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ANALYSIS OF STRESS DISTRIBUTION IN CONTACT AREA OF CROSS LAMINATED TIMBER SHEAR WALL TO FLOOR CONNECTION

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ABSTRACT: This paper investigates the behavior of shear walls in multi-story wooden buildings constructed with Cross-Laminated Timber (CLT) in relation to the stiffness of the floor structure. A full-scale shear wall test and a series of accompanying connection and anchorage tests were conducted to determine the influence of the floor panel's contact area in compression perpendicular to plain on the stress and deformation of the wall panels. It can be seen from the analysis that the loading of the lower wall panel by the upper wall panel is uneven. This is contrary to classic models which consider the uniform loading of the upper edge of the lower panel along its entire length. The stiffness of the floor panel also affects the load on the tension anchors.

KEYWORDS: cross laminated timber, shear wall, anchor, compression perpendicular to plane, stress distribution

1 – INTRODUCTION

Cross-laminated timber (CLT) has become an increasingly popular material for the construction of multi-storey buildings due to its high strength-to-weight ratio, sustainability, and potential for prefabrication. The structural performance of such buildings depends not only on the individual stiffness of walls and floors, but also on the effectiveness of their connections. Among these, wall-to-floor connections play a crucial role in ensuring both vertical and lateral load transfer throughout the structure.

Despite their importance, wall-to-floor joints are often simplified in design and modelling. Many analytical approaches assume a uniform load distribution along the contact interface between panels, neglecting the effects of local compliance, connector flexibility, and anisotropic behaviour of CLT. These simplifications can lead to discrepancies between predicted and actual deformation behaviour, particularly in the case of large wall assemblies or buildings subjected to seismic or wind loading [1,2,3].

To better understand these effects, this study focuses on the interaction between the CLT wall and the floor slab, with particular attention to the role of floor stiffness and connector performance in shaping the overall deformation and stress distribution at the wall-to-floor interface.

In order to assess this interaction, it is important to first understand the deformation mechanisms that occur when a CLT shear wall is loaded. When a shear wall is loaded, compression, shear and bending deformation occur in the plane of the CLT wall panel, along with rotation and horizontal and vertical displacement of the panel as a rigid body due to the elasticity of joints

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and connected structures. Typically, rotation in the plane of the panel is the largest component of shear wall deformation [4]. The floor slab counteracts this rotation, but in doing so, it bends. This bending often occurs laterally relative to the panel.

CLT panels generally exhibit significantly lower stiffness in the secondary direction compared to the primary stress direction. The stiffness of the floor slab influences the stiffness and strength of the wall and floor panel connection [5]. Experimental analyses were used to investigate the floor slab's impact on the performance of the shear walls.

2 – EXPERIMENTAL PROGRAM

The experimental analysis includes a full-scale shear wall test with the connection between the floor structure and the lower and upper wall panels, along with additional tests of the shear and tensile anchor connections.

2.1 TEST METHODS AND PROCEDURES

Full-scale Shear Wall Test

The loading of the test specimen during the experiment followed the procedure specified by the test standard EN 594 [6]. The scheme of the test set-up is shown on Fig. 1.

The anchoring of the specimen's base to the foundation structure was designed to replicate common wall panel connections used in construction practice. This was achieved using a local connection with a tension anchor, which allowed the partial uplift of the bottom surface of the wall panel. During the experiment, the base of the lower wall panel was placed into a steel fixture with a stop at the end, preventing horizontal displacement of the panel base. The bottom corner of the panel base, adjacent to the applied horizontal load, was connected to the foundation structure using two Rothoblaas WHT 340 tension anchors. Each anchor was attached to the panel with 20 LBA 4×40 mm nails and to the foundation with an M12 8.8 threaded rod.

The horizontal and vertical loads were applied using hydraulic actuators with a maximum force of 400 kN. The vertical load was transferred via steel cylindrical bearings and a beam HEB 120. The loading process followed the scheme shown in Fig. 2 (left), where the vertical loading was force-controlled, while the horizontal loading was displacement-controlled. The floor slab was additionally loaded along its longitudinal edges with two steel weights simulating the self-weight of the floor structure.

