer compressive stress zone in the girder. At the point of reaching the steel yield strength, the maximum normal stresses in the CLT are higher in test series C1.3 than in C1.1. The change in the cross-sectional configuration to a smaller steel girder and a more solid CLT element thus leads to a higher stress on the CLT. At the point the steel yield strength was reached, the maximum normal stresses in the CLT also approached the maximum strength values. The entire composite beam was then efficiently activated. However, the greater the stiffness of the CLT, the closer the yield strength of the steel and the brittle tensile failure of the CLT came to each other. A greater steel stiffness therefore ensures a ductile failure of the composite beam. The effects of different cross-section configurations on the elastic load-bearing behaviour are discussed in more detail in the following chapter.

The use of cast-in shear studs as shear connectors in series C2.1 shows a premature loss of stiffness in the composite joint, as explained by means of **Fig. 5**. As a result, a similar load level is achieved in the elastic area, but the deflection w at midspan is 30 % higher and the slip v in the composite joint is 105 % higher compared to the tests with screws as shear connectors. The higher horizontal slip in the composite joint, when achieving the elastic limit load capacity of the composite beams, leads to a significantly lower composite effect compared to the other test series. As a result, the zero-stress lines in the slab and girder move further away from the composite joint, as can be seen in **Fig. 6**. In the case of no composite, the zero-stress lines are in the shear centre of the individual cross-sections. With a full composite, there is only one zero-stress line across the entire composite beam.

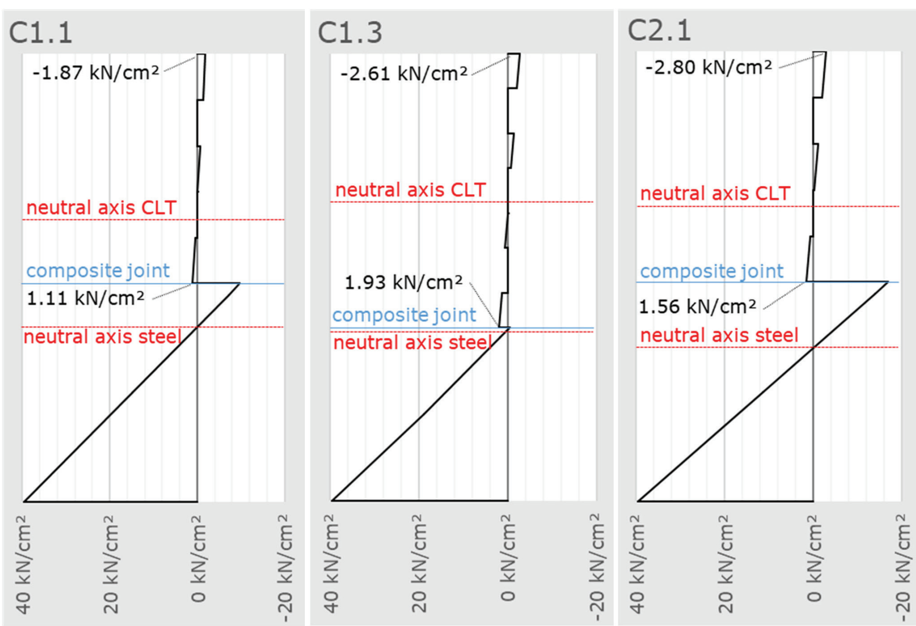


Fig. 6

Normal stress curve along composite beam cross-section for test series C1.1, C1.3 and C2.1 (positive values = tension, negative values = compression)

Bending stiffness

The global bending stiffness was calculated from the test results in accordance with EN 408 10.3 (European Committee for Standardization, 2012) and is shown in Table 5. The elastic bending stiffness is calculated firstly for the range between 10% of the overall maximum load and the elastic maximum load (EI_{elast}) and secondly for the initial range between 10% and 40% of the maximum load ($EI_{40\%}$). For the bending stiffness up to the elastic maximum load (EI_{elast}), the degree of composite n_{elast} is determined as a function of the values for an analytically calculated full composite ($EI_{k=\infty}$) and without any connection ($EI_{k=0}$) according to the following equation:

$$\eta_{elast} = \frac{\mathrm{EI}_{\mathrm{elast}} - EI_{k=0}}{EI_{k=\infty} - EI_{k=0}} \quad (1)$$

The comparison of the bending stiffnesses of the identical systems C1.1 and C1.2, or C1.3 and C1.4, shows the effect of the span. The 2.70 m larger span increases the bending stiffness by about 20 % to 23 %. A reason could be the effective width of the CLT, which can form better with larger spans.

