

INNOVATIVE SLAB-COLUMN CONNECTOR FOR A POINT-SUPPORTED CROSS-LAMINATED TIMBER SLABS IN TALL BUILDINGS – A NUMERICAL ANALYSIS

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Abstract

This study investigates an innovative slab-column connector designed to enhance the structural performance of point-supported cross-laminated timber (CLT) slabs. The proposed connection increases the supported area and pushes the critical circumference further from the column, thereby improving the punching shear capacity. A numerical analysis was conducted to compare the performance of unreinforced and reinforced slabs. The results indicate that the reinforced slab exhibits a punching capacity increase by a factor of 1.63, which shifts the failure mode from rolling shear failure in the unreinforced slab to bending tensile failure in the outer lamellae. The bonded-in steel connector functions as a wider point support and significantly increases the bending stiffness and the critical circumference for the validation of the punching shear. While adhesive failure was mitigated through design modifications, the numerical models were constrained by the assumption of linear elasticity, which limited the representation of the stress redistribution and cracking effects in the CLT.

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Key words

- Multi-storey buildings,
- Point-supported flat slabs,
- Cross-laminated timber,
- Reinforcement,
- System connector,
- Numerical analysis.

1 INTRODUCTION

High-rise structures are very common due to their efficient use of space in urban planning. While they are mostly designed from reinforced concrete or steel, timber has recently been making a breakthrough (ORTA et al., 2020), (KAUFMANN et al., 2018). One of the main issues when building a volume-efficient high-rise is the height of a single floor (ZINGERLE et al., 2016). The best option to achieve the lowest possible construction height of a single floor is a flat slab system. This solution is very common in reinforced concrete construction and is well-founded by over 60 years of research (ŠARVAICOVÁ et al., 2024). However, in timber construction the flat slab system was introduced with the invention of cross-laminated timber (CLT) and other timber-based slab products like

laminated veneer lumber (LVL). CLT is a timber panel product that is used for in-plane and out-of-plane loading; the panel's dimensions are limited to 16 m x 3.5 m due to its production and transportation criteria (BRANDNER et al., 2016); creating a continuous slab ceiling would therefore require a stiff connection between the individual slabs (ZÖLLIG et al., 2016), (MAURER, 2020). In such a construction solution, shear resistance (rolling shear) dominates the flexural resistance, deflection, and vibration of the ceiling structure, as seen in line-supported slabs (KÁZSMÉR, 2024). Shear stresses induced along the cross-sectional planes, which cause the wood fibres to roll over other fibres, are called rolling shear stresses (MINGHAO et al., 2019). Reinforcing methods have been developed to overcome the low rolling shear strength of timber (MESTEK, 2011), (MUSTER, 2020); these methods

utilize self-tapping screws that are drilled diagonally or that increase the effective width of the CLT at the column head. A product for slab–column connections with the possibility of a rolling shear reinforcement, has been developed at the University of Innsbruck. The product has been subjected to extensive testing (ZINGERLE et al., 2016), (MAURER 2020), and has been proven to be an effective slab–column connection. Another possible method of reinforcement has been presented by (KÁZSMÉR, 2023). This method is based on expanding the supported area through a glued-in steel column head in a critical area.

Since the first development of CLT in the early 1990s (Görlacher, 2002), a lot of scientific work has been done on rolling shear strength, the biaxial load bearing behaviour of CLT, possible reinforcement methods, and glued-in steel members in timber. The first experiments on determining the rolling shear modulus and rolling shear strength were done in the early 2000s (Görlacher, 2002), (XAVIER et al., 2009). Since then, the experimental test setup was normalized in EN 408 (KON.005 - Timber structures, 2012). An increase in the strength of rolling shear when accompanied by compression stress was subsequently discovered and confirmed by many authors, e.g., (MESTEK, 2011), (MUSTER, 2020); it permitted an increase in the rolling shear strength by a factor of 1.2 for (MESTEK, 2011) or a factor of 1.6 for (MUSTER 2020).

One of the first full-scale biaxial experiments was done by (CZADERSKI et al., 2007), when he tested multiple samples of 2.5 m x 2.5 m 3-layered CLT panels (thickness 70 mm; 10/50/10 and 25/20/25 layer configuration) under different loading patterns (see Fig. 1). Czaderski compared the different analytical methods for experimental testing under different loading patterns.

(MESTEK, 2011) performed other full-scale biaxial bending experiments on a sample of 1.46 m x 1.1 m 7-layered CLT panels (thickness 189 mm; 7 x 27 mm layer configuration) (see Fig. 2). He used existing analytical methods in his work, including the shear analogy method, to define simplified procedures for an analysis of the point-supported CLT panels. Mestek also successfully used self-tapping screws to reinforce the CLT for rolling shear failure and proved their functionality in experiments and numerical simulations. Based on this research he defined analytical methods for the design of rolling shear reinforcement via self-tapping screws.

Beech plywood (also, beech plywood combined with spruce boards) panels (2.5 m x 2.5 m) were tested at ETH Zurich by Boccadoro (BOCCADORO, 2012); the specimens had 3 different thicknesses, i.e., 240 mm, 320 mm and 400 mm (see Fig. 3). All the specimens showed a great deal of resistance (ranging from 1350 kN to 3138 kN) and clear ductile behaviour.



Fig. 1 Experimental test set-up by Czaderski (CZADERSKI et al., 2007)



Fig. 2 Experimental test set-up by Mestek (MESTEK, 2011)

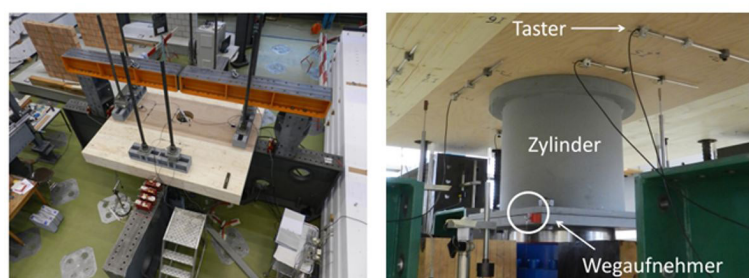
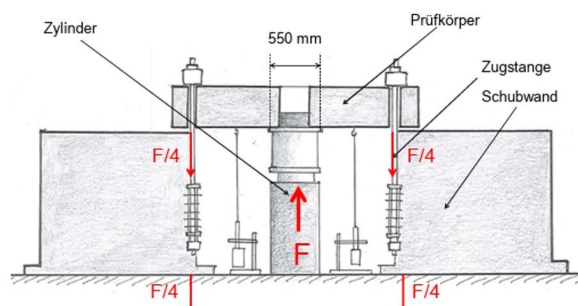


Fig. 3 Experimental test set-up by Boccadoro (BOCCADORO, 2012)



In 2020 (MUSTER, 2020), followed up on Mestek's research in his dissertation. Muster tested 6-layered 2.1 m x 2.1 m CLT panels (total thickness 180 mm 6 x 30 mm layer configuration) (see Fig. 4). He used symmetrical CLT in biaxial bending. His experimental setup was vertically oriented. In his work Muster successfully used methods common to working

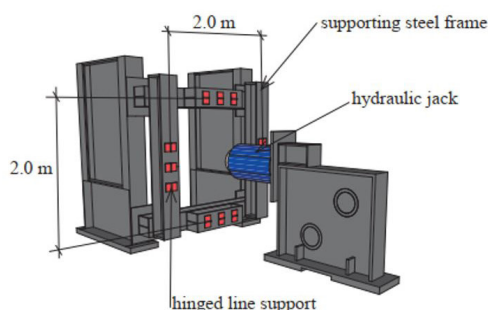


Fig. 4 Experimental test set-up by (MUSTER, 2020)

Around 2016, the development of a systemic solution for slab-column connections began (ZINGERLE et al., 2016) at the University of Innsbruck. The research resulted in the patenting of a "spider connector" (Rotho Blaas Srl, 2020). The development of the product was part of the dissertation of (MAURER, 2020), where he also focused on the advancement of a rigid and robust connection between individual floor