The test specimen was laterally braced at the top edge of the upper wall panel and at the level of the floor slab to prevent out-of-plane buckling. This bracing was provided by steel profiles equipped with guiding rollers, allowing free horizontal movement. The test specimen during the loading test is shown on Fig. 2 (right). Throughout the test, the applied loads and the displacements of the test specimen were continuously recorded at the measurement points defined in Fig. 1. The measurement of displacements was carried out using a combination of laser triangulation distance sensors, linear variable differential transformer (LVDT) transducers and spring-loaded potentiometric displacement sensors.

Accompanying Tests on Shear Bracket Connections

Three test specimens, denoted SH1 to SH3, were subjected to push-out tests. The objective of these tests was to determine the stiffness and load-bearing capacity of the connection made using shear brackets, which were used for panel connections in the full-scale loading test.

The loading of the specimens during the push-out tests was carried out in accordance with EN 26891 [7] for testing joints in timber structures. The arrangement of the loading test, including the placement of measuring sensors, is shown in the schematic diagram in Fig. 3. The loading was force-controlled and followed the loading diagram shown in Fig. 4 (left). For specimen SH1, the estimated maximum load-bearing capacity F_{est} was determined to be 240 kN prior to testing, and the loading diagram was based on this value. Based on the results obtained from SH1, the value of F_{est} was adjusted to 335 kN for specimens SH2 and SH3. Specimen SH1 was loaded at a rate of 0.8 kN/s, while for the subsequent specimens, the loading rate was increased to 1.1 kN/s.

The test specimen during the loading test is shown in Fig. 4 (right). The specimen was configured to provide rigid full-surface support for both outer panels while allowing vertical displacement of the inner panel under the vertical loading applied by the actuator. For all specimens, the applied load and the corresponding slip between the outer and inner panels were monitored. The slip was measured using potentiometric displacement sensors. For evaluation purposes, the data from all four sensors were averaged

