Structural safety and rehabilitation of connections in wide-span timber structures - two exemplary truss systems

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Summary

Following the Bad Reichenhall ice-arena collapse, numerous expertises on the structural safety of wide-span timber structures were carried out at the Chair of Timber Structures and Building Construction. It became evident that inadequate structural design and detailing as well as inadequate manufacturing principles were the main reasons for observed failures. The design and manufacture of connections in wide-span timber structures are still amongst the most challenging tasks for both the structural engineer as well as the executing company. This paper will, on the basis of two exemplary expertises, discuss specific issues in the structural reliability of connections in wide-span timber trusses and give recommendations towards a state-of-the art design of such connections.

1. Expertise 1

1.1 Introduction

The truss system to be discussed supports the roof of a 2-field gymnasium (31 x 27 m). The eight glulam trusses each span 30.6 m, resting on glulam columns. Two trusses at a distance of 2 m form a window-strip. Vertically nail-laminated timber plates, connected to the side of the bottom girders and spanning 6 m, form the roof between two pairs of trusses (see Fig. 1 and 2).

The girders and posts are glulam elements from larch lamellas and steel rods form the diagonal tension members. All joints are realized by steel plates and dowels. Due to transportation reasons, the trusses were delivered in two parts, giving the need for two main joints, one in each glulam girder.

A green roof between the window-strips results in a high permanent load, accounting for 65% of the total load.

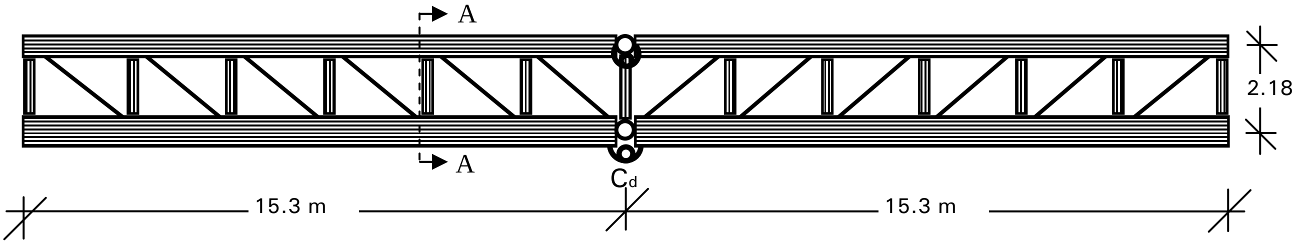


Fig. 1: Truss System

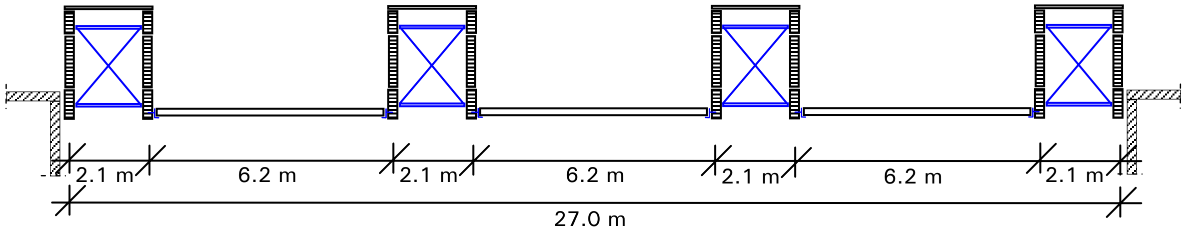


Fig. 2: Side-view of Roof Structure (A - A)

1.2 Failure Mechanisms

During the inspection of the lower girders inside the window strips, large cracks were identified which had developed around the steel-plate connection, forming the tension joint in mid-span (see Fig. 3 and 4). The crack pattern indicated failure due to block shear. Horizontal displacements of up to 10 mm between the connection block and the remaining cross-section of the tension member indicated that failure had already advanced considerably.