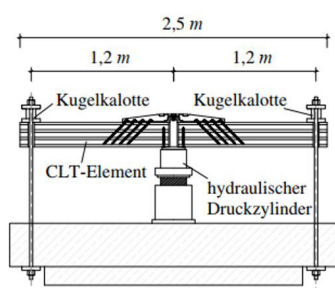
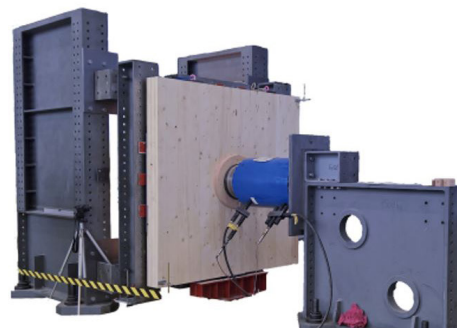


Fig. 5 Experimental test set-up by Maurer (MAURER, 2020)

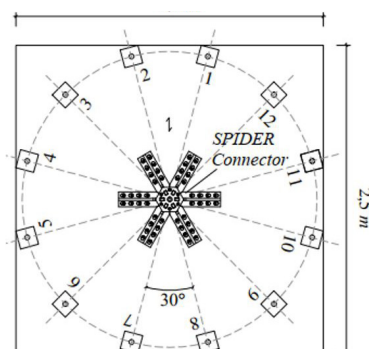
Research on rigid panel connections was undertaken by (ZÖLLIG et al., 2016); its aim was to find a flat slab solution for a column grid of 8 m x 8 m and a live load. Glued butt-joints are the solution for rigid panel connections according to his research. These joints show good resistance in tensile and bending tests; however, they have a brittle failure mechanism, as pointed out by Zhang (ZHANG et al., 2023). (HOSSEINI et al., 2025) tested various types of panel connections in three and four-point bending tests; they have shown that the required rotational stiffness of these connections can be achieved with self-tapping screws to meet the strength and serviceability requirements (HOSSEINI et al., 2025).

Glued-in steel members have also undergone intensive testing, including bonded-in rods (BiR) by (AYANSOLA

with reinforced concrete slabs to determine the shear forces in each column edge. Muster's work also focused on the effect of an opening above a column on the resistance of the slab. In his work he proposed and tested a reinforcement method based on increasing the effective width of a slab.



panels. Maurer tested 5 and 7-layered 2.5 m x 2.5 m CLT panels (total thickness 200 mm 20/40/20/40/20/40/20 and 160 mm 20/40/20/40/20 layer configuration) (see Fig. 5). He proved the functionality of the spider connector and the rigid connection between the panels in many numerical simulations and experiments.



et al., 2022) or (DUCHOŇ, 2016) and many others. Glued-in plates (notched) have mostly been researched in the context of the shear coupling of timber-concrete structures. Extensive state-of-the-art research was done by (ZEMAN et al. 2024). The German technical regulation, "Allgemeine bauaufsichtliche Zulassung/Allgemeine Bauartgenehmigung Nr. Z-9.1-770", which was issued by the Deutsches Institut für Bautechnik (DIBt) (TiComTec GmbH 2020), outlines the technical requirements, manufacturing processes, and assembly provisions necessary to ensure the safe and efficient use of glued-in perforated plates in construction. Additionally, the application of adhesives is defined by manufacturers in their respective technical guidelines, as exemplified by the XEPOX product from Rothoblaas (see Tab. 1).

Tab. 1 Characteristics of epoxy glue by Rothoblaas (Rotho Blaas srl, undated)

Normal adhesion tension	27	MPa
Compression strength	88	MPa
Tensile strength	36	MPa
Unitary shear strength by punch tool	28	MPa
Elastic modulus in tension	4600	MPa
Elastic modulus in compression	3098	MPa

2 SLAB-COLUMN CONNECTION DESIGN

A possible new solution for slab-column connections is being developed at the Slovak University of Technology in Bratislava. This connection increases the supported area and pushes the critical diameter further from the column. This solution aims to increase the bending stiffness of the panel directly above the CLT and also increase the punching capacity to save on-site assembly time (see Fig. 6).

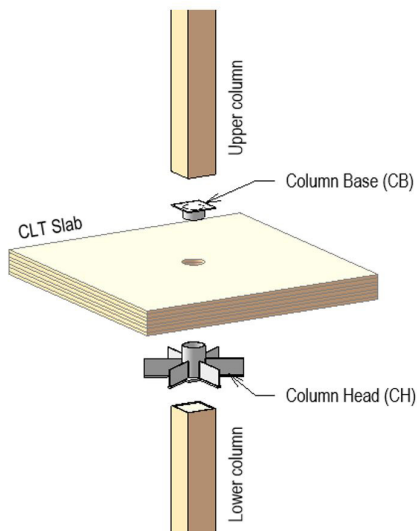


Fig. 6 Connector

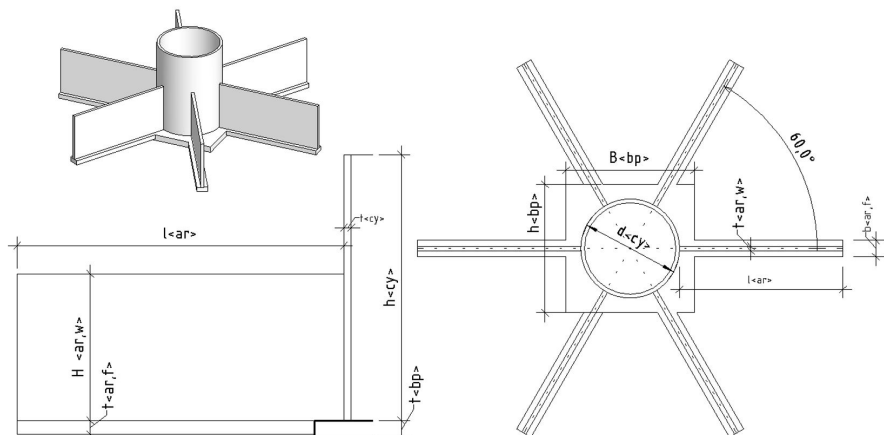


Fig. 7 Slab-column connector

The slab-column connection consists of 2 parts, i.e., the lower column's head (CH) and the upper column's base (CB). The CB is connected to the lower column through the base plate with self-tapping screws. The base plate can be rectangular, circular or any shape that corresponds to the column. The cylinder is welded to the base plate; the cylinder's diameter and web width are designed to resist the required forces from the upper column and the connected slab. The cylinder stands out from the CLT slab to assure a connection with the CB with partially threaded bolts to resist any twisting moments (or potential tensile force) of the upper column. The arms of the CB are arranged in the shape of a star; various number of arms can be used, but always in pairs. The arm's dimensions are designed to resist the design effects and to achieve the required degree of stiffness. The connection is fully parametric, and all the dimensions can be adjusted (see Fig. 7).

All the adhesive surfaces of the column head should be sandblasted in accordance with the adhesive manufacturer's specifications. For example, XEPOX by Rothoblaas requires sandblasting to class SA2.5/SA3, followed by the application of a protective layer of XEPOX P (Rotho Blaas srl, undated). Alternatively, the web can be notched, which eliminates the need for sandblasting and reduces labour intensity. However, this approach compromises the adhesive's resistance.

Design improvements have been implemented to minimize stress on the adhesive, to mitigate any brittle failure modes, and enhance ductile behaviour. The height of the connector's arms has been reduced by the thickness of the first two lamellae of the CLT, thus ensuring that the tensile stresses induced by the bending moment over the column are transferred through the timber rather than the glue-steel-timber interface. Additionally, a bottom flange has been added to the connector's arms to maintain residual resistance in the event of any failure of the glue-steel-timber interface. This flange also enhances the arm's bending stiffness and reinforces the overall structural performance.

Fire resistance in the structures can be enhanced by offsetting visible steel components by a desired dimension and covering them with timber. This approach prevents direct exposure of the steel to the fire, thereby significantly reducing its temperature and improving the overall performance of the fire resistance

3 NUMERICAL INVESTIGATION

3.1 Overview

Numerical modelling was performed to find the experimental set up that best utilized the options of the laboratory and described the behaviour of a real slab adequately, estimated the connector’s capacity, and compared an unreinforced with a reinforced sample.

The laboratory of interest is the Central Laboratories of the Slovak University of Technology in Bratislava. Here, punching test of concrete slabs were performed for over 10 years; the set-ups are well described in (ŠARVAICOVÁ et al., 2024) and (BOLEŠOVÁ and GAJDOŠOVÁ, 2023). The floor has anchorage points at regular rasters of ≈ 750 mm. Therefore, most common specimen dimensions are 2.5 m x 2.5 m. Such dimensions correspond to a flat slab supported by columns in a regular raster of approximately 5.50 m ($L = 5.50 \text{ m} \cong 1.185 \text{ m} / 0.22$) (BOLEŠOVÁ and GAJDOŠOVÁ, 2023).