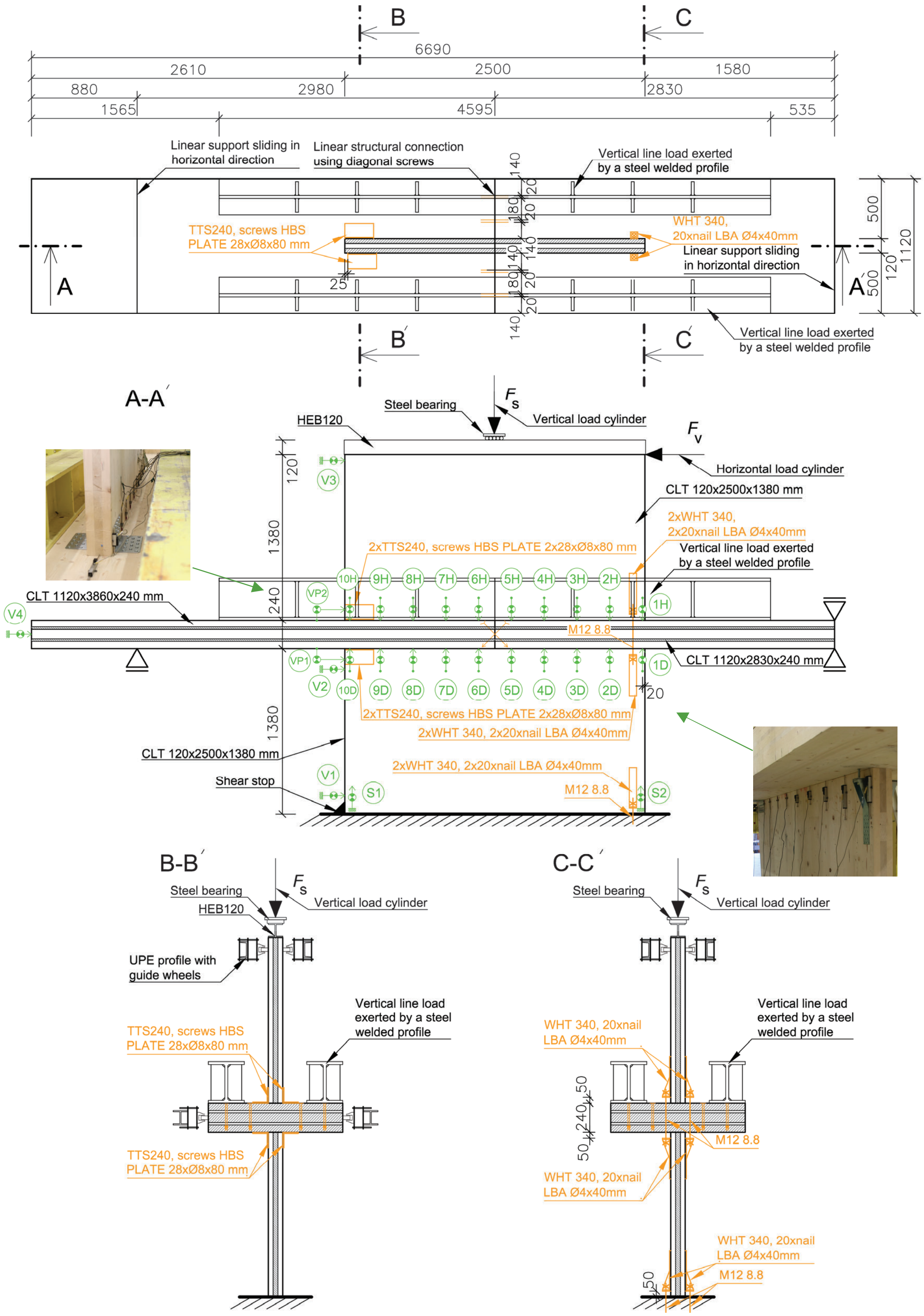

Figure 1. Wall to floor connection: scheme of the test set-up with display of anchoring and connecting elements (orange color) and sensor positions (green color).

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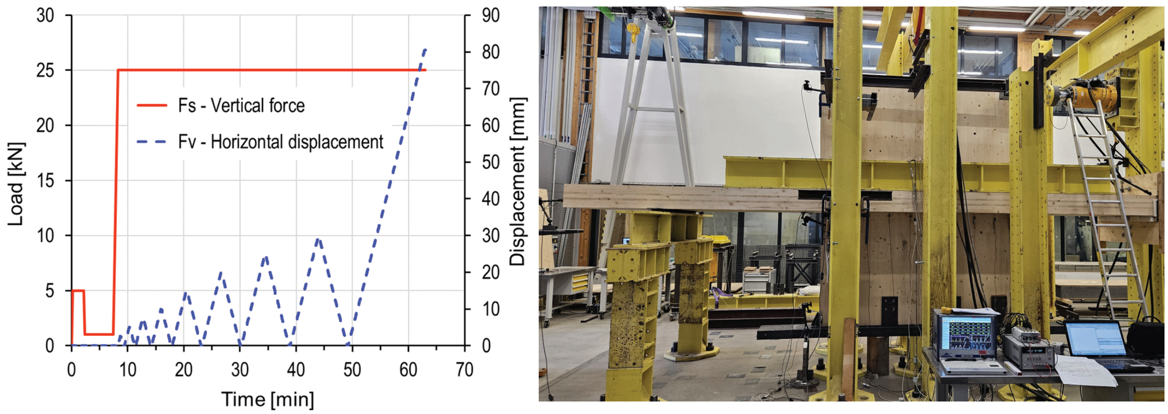

Figure 2. Loading diagram of the shear wall test (left), test specimen under loading during the experiment (right).

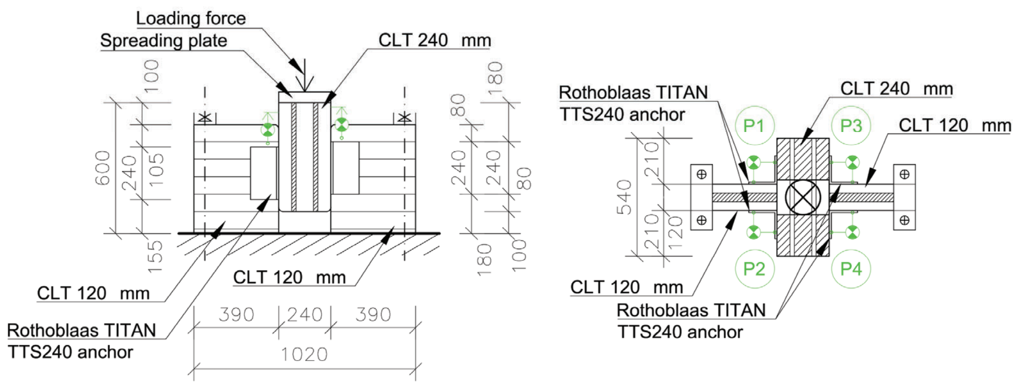

Figure 3. Loading test setup for shear brackets – side view (left) and plan view (right).