Opening the roof system to access the main joints from the outside supported abovementioned findings. Large (width 15 mm) horizontal cracks due to exceeded tension perpendicular to grain stresses could also be identified.

The lower girders had additionally rotated around their longitudinal axis, leading to an angle of rotation of up to 3° (see Fig. 6). This resulted in an inclination of the bottom girder which reduced the bearing area at the supports by up to 70%.

Fig. 3: Shear and Tension Cracks at the main joint

Fig. 4: Indentation of Dowel into Timber

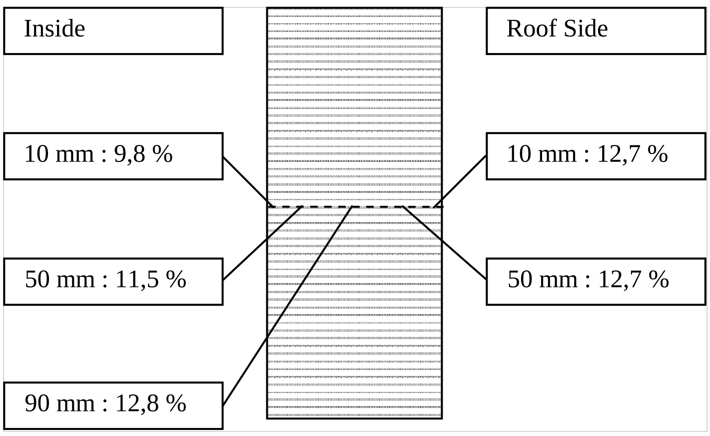


Fig. 5: Moisture Gradient in Beam Depth

Fig. 6: Rotation of Lower Girder

1.3 Reasons for failure

Reasons for block shear in main tension joints:

- The truss was calculated under the assumption of pinned connections. The calculation neglected the bending stiffness of the continuous girders as well as the rotational stiffness of the main joints. The bending moment in the girders was thereby underestimated by 50%. Combined with a higher reduction of cross-section than assumed, this lead to a 65% underestimation of the tension stresses in the lower girder.
- The substantial rotational stiffness of the main joints results in a bending moment in the connection which leads to a 49% increase of the maximum loads on the fasteners.
- In a later stage of design, it was decided to move the connection towards the upper surface of the lower girder, thus moving the centre of the connection away from the centre line of the glulam girder. The thereby generated bending moment resulted in an additional increase of the loads on fasteners of 51%. This situation was never verified by calculations.
- Shrinkage due to a reduction of moisture content from 15% upon erection to 9% in the outer parts of the cross-section during service reduced the tension perpendicular to grain strength and, in combination with high splitting forces from fasteners, facilitated the propagation of large cracks in grain direction (see Fig. 5).
- Two years after construction, the main joint was strengthened by additional steel plates which were installed below the original steel plates. This modification was carried out under full load. Therefore, the additional connection will only come into effect under additional loads like snow load. This measure decreased the maximum load on fasteners by 20% but it increased the maximum tension stresses in the lower girder by 16% due to the additional reduction of cross-section.
- The building code in effect during design requests, that for the verification of the timber side-member, the tension load be increased by 50% to account for bending moments due to eccentric load transfer. This verification was not carried out. It is exceeded by 51%. This excess explains the propagation of cracks in tension.

Reasons for tension perpendicular to grain failure:

- The nail-laminated timber plates, spanning between two pairs of trusses, are supported by L-profiles in steel, which are connected to the lower girder by screws at a distance of 160 mm from the bottom surface. The tension perpendicular to grain stresses introduced equal the tension perpendicular to grain strength. Considering, that the stresses are linearly introduced over a length of 30.6 m, leading to a large volume under stress, as well as the tension perpendicular to grain strength being reduced by abovementioned shrinkage process, the occurrence of examined tension perpendicular to grain failure can be explained.