Tab. 2 CLT 240 properties

CLT Thickness	Layers	$I_{x,net}$	$I_{y,net}$	$I_{x,net} / I_{y,net}$
[mm]	[mm]	[cm ⁴]	[cm ⁴]	[-]
240	30/40/30/40/30/40/30	74400	40800	1.82

3.2 Development of a numerical model

A numerical model was developed using the FEM software SOFiSTiK. A quadrangle type of element was used. Meshing was automatically generated with a mesh density set to 0.3 m with refinements at the short edges. The material’s nonlinearity and the second order theory (P-D-delta) were considered.

Specimens of this size can be manipulated well by a lab crane and the transport from a workshop to a lab. Although a panel dimension of 2.4 m would be optimal for truck transport, this configuration would result in an insufficient edge distance of approximately 75 mm for a hole diameter of 70 mm.

The desired CLT thickness is 240 mm in a 7-layered configuration (30/40/30/40/30/40/30); in this configuration the ration of the bending stiffness in the x and y directions is equal to 1.82. For two-way bearing slabs with a regular column grid, the ratio ≈ 1.0 is ideal (see Tab. 2) if the slab is continuous or, in the case of CLT, connected with rigid connections. A 6-layered symmetric CLT slab was tested by (MUSTER 2020); however, a 7-layered setup was chosen as it is a common configuration on the market, and most of the previous works have dealt with asymmetrical panels (MAURER, 2020), (MESTEK, 2011), (CZADERSKI et al., 2007).

The CLT is modelled as a layered 2D shell element, with characteristics corresponding to a CLT 240 panel with shorter glued edges, Tab 2. The material of the individual lamellae is C 24 with standardized stiffness properties and is linearly elastic. The CLT’s orthotropic behaviour was considered with the use of the layered material option in SOFiSTiK; this material definition allows assigning different material properties and thicknesses to distinct layers. See the stiffness matrix (Fig. 8).

No	type	mxx	myy	mxy	vx	vy	nxx	nyy	nxy
		[kNm]	[kNm]	[kNm]	[kN/m]	[kN/m]	[kN/m]	[kN/m]	[kN/m]
1	mxx	8184.00	0.00	0.00	-	-	0.01	0.00	0.00
2	myy		4488.00	0.00	-	-	0.00	-0.01	0.00
3	mxy			794.88	-	-	0.00	0.00	0.00
4	vx				174555.56	0.00			
5	vy					126196.37			
6	nxx						1320000.2	0.00	-0.01
7	nyy							1319999.9	-0.05
8	nxy								165600.01

Fig. 8 Stiffness matrix of the defined materia

The connector is modelled as a 2D shell element, with characteristics according to Tab. 3. The steel is nonlinear material with a tri-linear working diagram. For the dimensions of the connector, see (Tab. 3). The connection between the steel and CLT was fully rigid. A small notch was created in the CLT at the top of the arm to prevent the premature yielding of a single element of the steel arm, see (Fig. 9).

To determine the optimal test setup, a linear analysis was performed under various boundary conditions. The initial analysis considered an infinitely large floor model in order

to eliminate the edge effects of the slab and to represent the behaviour of an interior column under symmetric transverse loading. The CLT floor was modelled as a continuous slab over the entire floor plan, which was supported by a column grid of 5.1 m x 5.1 m. This modelling approach represents a floor system composed of individual CLT panels rigidly connected by edge gluing that acts as a continuous slab. No additional criteria were evaluated for the slab; the primary focus was its deformational behaviour over the column, specifically on a rectangular 2.5 m x 2.5 m cut-out. Subsequently, multiple models were analysed with the boundary conditions

representing potential test setups. The following configurations were examined: a) a 9-point test setup, b) a 13-point test setup, c) a rectangular rigid frame test setup and d) an octagonal rigid

frame test setup (see Fig. 10). Each model was subjected to the support reactions derived from the infinitely large slab analysis.

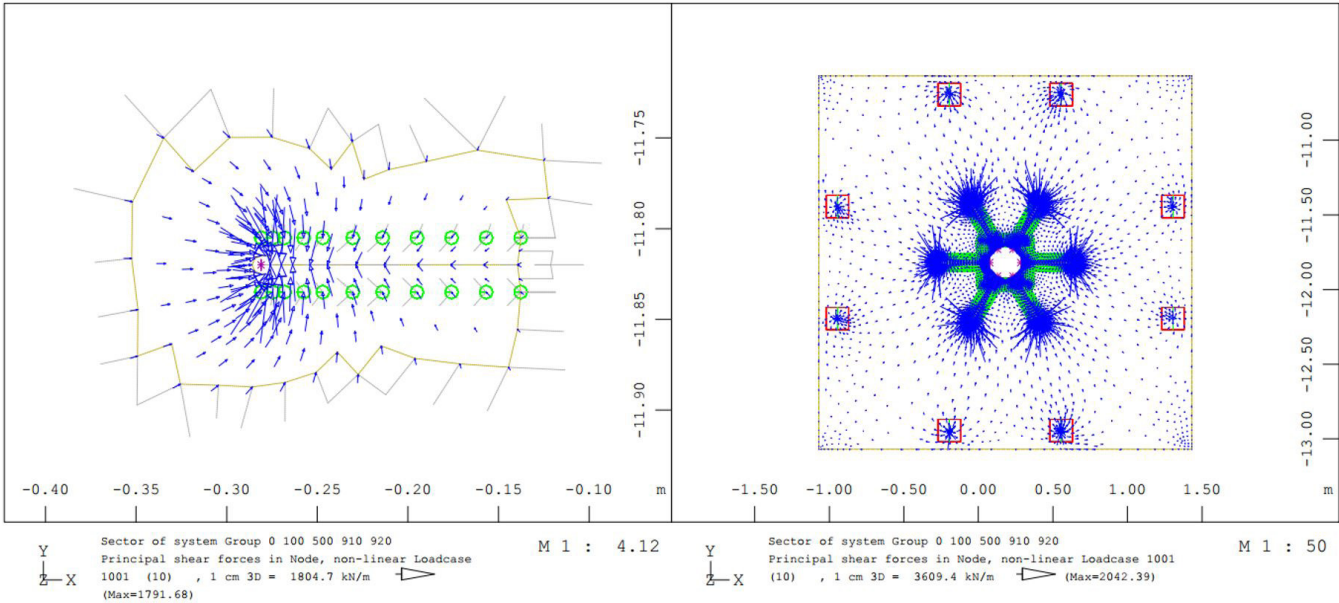


Fig. 9 Shear flow

Tab. 3 Column head dimensions

Base plate			
Breadth	b_{bp}	300	mm
Height	h_{bp}	300	mm
Thickness	t_{bp}	15	mm
Cylinder			
Diameter	d_{cy}	218	mm
Thickness	t_{cy}	10	mm
Height	h_{cy}	300	mm
Arm			
Total length	l_{ar}	352	mm
Height of arm's web	$H_{ar,w}$	155	mm
Thickness of arm's web	$t_{ar,w}$	10	mm
Breadth of arm's flang	$b_{ar,f}$	60	mm
Thickness of arm's flang	$t_{ar,f}$	15	mm

After the establishment of the appropriate test setup, the two models were examined; one was unreinforced with an opening for the column connection, and the other was reinforced with a steel column head. The resistance of the unreinforced slab was determined based on equation (1); it was chosen as suitable based on a publication comparing various calculation approaches for point-supported slabs (KÁZSMÉR, 2024)

$$\tau_{iz} = \frac{V_{iz} S_{iy}}{I_{iy} b_{(z)}} \tag{1}$$

where τ_{iz} is the shear stress in kPa; V_{iz} is the shear force in kN; S_{iy} is the static moment in m³; I_{iy} is the moment of inertia in m⁴; and $b_{(z)}$ is the effective width in m.

The unreinforced model was then loaded by the force F1, which is defined as the load level at which the rolling shear stress in the panel reaches the rolling shear strength $f_{vr,mean} = 1.6$ MPa, increased by a factor of $k_{r,pu} = 1.2$, as per Muster (MUSTER, 2020).

The reinforced slab was subsequently loaded with the same force F1 to compare the samples. Next, a nonlinear analysis

was performed to find the capacity of the connector. Ten steps were calculated with increasing force in the hydraulic press by 50 kN (initial force F1).

