Structural safety and robustness of connections in wide-span timber structures – evaluation of an exemplary truss system

Philipp Dietsch, Peter Mestek, Stefan Winter, Heinrich Kreuzinger

Chair of Timber Structures and Building Construction,
Technische Universitaet Muenchen, Arcisstrasse 21,
Munich, Germany; +49-89-28922044; dietsch@bv.tum.de

Abstract

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 an exemplary expertise, 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.

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 laminated beams, connected to the side of the bottom flanges and spanning 6 m, form the roof between two pairs of trusses (see Fig. 1 and 2).

The flanges 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 flange.

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

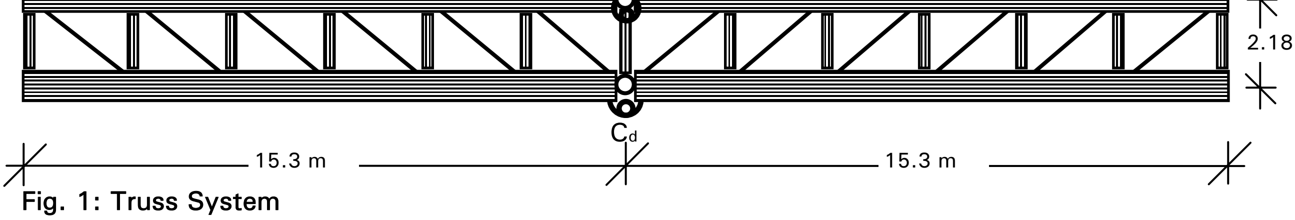

Fig. 1: Truss System

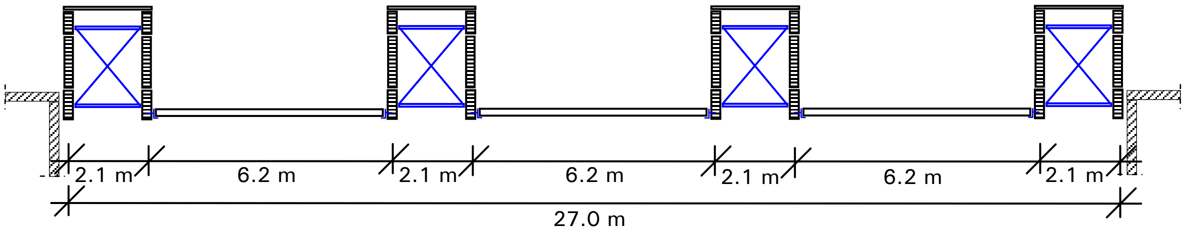

Fig. 2: Side-view of Roof Structure

Failure Mechanisms

During the inspection of the lower flanges 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.

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

Fig. 4: Indentation of Dowel into Timber

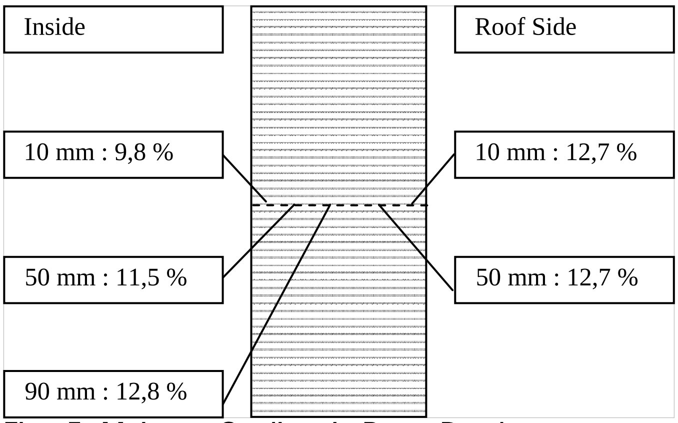

Fig. 5: Moisture Gradient in Beam Depth

Fig. 6: Rotation of Lower Flange

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 flanges 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 flange which reduced the bearing area at the supports by up to 70%.

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 flanges as well as the rotational stiffness of the main joints. The bending moment in the flanges 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 flange.

- 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 flange, thus moving the centroid of the connection away from the centroid line of the glulam flange. 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 flange 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 flange 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 flange 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 flange. During design, the lateral stability of the individual flanges was not considered. The reduction of bearing area due to an inclined bottom flange, established an eccentric load transfer to restore the equilibrium of stresses.

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

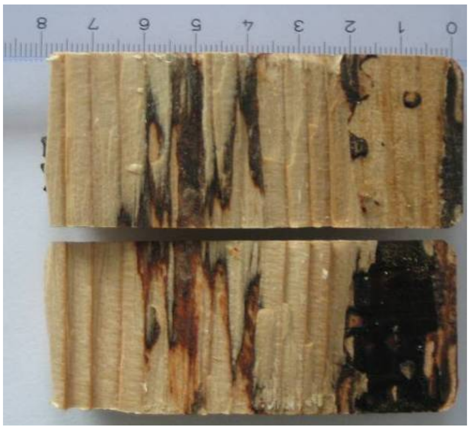

Fig. 9: Finger Joint without Glue

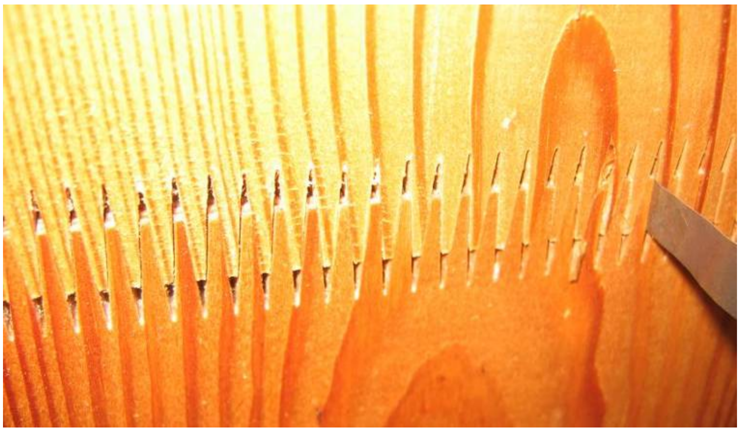

Fig. 10: Finger Joint without Glue

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.