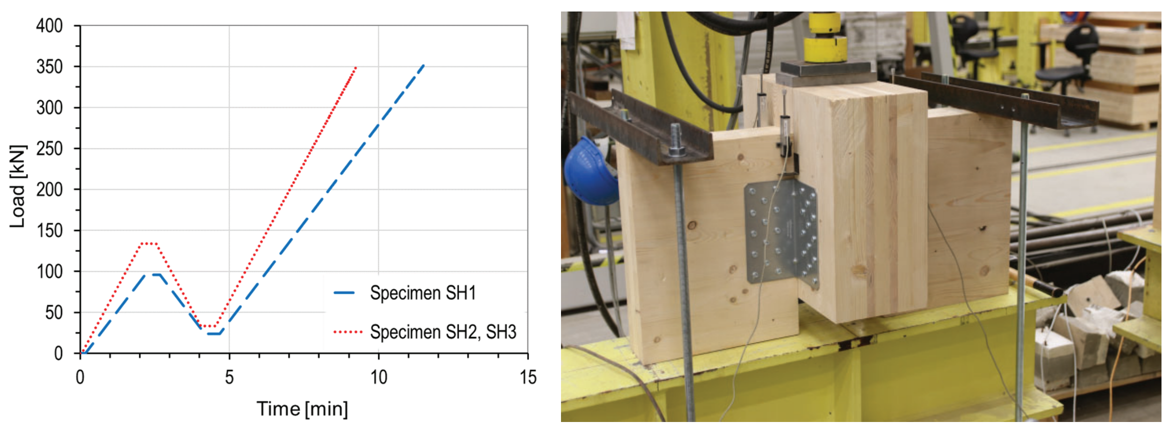

Figure 4. Loading diagram of the shear connection tests (left), test specimen under loading during the experiment (right).

Accompanying Tests on Tension Anchor Connections

Three specimens, denoted TE1 to TE3, were subjected to uniaxial tensile loading to determine the stiffness and load-bearing capacity of the tension anchor connection used in the full-scale wall-to-floor assembly. The tests were carried out in accordance with the methodology defined in EN 26891 [7]. The test configuration, including the positioning of the displacement sensors, is illustrated in Fig. 5.

The loading was force-controlled, following a stepped loading diagram based on an initial estimated capacity (F_{est}) of 95 kN, as shown in Fig. 6 (left). All specimens were loaded at a constant rate of 0.3 kN/s.

Photographs of the specimens under tensile load are shown in Fig. 6 (right). During testing, both the applied tensile force and the resulting displacement at the anchor interface were continuously recorded using potentiometric displacement sensors. Four sensors were

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used per specimen and the measured values were averaged for further analysis.

2.2 DESCRIPTION OF THE TEST SPECIMENS

Full-scale Shear Wall Test

The test specimens consisted of two three-layer CLT wall panels and a seven-layer floor slab positioned between them. Each wall panel measured 2500 mm in length, 1380 mm in height and 120 mm in thickness. The floor slab was composed of two longitudinally joined CLT panels, each with a width of 1120 mm, a thickness of 240 mm and lengths of 2830 mm and 3860 mm, respectively. The outer layers of the wall panels were oriented vertically, while the outer layers of the floor slab were aligned parallel to the longitudinal axis of the specimen. The test specimen had overall dimensions of 1120 × 3000 × 6690 mm (width × height × length). Connection between the wall panels was achieved by two Rothoblaas WHT 340 tension anchors, utilizing M12 8.8 threaded rods. Each anchor was fixed to the wall panels using 20 LBA 4×40 mm nails. Furthermore, each wall panel was connected to the floor slab through two Rothoblaas TITAN TTS 240 steel angle brackets, with each bracket fastened by 28 Rothoblaas HBS PLATE 8×80 mm screws.