A rolling shear failure is expected at the major axis of the slab at force $F1 = 555.72 \text{ kN}$, which assumes that the reinforcing screws perpendicular to the grain are present at the loaded area.

4 RESULTS

4.1 Configuration of an appropriate test setup

Below is a comparison of the displacements of a supported area of the continuous floor structure (Fig. 10. a) to the possible set-ups used in the punching tests (Fig. 10 b) – e)).

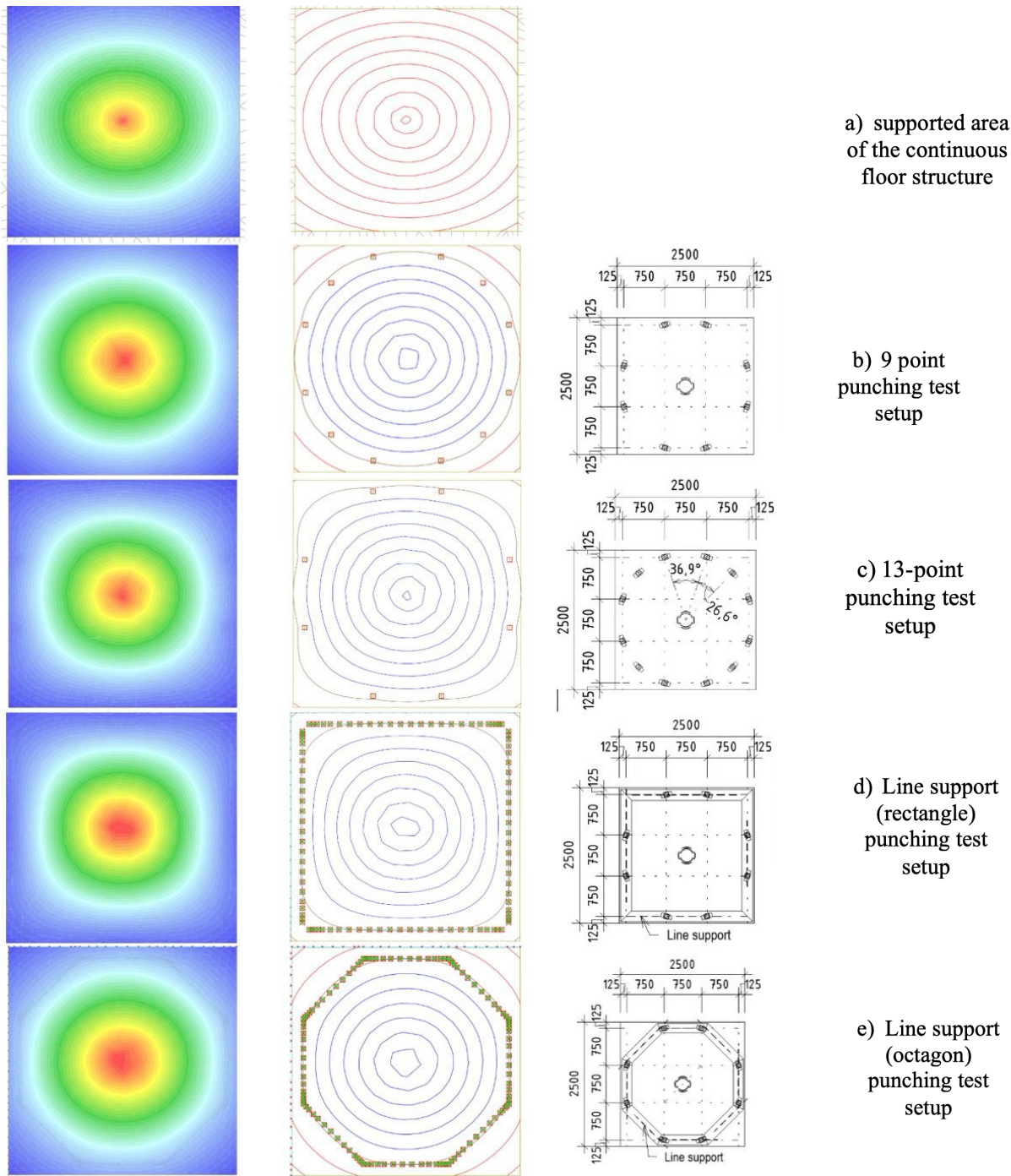


Fig. 10 Iso-areas and iso-lines of displacement of the specimen and their schematic set-ups

4.2 Results from the unreinforced slab

The following results are all from the analysis of the set-up according to b) the 9-point punching test (Fig. 10). The

load acting on the sample in its centre is $F1 = 555.7$ kN. The displacement of the sample at the loaded area reached 11.242 mm. The tensile stresses on the outer lamellae at the edge of the opening reached 72.81 MPa (see Fig. 11)

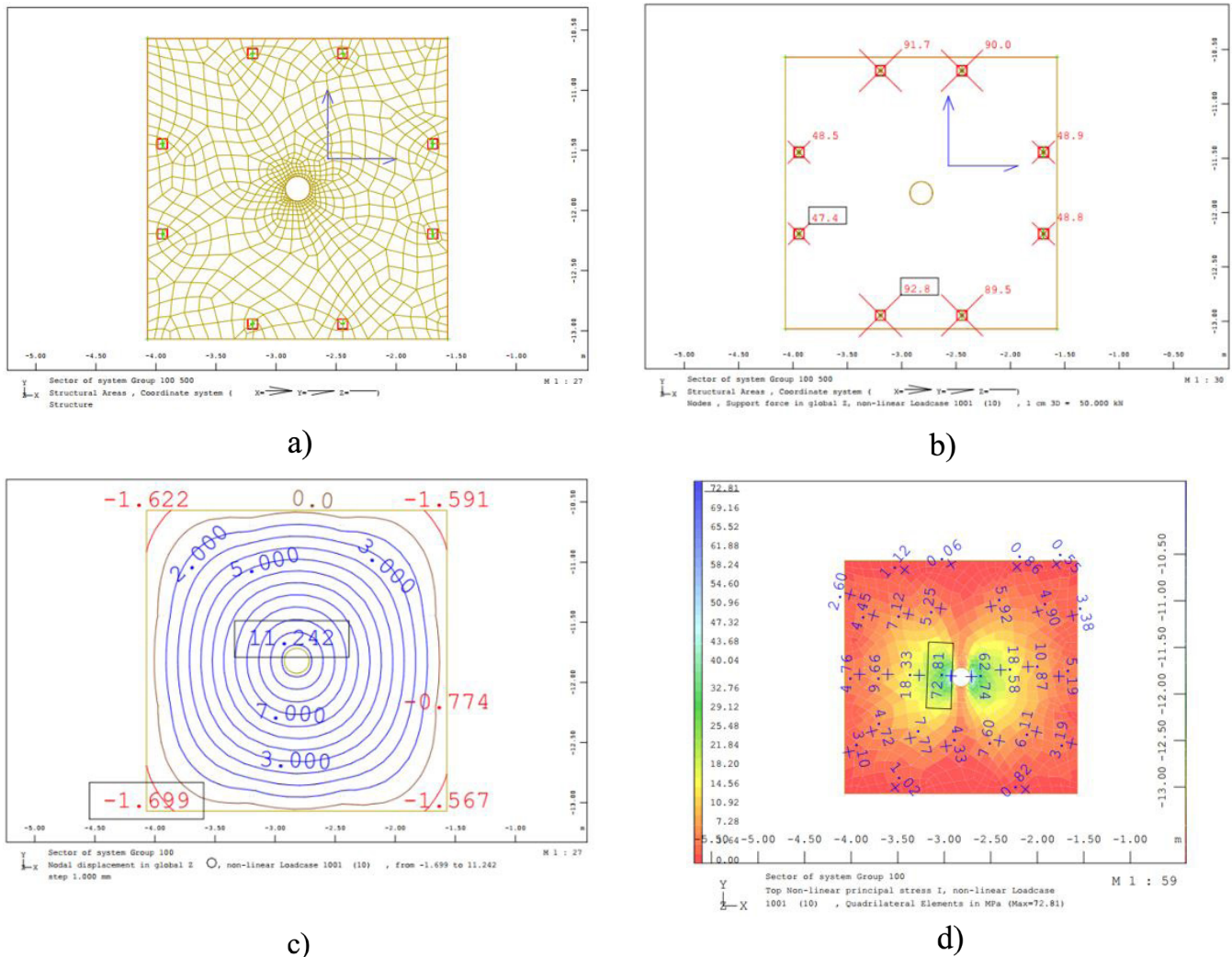


Fig. 11 Results for the unreinforced slab at force $F1 = 555.7$ kN a) FEM model b) Support force in global Z direction in kN c) Nodal displacement in global Z direction in mm d) Top non-linear principal stress I in MPa

The rolling shear stress in the major slab axis reaches its expected mean strength of 1.92 MPa ($f_{v,mean} \times k_{r,pu}$) (see Fig. 12). The panel width effective for punching in each column face is 468.05 mm (loaded area (300 mm x 300 mm) at the

position of the hydraulic press). The forces at each edge of the column are $V_x = 159.19$ kN and $V_y = 118.67$ kN. The analytical solution is according to the literature (KÁZSMÉR, 2024).