Reasons for rotation of the lower girder around its longitudinal axis

- The connection of the nail-laminated timber plates leads to an eccentric load transfer, resulting in a torsional moment in the bottom girder. During design, the lateral stability of the individual girders was not considered. The reduction of bearing area due to an inclined bottom girder, established an eccentric load transfer to restore the equilibrium of stresses.

1.4 Strength of Material

The documentation of the manufacturing process indicated strength properties of glulam BS 16c (GL 32c). The delivery receipt indicates both BS 14c (GL 28c) and BS 14h (GL 28h) as strength classes of delivered members.

GL 32c can only be obtained by machine grading. The grading machine used is accredited for grading spruce and fir but not larch, the chosen timber for this construction. From this follows that the larch lamellas can only be visually graded, enabling a maximum strength class of GL 28. A visual grading of the outer lamellas during the inspection gave borderline values for a grading towards GL 28c.

The strength of the resorcinol glue lines could be verified by testing core samples. During this assessment it was detected that cracked glue lines had been sealed during manufacture. Opening core samples at these glue lines revealed that the crack had only been covered but not filled and that the outer areas of lamellas in these areas had never been glued together (see Fig. 7 – 9).

This finding implicated improper manufacture of the glulam members and was therefore followed by an assessment of the finger joints, joining two lamellas. It was found that some finger joints were open, having never been glued together (see Fig. 10).