In general, the initial bending stiffness $EI_{40\%}$ is, as expected, greater than that in the entire linear-elastic range. The initial bending stiffness $EI_{40\%}$ of test series C2.1 with shear studs as shear connectors is only slightly lower than that of the comparable specimen with screws (C1.1). Over the entire elastic range, a significantly lower bending stiffness is achieved with the shear studs due to the early cracks in the mortar at about 30% of the maximum load. Shortly before cracking of the mortar, the bending stiffness of test series C2.1 of $EI_{30\%}$ = 30,742 kNm² is even higher than in C1.1 with screws. It should be noted that the shear studs cannot carry any uplifting loads, which occur especially with larger vertical deformations. Therefore, a small number of screws or bolts could be used to prevent the composite joint from opening. Alternatively, a direct transfer of the uplifting loads directly between CLT and mortar could be secured by screws drilled into the timber and mortared or a mortar shape tapering upwards.

Despite the different cross-sectional configurations, the test series C1.1 and C1.3 or C1.2 and C1.4 achieve different elastic limit loads (compare Table 4), but similar bending stiffnesses. While the test series C1.1 and C1.2 achieve degrees of composite of 51 % and 71 %, the degrees of composite are 50 % and 73 % for the test series C1.3 and C1.4 with more solid CLT and smaller steel girders. The degrees of composite are obviously similar, but only represent a ratio between full and no composite. The actual increase in bending stiffness in test series C1.1 and C1.2 compared to no composite is significantly greater than in C1.3 and C1.4. In test series C1.1 and C1.2, the bending stiffness increases by around 3,000 kNm² more, due to the different stiffness distribution between steel and CLT with an identical shear connection. With an increasing CLT stiffness, more shear force is transmitted through the composite joint. The flexibility of the shear connectors causes a greater loss of stiffness than if the steel girder provides most of the stiffness. For the same increase in bending stiffness in absolute values, a CLT slab that is stiffer relative to the steel girder will require significantly more shear connectors. Since the spacing of the shear connectors used in the bending tests was significantly greater than the minimum distances, there is still a clear potential to increase the degree of composite and thus the bending stiffness.

Table 5

Global bending stiffness EI_{elast} and $EI_{40\%}$ and degree of composite action nelast and $\eta_{40\,\%}$

Test series	Span	Configuration	EI_{elast} [kNm²]	n_{elast} [%]	$EI_{40\%}$ [kNm²]	$n_{40\%}$ [%]
C1.1	8.1 m	CLT200 L5s + HEA200	26,058	51	29,231	67
C1.2	10.8 m	CLT200 L5s + HEA200	30,241	72	34,691	95
C1.3	8.1 m	CLT240 L7s + HEA160	26,130	50	29,525	74
C1.4	10.8 m	CLT240 L7s + HEA160	29,286	73	32,964	99
C2.1	8.1 m	CLT200 L5s + HEA200	19,401	17	27,614	59

The optimised ratio of the stiffness shares of the two materials for an efficient composite system is questionable. The cross-sections can be categorised according to the ratio of the pure bending stiffnesses of the two individual materials without composite. Using a steel girder with a large stiffness share, the contribution of the CLT to the bearing capacity drops to such an extent that the formation of the composite effect is no longer worthwhile. In addition, maximising steel consumption could be contrary to the goal of a ceiling system that is as resource-efficient as possible. As a contrast, a too solid CLT would mean an excessive, inefficient timber consumption and only a small advantage compared to a pure CLT system. An efficient composite beam requires a balanced contribution of both materials to the composite bending stiffness. Both materials should have a similar normal stress level in relation to their limit strength. The exact value of an optimized ratio of the individual stiffness of CLT to steel depends on load, span and the preferred material and is strongly affected by the stiffness of the shear connectors.