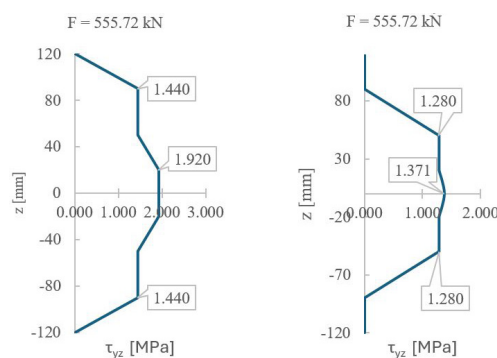


Fig. 12 Shear stresses in minor and major slab axes at $F1 = 555.7$ kN

4.3 Results from the reinforced slab

The reinforced sample was tested under the same boundary conditions. The first results at load F1 are presented to

compare the two specimens. The displacement of the sample at the loaded area reached 8.220 mm. The tensile stresses on the outer lamellae at the edge of the opening reached 47.36 MPa (see Fig. 13).

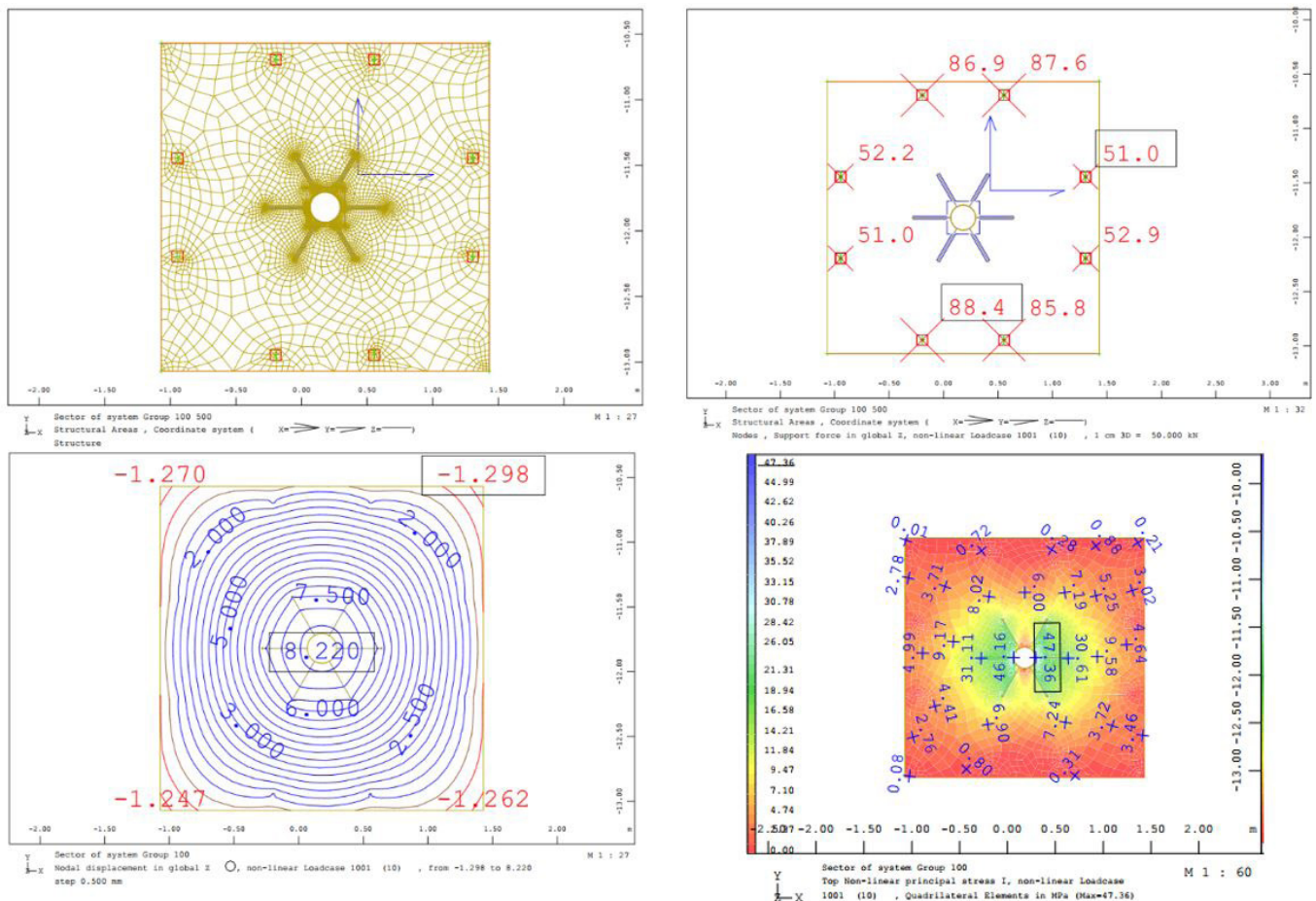


Fig. 13 Results from the reinforced slab at force $F1 = 555.7$ kN a) FEM model b) Support force in the global Z direction in kN c) Nodal displacement in the global Z direction in mm d) Top non-linear principal stress I in MPa

The same analytical approach to determine the shear stresses and flow could not be used for the reinforced slab, due to the bonded-in steel plates that significantly increased the stiffness of the slab and pushed the critical circumference

further from the support. The shear forces on the expected critical circumference are displayed below (see Figs. 14 and 15).

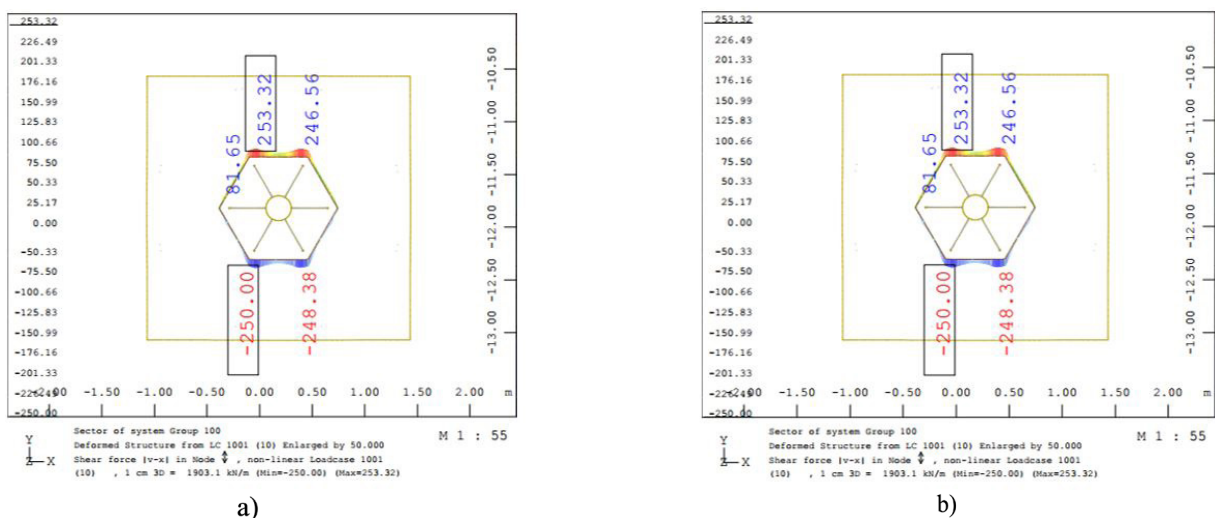
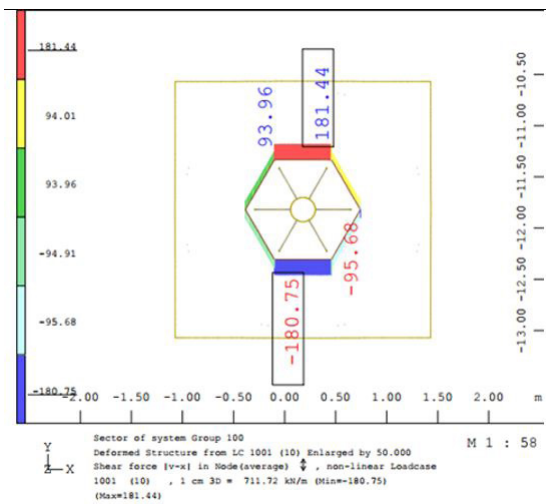
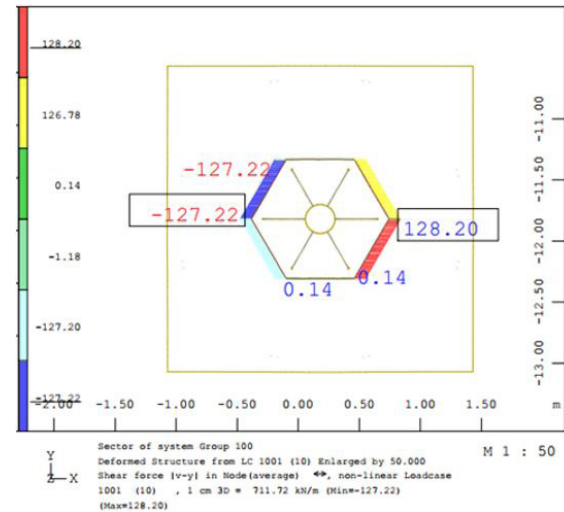