Accompanying Tests on Shear Bracket Connections

Each test specimen was composed of a pair of three-layer CLT panels, with a thickness of 120 mm, a length of 390 mm and a height of 500 mm, between which a seven-layer CLT panel measuring 240 mm in thickness, 540 mm in width and 500 mm in height was positioned. The outer layers of the three-layer panels were oriented perpendicular to the direction of the applied load, whereas the outer layers of the seven-layer panel were oriented parallel to the loading direction. Each orthogonal connection between the panels was fitted with two Rothoblaas TITAN TTS 240 steel angle brackets. Each bracket was fastened to the panels using 28 Rothoblaas HBS PLATE 8×80 mm screws. The overall dimensions of the test specimen were 540 × 1020 × 600 mm (width × length × height).

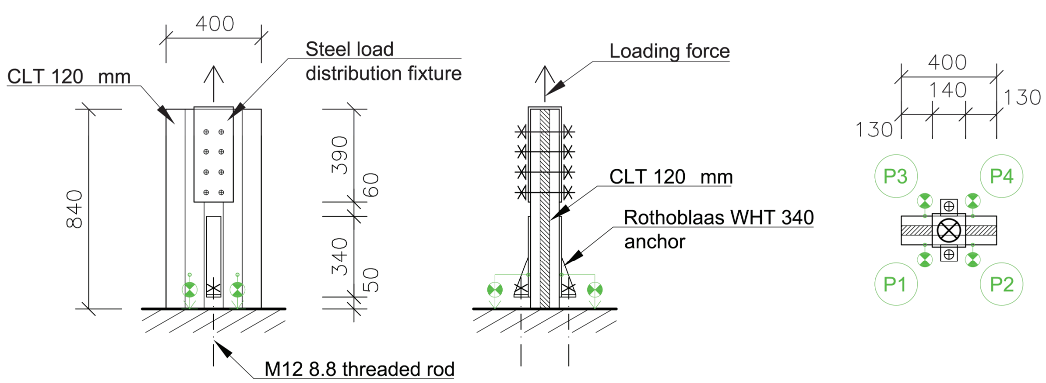
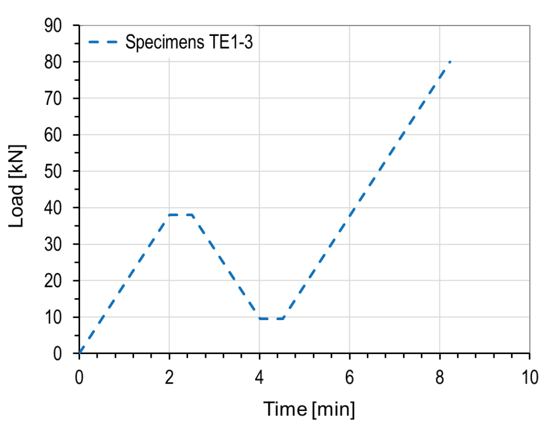


Figure 5. Loading test setup for tensile anchors – side views (left, middle) and plan view (right).



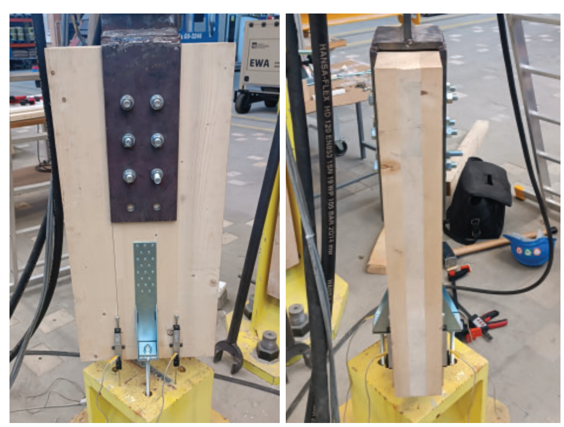


Figure 6. Loading diagram of the tension connection tests (left), test specimen under loading during the experiment (right).

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Accompanying Tests on Tension Anchor Connections

Each specimen consisted of a single three-layer CLT panel with dimensions 400 mm in length, 840 mm in height and 120 mm in thickness. The panel was anchored to a rigid steel base using two Rothoblaas WHT 340 metal tension anchors. The outer layers of the CLT panels were oriented parallel to the direction of the applied tensile force. Each anchor was fastened to the panel with twenty LBA 4×40 mm nails and connected to the base using an M12 8.8 threaded rod.