Fig. 7: Crack in Glue Line and Seal

Fig. 8: Crack in formerly sealed Glue Line

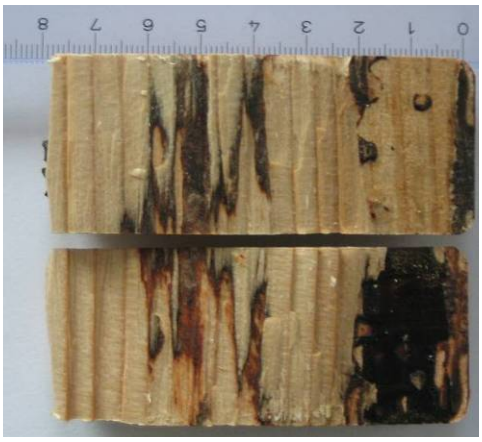


Fig. 9: Open Core Sample

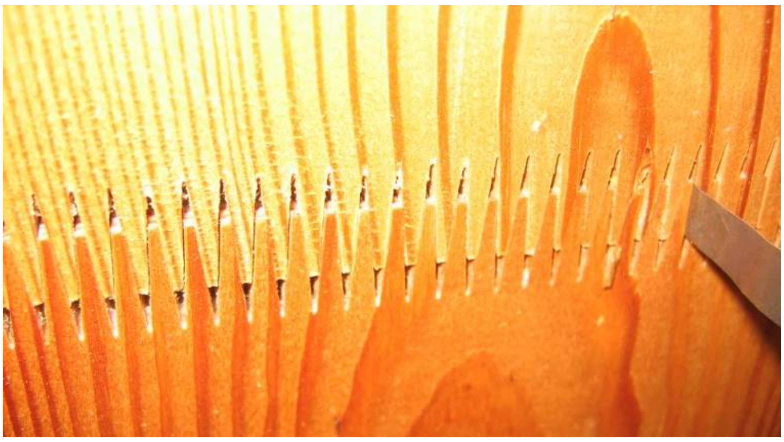


Fig. 10: Finger Joint without Glue

1.5 Rehabilitation Measures

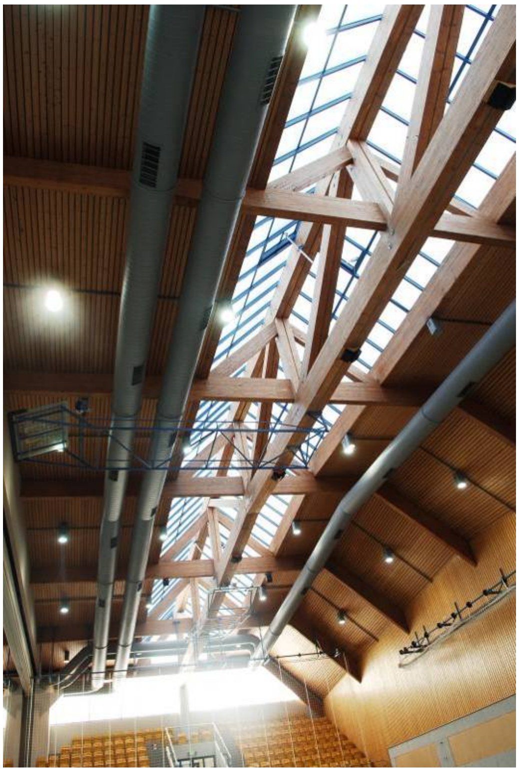
Both, strengthening structural members to compensate for exceeded strength properties and repairing measures for the failed main tension joint, are imaginable. But the impossibility to assess residual strength properties for the remaining structural members impedes a clear specification of a safety level, which is indispensable for the validity of rehabilitation measures. It was therefore decided to exchange the trusses with trusses made from LVL. Abandoning the main joints and changing the supporting system for the nail-laminated timber plates eliminates the main failure mechanisms. By temporarily supporting the nail-laminated timber plates, disconnecting one pair of trusses at a time and levying in a complete new pair of trusses, the rehabilitation of one pair of trusses can be accomplished in one day, thereby minimizing the time of exposure of the gymnasium to direct weathering.

2. Expertise 2

2.1 Introduction

The second structure to be discussed supports the roof of a 3-field gymnasium (46 x 34 m). Three main glulam trusses at a distance of 15.5 m with girders running parallel with an inclination of 5° degrees span 34 m. They are resting on reinforced concrete columns. The diagonals in the outmost field of the trusses are reinforced with flat-bar steels.

The secondary system between the main trusses consists of triple span beams at a distance of 5.7 m. These are connected to the columns of the trusses respectively are resting on reinforced concrete structures. Additionally they are coupled by trusses with inclinations of 45° degrees to the upper girder (see Fig. 12).



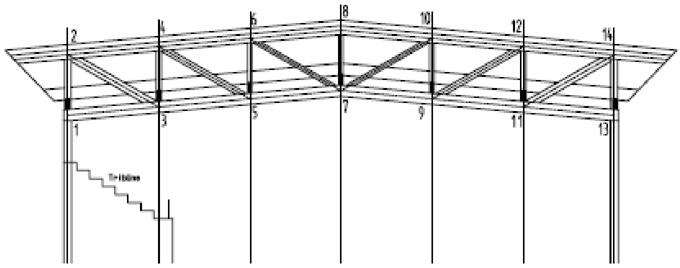


Fig. 12: Truss System

Fig. 11: Truss System

Fig.13: inclined Bar of the Secondary System

2.2 Failure Mechanisms

During the inspection, large cracks in the tension joints of the inclined trusses of the secondary system were identified. The deepest cracks had developed between the single joints of the steel-plate connection. Marking of the cracks, as it is demonstrated in Fig. 14, is generally done using the following systematic: depth – width - length of the crack.

Similar damage symptoms were indicated at the tension joints of the main trusses. Once again the cracks appeared in the area of the steel-plate connections in the lower girder. Crack reinforcements with epoxy-glue, which had previously been implemented, showed failures as well (see Fig. 15).

The main reason for the distinct crack pattern can be explained by the dry climate conditions of the gymnasium. During the inspection moisture contents of about 7.5% at the surface of the girder and about 8.5% in a depth of 90 mm were measured. The reduction of the moisture content from erection upon to the measured values caused shrinkage in the wooden component parts. In the areas of the steel-plate connections the shrinkage actions are constrained by the joints. This results in stress perpendicular to the grain which effects stress relief by the appearance of cracks. As it is already explained in the former expertise the rotational stiffness of the tension joint in the lower girder causes splitting forces from the fasteners that intensify the effect.