Fig. 14 Shear force in corresponding slab directions at the expected critical circumference at $F1 = 555.7$ kN a) Shear force $|v-x|$ in kN/m b) Shear force $|v-y|$ in kN/m



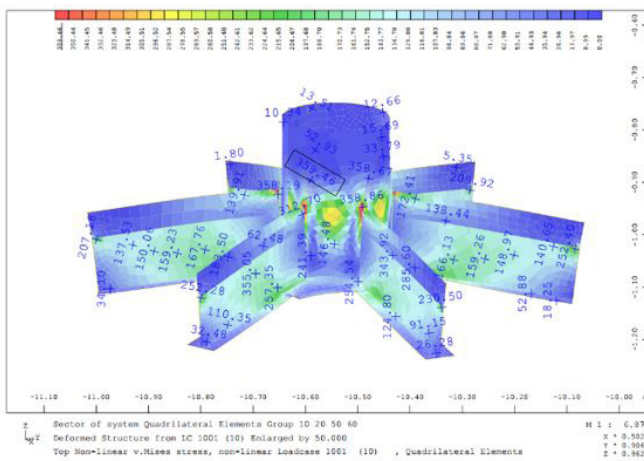
a)



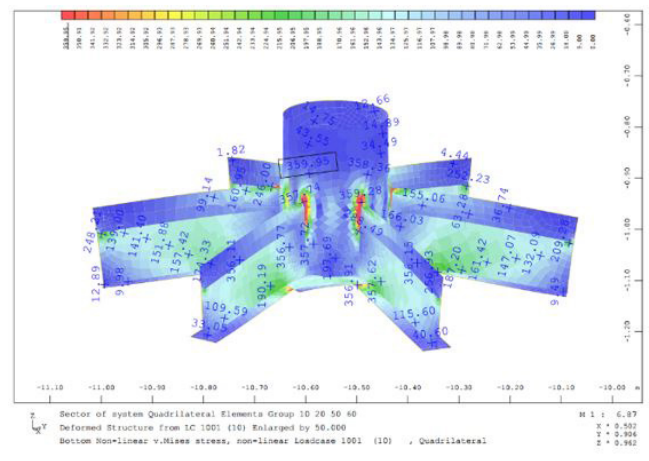
b)

Fig. 15 Shear force in corresponding slab directions (average value on line) at the expected critical circumference at $F1 = 555.7$ kN a) Shear force |v-x| in kN/m b) Shear force |v-y| in kN/m

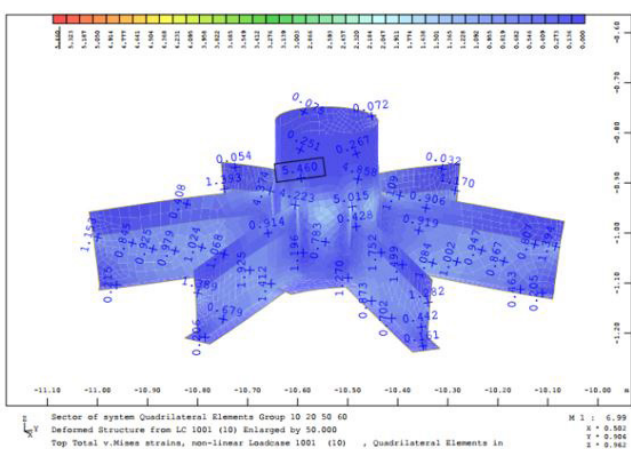
The von Mises stresses and strains were inspected on CH at load F1 (Fig. 16).



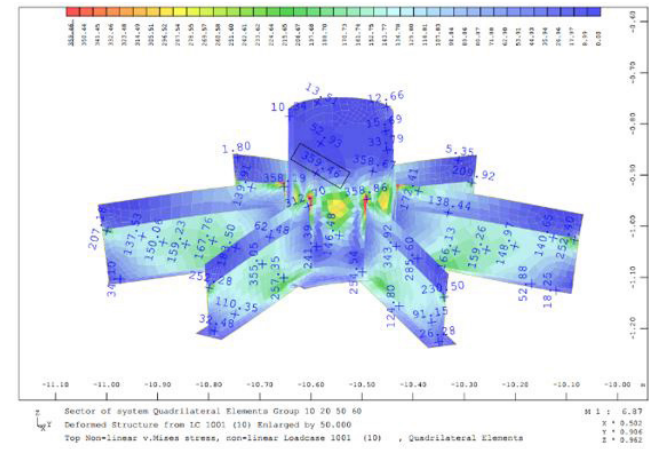
a)



b)



c)

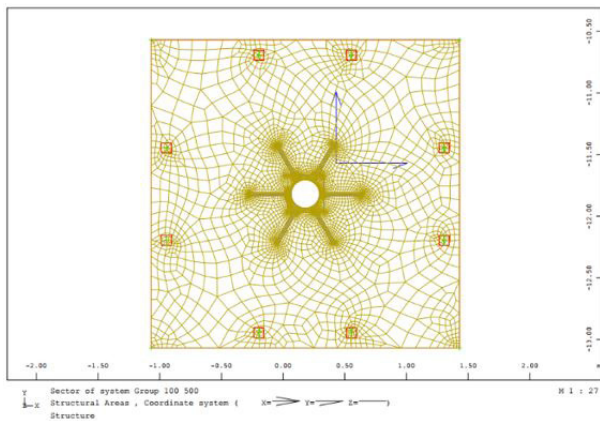


d)

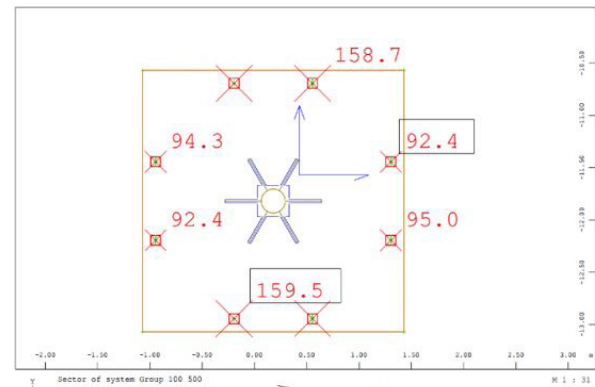
Fig. 16 Von Mises stresses and strains at upper and lower surfaces of CH at load $F1 = 555.7$ kN a) Top non-linear von Mises stresses b) Bottom non-linear von Mises stresses c) Top total non-linear von Mises strains d) Bottom total non-linear von Mises strains

The results from the last step are presented next with the load $F_{10} = 1055.7 \text{ kN}$ ($F_1 + 9 \times 50 \text{ kN}$). The displacement of the sample at the loaded area reached 15.027 mm . The

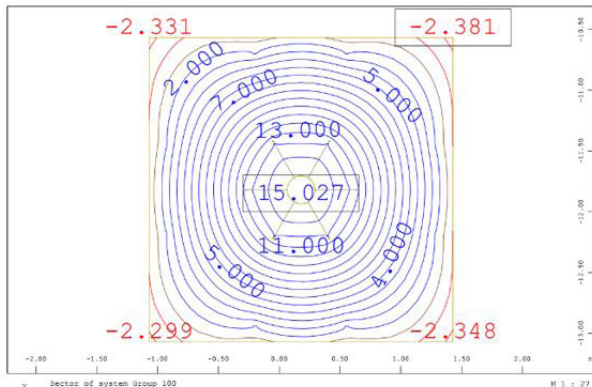
tensile stresses on the outer lamellae at the edge of the opening reached 85.88 MPa , see Fig. 17.