In test series C1.1, C1.2 and C2.1 the ratio of the individual stiffnesses of CLT and steel is 1.1 to 1.2. In test series C1.3 and C1.4 the ratio is 4.5 to 4.6. The composite beams analysed by Hassanieh et al. (Hassanieh et al., 2017b) with a similar construction height of 42 cm had a significantly higher stiffness ratio of the steel beam. A smaller CLT thickness (12 cm) and width (80 cm) resulted in a very low ratio of the individual stiffnesses of CLT to steel of only 0.1. Nevertheless, a comparable bending stiffness developed at a similar overall construction height, which only deviated by around 4 % from the elastic bending stiffness of test series C1.1. At a load level of 40 % of the maximum load, the degree of composite of the system of Hassanieh et al. is 7 % to 14 % higher than in test series C1.1 and C1.3. The stiffness of the CLT, even if it is low, is utilised to a very high degree in Hassanieh et al.. Furthermore, with a smaller CLT cross-section, the overall centre of gravity shifts in the direction of the steel centre of gravity. The lever arm of the CLT centre of gravity to the overall centre of gravity increases and the effect of the CLT in the composite bending stiffness increases slightly. A lower stiffness component of the CLT therefore leads to a more efficient utilisation of the timber. The stiffer the composite joint, the lower the required CLT height for a balanced contribution of the two materials.

However, the higher degree of composite merely means better utilisation of the stiffness potential of the CLT, but not a higher stiffness in absolute values. The composite beam of Hassanieh et al. with a large steel beam only doubles its bending stiffness compared to a pure steel beam. In the test series C1.1, C1.2 and C2.1, the bending stiffness increases by a factor of three to four, in the test series C1.3 and C1.4 even by a factor of seven to eight compared to a pure steel beam. The efficiency of the composite design in relation to the bending stiffness of a steel beam thus increases parallel with an increasing stiffness ratio of the CLT.

Long-term studies of CLT show creep effects. Over time, the steel girder in the composite beam would take on more load than at the beginning (Chiniforush et al., 2021). With a very large steel girder, the benefit of the composite thus decreases further.

The test specimens in test series C1.1 achieve 40 % of the bending stiffness of an identical composite beam but a concrete slab. However, the lower stiffness is largely offset by the significantly lower weight of the CLT-steel composite beams.

Vibration behaviour

To evaluate the vibration behaviour, the test specimens were induced at mid and quarter span to vibrate without load. The results are given in Table 6. The type and position of the excitation as well as the measurement position did not lead to different fundamental frequencies. The test series C1.1 and C2.1 with different shear connectors both achieved a first fundamental frequency of 10.2 Hz. Type and spacing of the shear connectors did not influence the result. In the same span, test series C1.3 with a higher stiffness contribution of the CLT resulted in a similar, but slightly higher fundamental frequency of 10.4 Hz. For the longer spans, the fundamental frequency dropped to 6.1 to 6.2 Hz. Especially in test series C1.4, the vibration behaviour could be increased to over 8 Hz by increasing the degree of composite through a stiffer composite joint, as well as considering the additional stiffness of a screed. Chiniforush modelled composite beams with a total construction height of 47 cm with a 16 cm thick CLT cross-section and a steel girder spacing of roughly 1.25 m in its FE models. A first fundamental frequency of 9.77 Hz for a span of 8 metres was obtained (Chiniforush, 2018). This agrees well with the results of test series C1.1, C1.3 and C2.1. In both cases, the first fundamental frequency was generated by a bending mode shape. Although the height, span and bending stiffness were slightly higher for Chiniforush, the fundamental frequency was slightly lower than in tests C1.1, C1.3 and C2.1. A reason for this could be the damping of the timber, which leads to slight improvements with a more solid CLT cross-section.