3 – RESULTS

Full-scale Shear Wall Test

Failure of the test specimen occurred due to deformation of the tension anchor plates and local embedment deformation of the wood around the nails connecting the anchors to the panels. This type of failure was observed to a similar extent at the connection between the lower wall panel and both the foundation and the floor slab, as well as at the connection between the floor slab and the upper wall panel (see Fig. 7). In the shear brackets, horizontal displacement and slight rotation were recorded. The failure mode of both types of anchors was similar to that observed in the accompanying tests. The test was terminated due to the out-of-plane buckling of the horizontal actuator.

Fig. 8 shows the absolute (V1 – V4) and relative (VP1 and VP2) displacements (left) and the vertical displacements at the base of the lower wall panel (right), each plotted against the corresponding horizontal load. The individual curves displayed in the graphs correspond to the positions of the sensors shown in Fig. 1. Fig. 9 shows the vertical displacement profiles (uplift or embedment) at the contact between the wall panels and the floor slab. Measurement points 1 to 10 on the horizontal axis correspond to the positions of sensors 1D–10D and 1H – 10H, respectively

Accompanying Tests on Shear Bracket Connections

The behavior of all three tested specimens during loading, including their failure modes, was similar. During loading, gradual vertical displacement of the inner (floor) panel occurred.

Figure 7. Observed failure of tension anchor and shear brackets in the connections of the full-scale test specimen.

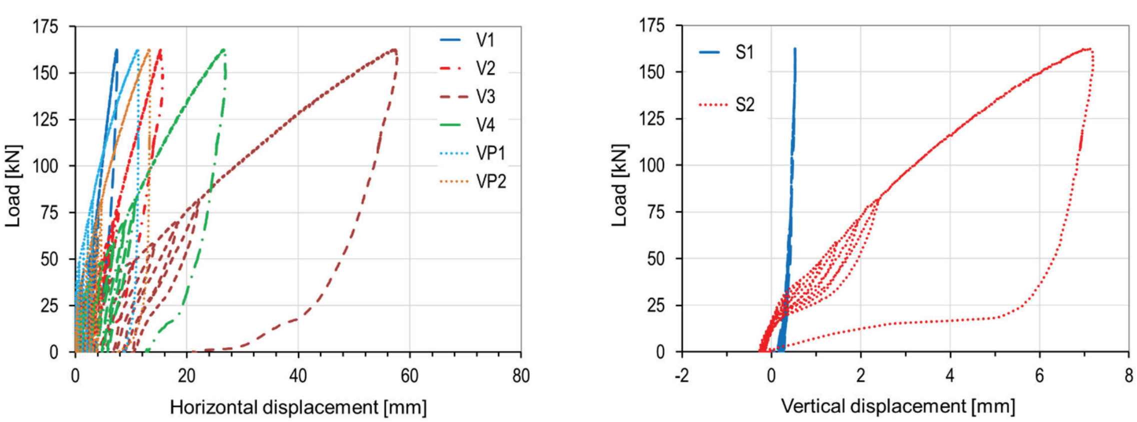

Figure 8. Horizontal displacement vs. horizontal load (left), vertical displacement vs. horizontal load (right).

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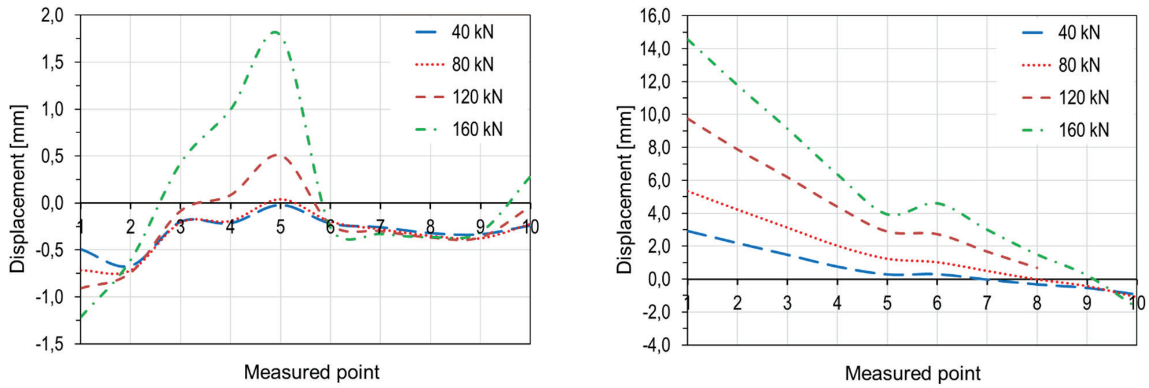

Figure 9. Vertical displacement profile along the lower wall–floor slab joint (left), upper wall–floor slab joint (right).