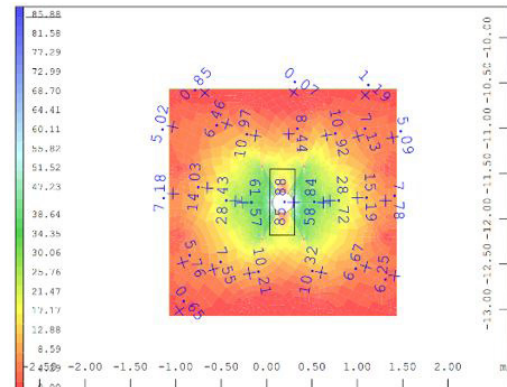
a)



b)



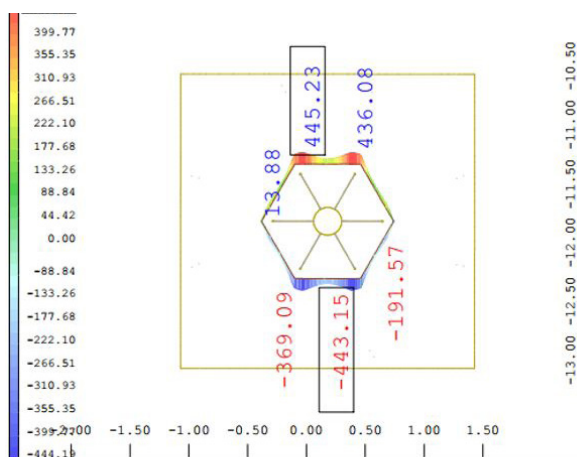
c)



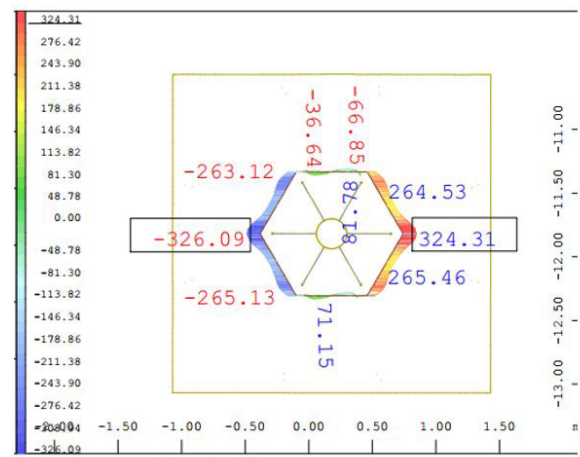
d)

Fig. 17 The results from the reinforced slab at force $F_{10} = 1055.7 \text{ kN}$ a) FEM model b) Support force in the global Z direction in kN c) Nodal displacement in global Z direction in mm d) Top non-linear principal stress I in MPa

The shear forces on the expected critical circumference are displayed below, Figs. 18 and 19.



a)



b)

Fig. 18 Shear force in corresponding slab directions on the expected critical circumference at $F_{10} = 1055.7 \text{ kN}$ a) Shear force $|v-x|$ in kN/m b) Shear force $|v-y|$ in kN/m

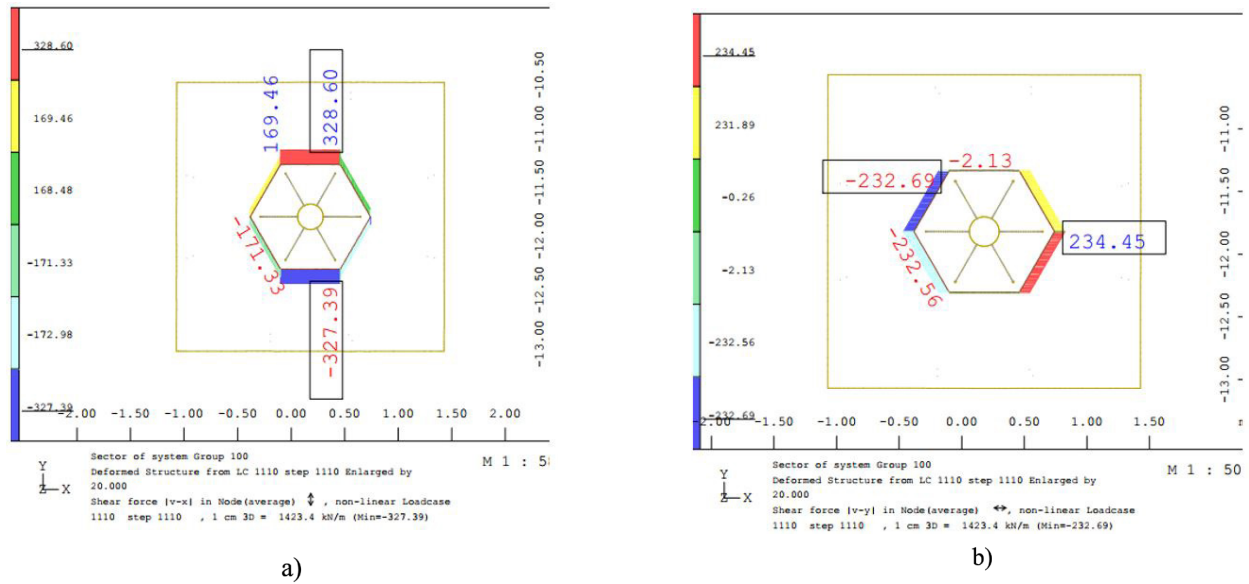


Fig. 19 Shear force in corresponding slab directions (average value on line) on expected critical circumference at $F_{10} = 1055,7$ kN a) Shear force $|v-x|$ in kN/m b) Shear force $|v-y|$ in kN/m

The von Mises stresses and strains were inspected on CH at load F_{10} (see Fig. 20).