Table 6

Fundamental frequency of vibration tests

Test series	Span	Configuration	f [Hz]
C1.1	8.1 m	CLT200 L5s + HEA200	10.2
C1.2	10.8 m	CLT200 L5s + HEA200	6.2
C1.3	8.1 m	CLT240 L7s + HEA160	10.4
C1.4	10.8 m	CLT240 L7s + HEA160	6.1
C2.1	8.1 m	CLT200 L5s + HEA200	10.2

Performance under serviceability load state

To assess the practical application potential of CLT-steel composite beams, the bending tests (Fig. 7) were evaluated for a test load generating the same bending moment, that results of a design line load in the serviceability limit state (SLS) load combination. A girder spacing of 1.5 m results in a load of roughly 10 kN/m as a SLS design line load, considering the dead weight, a floor structure with 3 kN/m² and a live load of 3 kN/m². The line load generates a bending moment. From the test results, the load level that produces the same bending moment as the previously specified line load in the SLS was selected. Table 7 shows the test load, the maximum deflection at midspan and the horizontal slip in the composite joint at the support. This consideration represents an approximate calculation, as the moment gradients due to two point loads or a line load differ for the same maximum moment.

The different cross-sections and shear connectors of test series C1.1, C1.3 and C2.1 clearly satisfy the limits of the instantaneous deflections according to EN 1995-1-1 7.2 (European Committee for Standardization, 2010a). Contrary to the previous considerations, the shear studs as shear connectors of test series C2.1 deform by far the least. The maximum deflection w at midspan

Table 7

Test load F, deflection w and horizontal slip v for serviceability load state

Test series	Span	Configuration	F [kN]	w [mm]	l/300 [mm]	v [mm]
C1.1	8.1 m	CLT200 L5s + HEA200	55	21.1	27	0.75
C1.2	10.8 m	CLT200 L5s + HEA200	67.5	52.0	36	0.81
C1.3	8.1 m	CLT240 L7s + HEA160	55	19.7	27	0.63
C1.4	10.8 m	CLT240 L7s + HEA160	67.5	54.5	36	0.75
C2.1	8.1 m	CLT200 L5s + HEA200	55	19.1	27	0.46

Fig. 7

Large deformation of the CLT-steel composite beam in 4-point bending test is also lowest here. This confirms the application potential of shear studs as shear connectors. The vertical deflections of the test series C1.2 and C1.4 at a span of 10.80 is above a limit value of the instantaneous deflection of L/300 according to EN 1995-1-1 7.2 (European Committee for Standardization, 2010a). Without changing the cross-section, a higher degree of composite or a precambering, the composite beams would only comply with a limit value of L/200 for the instantaneous deflection in a span of 10.8 m. An optimised cross-section configuration that meets the deformation criteria will be developed using numerical models.

EN 1995-1-1 7.3.3 (European Committee for Standardization, 2010a) only states the vibration limits for residential buildings. A distinction is made between a ceiling within a residential unit and between two residential units. The evaluation of the vibration behaviour shows that for all tested configurations in spans of 8.10 m, a fundamental frequency of about 10 Hz meets even the more stringent vibration requirement of floor slabs between different residential units according to Hamm and Richter (Hamm et al., 2009) of 8 Hz. For larger spans of 10.80 m a fundamental frequency of 6 Hz was determined, so that the requirements of 6 Hz according to (Hamm et al., 2009) are hit. As can be seen from Table 5, there is still potential to increase the degree of composite. The use of a larger number of shear connectors would lead to a stiffer composite joint and thus a greater composite bending stiffness. As a result, stricter limits of deflections and vibrations could also be met for these spans. If the screed stiffness is also considered, the increased frequency requirement of 8 Hz according to (Hamm et al., 2009) can also be met with the cross-section configuration from test series C1.4.