Deformations (bulging) of the steel bracket plates occurred between the outer row of screws and the joints between the panels. Sharp edges formed on the bracket plates cut into the sides of the outer supported three-layer (wall) panels. In specimens SH2 and SH3, the inner seven-layer panel fractured during the final stage of loading. An example of the observed failure mode of the test specimens is shown in Fig. 10 (left). Fig. 10 (right) presents the panel-to-panel slip vs. load response of the test specimens comprising four brackets.

The maximum load F_{max} was defined as the force acting at the failure of the test specimen. For the determination of stiffness, the measured values corresponding to 10 % and 40 % of the estimated maximum load F_{est} and the associated displacements v01 and v04 were used. The evaluated stiffness values, referred to a single bracket, are presented in Tab. 1.

Accompanying Tests on Tension Anchor Connections

The failure of the Rothoblaas WHT 340 tension anchor connection involved a combination of plastic deformation of the metal component and failure of the timber in the connection zone. The failure mechanism can be classified as block shear, comprising a combination of shear failure along the grain and tensile rupture across the grain, resulting in the detachment of the entire area surrounding the nails, including partial separation of the outer CLT layer. The typical failure pattern is shown in Fig. 11 (left). The steel anchor exhibited plastic deformation and slight rotation of the vertical leg, particularly in the area where the reinforcing stiffeners begin. This behaviour indicates a flexural response of the anchor caused by the combined effect of axial force and eccentricity in the applied load. Fig. 11 (right) presents the displacement at the connection interface vs. load response of the test specimens comprising two tension anchors.

The evaluation of the measured data was performed using the same method as for the shear bracket tests. The stiffness values, normalised to a single tension anchor, are presented in Tab. 2.

4 – CONCLUSION

This study focused on the stress distribution and deformation behaviour in the contact zone between wall panels and the floor slab in multi-storey CLT buildings. A full-scale shear wall test, supplemented by accompanying push-out tests on shear brackets and tension anchors, provided a comprehensive insight into the mechanical response of the wall-to-floor connection.

The experimental results showed that the load from the upper wall panel was not transferred uniformly along the top edge of the lower wall panel, which contrasts with assumptions commonly used in analytical and numerical models of shear walls. The concentration of stresses and local deformations in the compression zone perpendicular to the grain depended significantly on the compliance of the floor slab.

Both types of mechanical connectors – shear brackets and tension anchors – exhibited failure modes corresponding to local embedment and plastic deformation of metal components. In the tension anchor tests, a flexural response due to eccentric tensile loading and block shear failure was observed. In the shear bracket tests, horizontal displacement of the brackets occurred, followed by the upper edge of the bracket cutting into the side of the CLT wall panel.

The quantified stiffness values of the connections, obtained from both the full-scale and accompanying test series, provide an important basis for validating numerical models of wall-to-floor connections.

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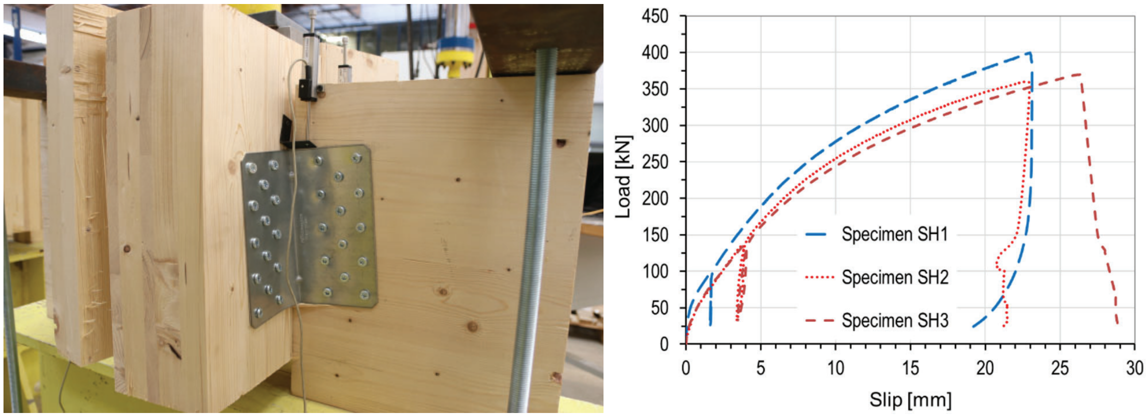

Figure 10. Typical failure of the test specimen with a shear bracket (left), panel-to-panel slip vs. load of the test specimen (entire specimen with four brackets, right).