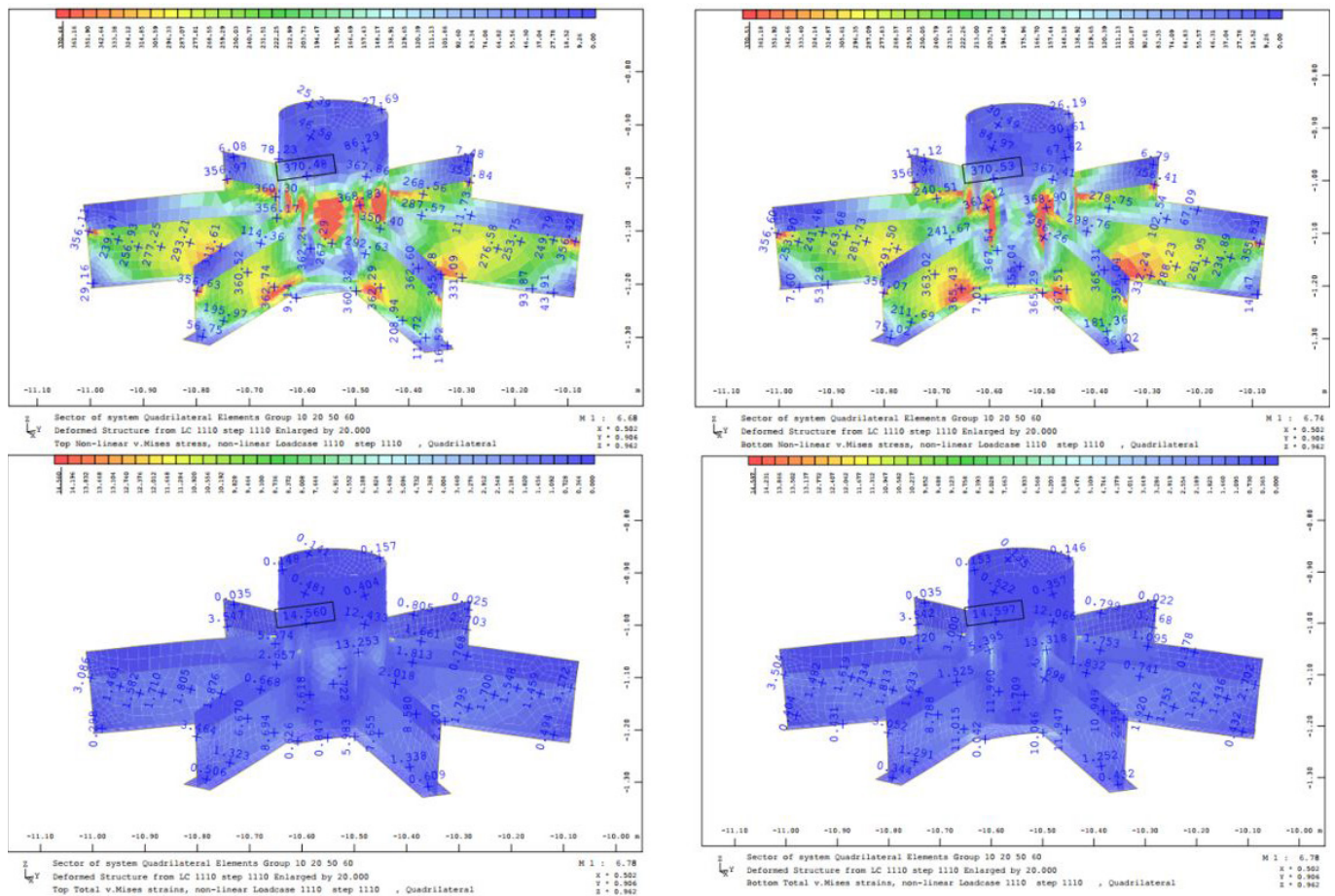


Fig. 20 Von Mises stresses and strains at upper and lower surfaces of CH at load $F_{10} = 1055,7$ kN a) Top non-linear von Mises stresses b) Bottom non-linear von Mises stresses c) Top total non-linear von Mises strains d) Bottom total non-linear von Mises strains

5 DISCUSSION

Fig. 10 presents a comparison between the different set-ups and the deformational behaviour of a real slab above the supported area. All the set-ups exhibit satisfactory performances under the loads; however, set-up e) displays circular iso-lines instead of the desired elliptical shape. For the next steps, set-up b) was selected as the most suitable due to its comparable deformational behaviour. Additionally, this configuration was optimal, considering the available tools and capabilities of the laboratory of interest.

First, a slab without any reinforcement was analysed. The slab was subjected to a force of $F_1 = 555.7$ kN, which corresponds to the bearing capacity of the unreinforced slab. Under this load, the rolling shear stresses reached their mean strength values, which led to failure along the major slab axis. Since the CLT material was modelled as linear-elastic, its residual strength could not be observed. However, previous experimental testing (MAURER, 2020), (MESTEK, 2011), (MUSTER, 2020), suggested its presence. High tensile stresses due to bending were observed on the cover lamellae at the circular opening's edge; they reached 72.8 MPa, which is approximately the expected mean bending strength of timber (≈ 70 MPa). Small cracks were anticipated at the edge of the opening, although they were not expected to reduce the specimen's capacity. These stresses were highly localised on either side of the opening. Similar cracks were also examined by Muster (MUSTER, 2020). In this specific test set-up and panel configuration, the bending stresses were always higher in the outer lamellae, which made failure due to bending in the minor axis unlikely. F_1 is considered the mean force at which failure occurs in the hydraulic press.

Next, a reinforced slab was modelled using the same set-up and panel configuration. The bonded-in steel member behaved more like a wider point support similar to Muster's column capital tests (MUSTER, 2020), rather than as a rolling shear reinforcement, as seen in Mestek's self-tapping screw tests (MESTEK, 2011). This behaviour was expected. Additionally, the bonded-in CH increased the bending stiffness of the supported area, as illustrated in Fig. 21.

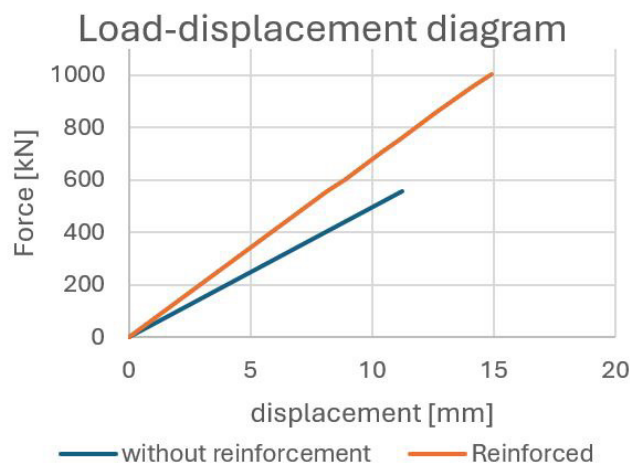


Fig. 21 Load-displacement diagram of two specimens

At load F_1 , a reduction in tensile stress was observed due to the coupling effect with the bonded-in steel member, which lowered the stress level to approximately 47 MPa at the opening's edge. Moreover, the shear forces at the new critical circumference were low; however, Fig. 9 shows a concentration of the shear flow at the tips of the arms. Here, failure was expected at a shear force of ≈ 360 kN/m along the major slab axis, i.e., over a length of 0.190 m, which would induce rolling shear stresses of approximately 1.9 MPa (see Fig. 22). The critical circumference was expected to be offset from the polygon connecting the tips of the arms.

The reinforced slab was then analysed across multiple loading steps, at increments of 50 kN, starting from $F_1 = 555.7$ kN. A total of 10 steps were calculated, which reached a final load of $F_{10} = 1005.7$ kN. At F_{10} , the tensile stresses in the outer lamellae reached 88.58 MPa; they were no longer concentrated at the opening's edge. This indicated that this step exceeded the bending failure threshold of the specimen. An acceptable level of tensile stress was observed at $F_7 = 905.7$ kN, where the stresses reached values comparable to those at which cracks were expected in the unreinforced slab (72.59 MPa). Further increasing the force by 50 kN led to an 8 MPa rise in the stress, with an expansion of the affected area. As a result, failure due to bending was expected between F_7 and F_8 . The peak shear force reached 380 kN/m, which is still within an acceptable range, and no rolling shear failure was expected at $F_7 = 905.7$ kN.

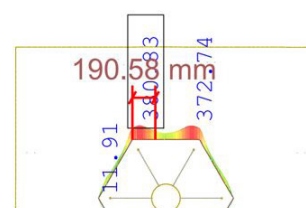


Fig. 22 Close-up of the critical circumference, Load $F_7 = 905.7$ kN

Finally, the capacity of the connector's CH was assessed. The von Mises stresses and strains on both plate surfaces were evaluated in the previous section at different loading steps. At F_1 , no significant strains were observed on the column head. At $F_{10} = 1005.7$ kN, the CH exhibited plastic behaviour in critical areas; however, its potential capacity had not yet been fully reached.

6 CONCLUSION

The connector operates on the fundamental principle of increasing the supported area and pushing the critical circumference further from the column's edge, thereby enhancing the punching capacity. The product is designed to be fully parametric and capable of handling a wide range of loads. Additionally, it has the potential to function as a rotationally stiff (or semi-stiff) connection to the steel columns, which could positively affect the deflection and eigenfrequencies of a real slab.

A preliminary numerical analysis indicates a significant increase in the punching capacity of the CLT slab. In this specific case, the capacity is increased by a factor of 1.63, where the unreinforced slab has a capacity of $F_1 = 555.7$ kN, while the reinforced slab reaches $F_7 = 905.7$ kN. The failure mechanism shifts from a rolling shear failure in the unreinforced slab to a bending tensile failure in the outer lamellae of the reinforced slab. Failure of the glued interface is not expected, as design modifications were implemented to minimise the risk of adhesive failure. Moreover, the adhesive and cohesive strength characteristics of the epoxy glue exceed the strength of the timber itself.

This study is limited by the assumption of the linear elastic behaviour of timber in the numerical model. Consequently, any potential stress redistribution and reductions in capacity due to cracking cannot be accurately represented. Further numerical and experimental investigations are required to validate the numerical models and to identify complex failure mechanisms that are not immediately apparent and are difficult to predict

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