Fig. 14: *Inclined Truss*

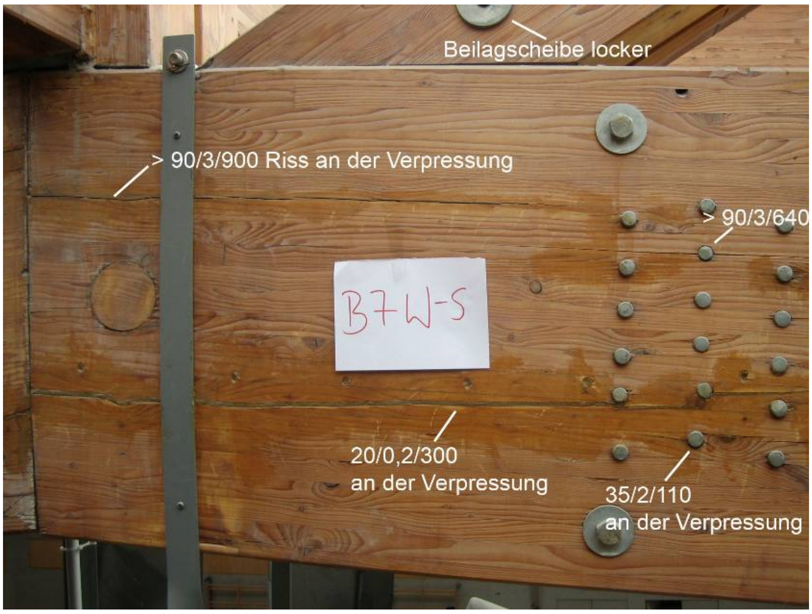


Fig. 15: *Main Tension Joint*

Furthermore the analyses of the original design showed that the verification of block shear was neglected. Considering the state-of-the-art load and safety factors, the verification of block shear is exceeded by about 50%.

2.3 Reinforcements of the inclined truss

Due to reasons of organisation it was decided to reinforce the structure temporarily for one winter period while allowing just a minimum of snow load. For this first step the joints of the inclined trusses were reinforced by screws with continuous threads, positioned perpendicular to the grain and the axe of the fasteners (see Fig. 16). To verify the strength of the reinforced connection, experimental investigations became necessary. Thereby, wooden components of a back-built ice arena were used as test pieces to take into account the influence of elements, which had already been stressed by load and changing climate conditions stressed. With these components, the inclined trusses including the steel-plate connections were replicated. Different crack patterns were simulated by cuts with a circular saw. These test pieces were reinforced by screws with continuous threads as demonstrated in figure 16, taking into account the unsymmetrical connection on both sides of the truss.

6 screws of continuous thread ∅ 8 mm, l_{thread}= 330 mm
19 joints ∅ 16 mm, l= 160 mm

6 screws of continuous thread ∅ 8 mm, l_{thread}= 330 mm
16 joints ∅ 16 mm, l= 160 mm

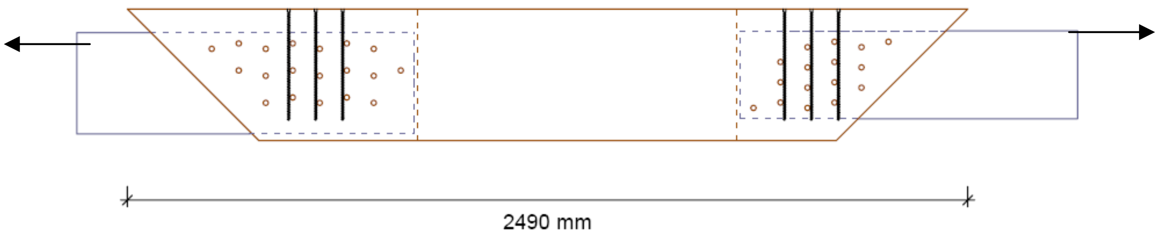


Fig. 16: Test Piece of a reinforced Inclined Truss

The test results of the three test pieces are demonstrated in the following diagram.