Table 1: Evaluation of shear bracket stiffness (per single bracket)

Specimen	F_{max} [kN]	$F_{max,avg}$ [kN]	F_{est} [kN]	F_{01} [kN]	v_{01} [mm]	F_{04} [kN]	$v_i = v_{04}$ [mm]	$v_{i;mod}$ [mm]	k_i [kN/mm]	$k_{i,mean}$ [kN/mm]	k_s [kN/mm]	$k_{s,mean}$ [kN/mm]
SH1	99.95		60.00	6.00	0.08	24.00	1.74	2.22	13.76		10.80	
SH2	89.78	**94.04**	83.75	8.38	0.41	33.50	3.91	4.67	8.57	**10.17**	7.17	**8.23**
SH3	92.38		83.75	8.38	0.37	33.50	4.10	4.97	8.18		6.73	

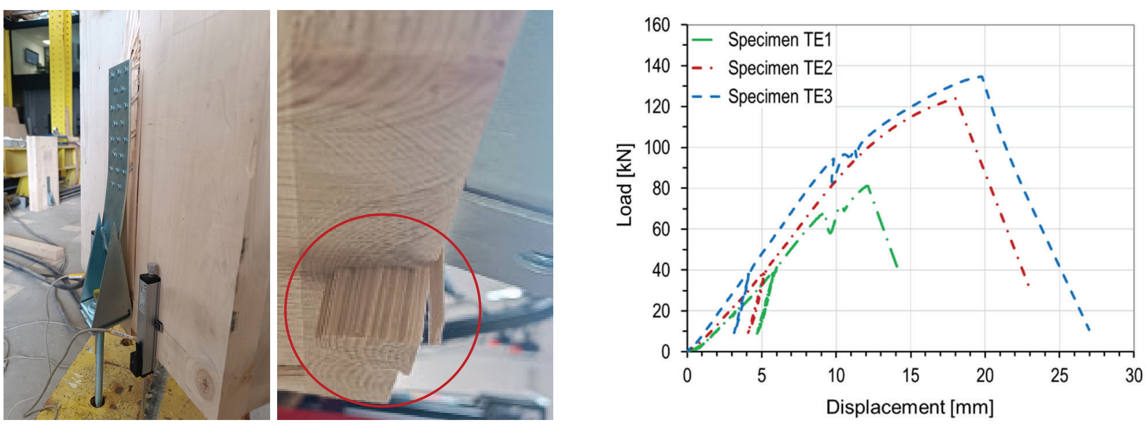

Figure 11. Typical failure of the test specimen with a tension anchor (the circled area in the photo highlights the block shear failure, left and middle), displacement in the connection vs. load of the test specimen (entire specimen with two anchors, right).

Table 2: Evaluation of tension anchor stiffness (per single anchor)

Specimen	F_{max} [kN]	$F_{max,avg}$ [kN]	F_{est} [kN]	F_{01} [kN]	v_{01} [mm]	F_{04} [kN]	$v_i = v_{04}$ [mm]	$v_{i;mod}$ [mm]	k_i [kN/mm]	$k_{i,mean}$ [kN/mm]	k_s [kN/mm]	$k_{s,mean}$ [kN/mm]
TE1	40.58		47.50	4.75	1.85	19.00	5.83	5.30	3.26		3.58	
TE 2	62.01	56.64	47.50	4.75	1.54	19.00	5.21	4.88	3.65	**3.82**	3.89	**4.05**
TE 3	67.33		47.50	4.75	1.14	19.00	4.18	4.05	4.55		4.69	

The findings highlight the need to account for non-uniform load transfer in the design and modelling of such joints. Further research should focus on parametric studies involving different types of connectors, panel configurations, and loading scenarios, aiming to develop more accurate design models that better reflect the real behaviour of connection zones.

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