Under the line load investigated above, the maximum elastic normal stresses of steel and CLT are not even close to their resistances in the load combination of the ultimate limit state (ULS). Since unprotected steel quickly loses its high strength in case of fire and the ULS verifications are usually not decisive, a simple design approach could be to consider only the CLT for load transfer in case of fire. Using the method of reduced cross-sections according to EN 1995-1-2 4.2.2 (European Committee for Standardization, 2010b), the following results are obtained. For a span of 8.10 m, the CLT elements considered can carry the load alone up to a fire duration of 90 minutes. At a span of 10.8 m, a resistance of R60 can almost be achieved with a CLT240, whereas with a CLT200 a fire protection cladding on the underside or an involvement of the steel girder becomes necessary. This simple and safe approach can therefore already lead to a satisfactory result in many cases, but requires that the CLT rests on the support itself and not just the steel girder.

If the components were used as a roof structure, all serviceability limit values would be fulfilled even with a span of 10.8 m due to the lower loads without any further changes to the cross-section configuration or the composite means.

As already stated, a geometrically equivalent composite girder with a concrete slab would achieve around 2.2 times the bending stiffness of a CLT-steel composite beam. In the application scenario considered, the CLT-steel composite beam is only subjected to a maximum load of around two thirds of that of the concrete composite beam, despite the heavy filling only required on the timber ceiling. The low degree of composite of the CLT-steel composite beam compared to the rigid concrete composite beam can be further increased compared to the tests. As a result, the stiffness advantages of the concrete could be completely cancelled out.

Conclusion

In this article, the bending stiffness and the elastic bearing capacity of CLT-steel composite beams were investigated using 4-point bending tests with spans of 8.10 m and 10.8 m. For this purpose, cross-sections with different stiffness distributions between the materials, 5- and 7-layered CLT and different continuously and discontinuously arranged shear connectors were used. In detail, the following conclusions can be drawn:

- For spans over 8 m, the deflection limits are clearly decisive compared to the stress limits. Therefore, the approach of the exclusively elastic bearing capacity is sufficient. Additional plastic load-bearing capacities are not needed. This ensures ductile, predictable failure for the investigated composite beams since the brittle timber failure only occurs after significant plasticization of the steel.

- The used configuration of fully threaded screws is sufficient to transfer the shear forces in the composite joint. This ensures the full bearing capacity of the composite beam to be utilised up to the steel yield point. An increase in the number of screws would nevertheless optimise material utilisation, so that the currently moderate degrees of composite would be improved.
- The cracks in the mortar of the shear studs produced an early drop in the stiffness of the composite joint. Doubling the number of shear studs would produce a higher degree of composite and a greater bending stiffness compared to composite beams with screws. The drop in stiffness in the composite joint would then only occur after the steel yield point has been reached.
- At the same construction height, the composite beams with a higher stiffness contribution of the CLT has a higher bearing capacity despite the lower improvement to no composite. Particularly in view of the long-term decrease in CLT stiffness, the ratio of the individual stiffness of steel and CLT should be selected with a sufficient contribution of the CLT to the stiffness over the entire service life. The bending stiffness should be at least doubled compared to a pure steel girder. For the shear connectors used here, a ratio of the individual bending stiffnesses of CLT and steel of about 1 seems to approach a more efficient system.
- The first fundamental frequency is not directly influenced by the type, the stiffness or the spacing of the shear connectors. The central, material-specific influencing factor is the composite bending stiffness. There is a favourable damping effect with more solid CLT, but it is rather small.
- Under a realistic load for office and industrial structures, the ultimate and serviceability limit values according to EC5 would be completely fulfilled for a span of 8.10 m. For a span of 10.80 m, the deflection limits according to EC5 are critical but can be solved by small adjustments to the cross-section or the shear connector configuration. A practical application of CLT-steel composite beams in spans of more than 8 metres is proven in terms of short-term performance.

The results presented previously are used to validate numerical models for a parametric study. This allows, for example, the effect of the contributing panel width, as well as the arrangement and stiffness of the shear connectors to be investigated. From these findings, an analytical calculation approach of the elastic limit load capacity and bending stiffness will be validated, which is comprehensively valid for all spans, shear connectors stiffnesses in continuously and discontinuously spacings, as well as CLT cross-sections with any number of layers.

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