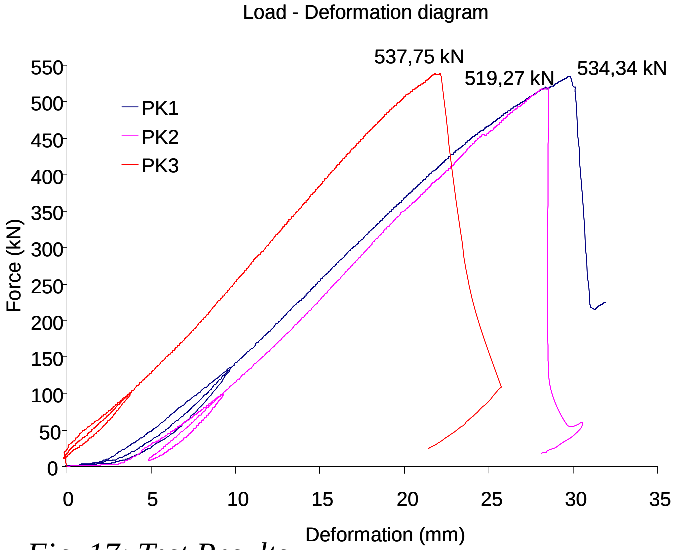


Fig. 17: Test Results

Fig. 18: Test Results

The result was that the screws could increase the strength of the joints which were damaged by cracks. Failure was always caused by block shear as it is shown in figure 18. The calculated security level according to the chosen maximum snow load was adequate to guarantee the structural safety for these components. Therefore all inclined trusses of the secondary system were reinforced by this method as a temporary measure.

In the course of extensive rehabilitations in the following summer period further reinforcements of the components were done to enable the structure to bear the total snow load. Therefore external pre-stressed tensile bars of steel were applied next to the inclined trusses (see Fig. 19 and 20). These steel bars were designed to cover the snow load, while the original inclined trusses have to carry the dead load.

Fig. 19: Pre-stressed steel Tensile Bar

Fig. 20: Connection of the Tensile Bar

2.4 Reinforcements of the main tension joint

Cracks in the area of the main tension joints in the bottom girder were grouted under pressure with epoxy-glue to recover the strength of the glue line. It seemed not reasonable to strength the tension joints with additional external steel plates against of block shear because of the incalculable distributions of forces between the original connection and the reinforcement. Hence, another measure was favoured, that enables the reduction of the tensile forces in the bottom girder of the truss system. Therefore each of the three main truss systems was pre-stressed by external tensile bars. These were placed on each side of the bottom girders. The pre-stressing of the bars was designed so that the tensile stress could be reduced to a value, which enables the verification of block shear in the main tension joint. The measures are shown in the following figures 21 and 22.

Fig. 21: Pre-stressed steel Tensile Bar

Fig. 22: Connection of the Tensile Bar

3. Conclusion

The truss, an optimized system for bearing loads, is highly dependent on its connections. Both expertises have shown the high sensitivity of connections under tensile forces in truss systems in timber. The brittle failure mechanisms in shear (block shear) and tension perpendicular to grain failure, oftentimes superimposed by stresses through moisture effects, govern the robustness of such systems.

This paper aims at facilitating the identification of these important failure mechanisms as they are exemplary for a multitude of structures.

Considering the development of codes and design principles, the authors would like to present the following considerations for discussion towards an increased robustness of such systems.

- Regulations towards a more ductile behaviour of connections
- Reducing timber grades within connections with e.g. slotted-in steel plates
- Design principles towards an increased redundancy of such systems (e.g. the clear definition of governing details in structural calculations)
- The clear definition of factors (e.g. k_{crack}, $k_{volume,}$…) for design formulas in codes (instead of encrypting them in strength values), to increase transparency, thereby increasing the awareness of engineers towards the behaviour of materials like timber.