

Robustness considerations from failures in two large-span timber roof structures

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Summary

This sheet deals with the failures of two large span timber roof structures. One is the Siemens Arena in Ballerup, Denmark and the other the Bad Reichenhall Ice-Arena in Germany. The principal structures are described as well as the flaws that are believed to have caused the failures. The robustness of the structures is discussed, including the consequences of the one structure to be statically determined and the other to be in-determined.

Keywords

Failure, large span roof, timber, robustness.

Context and relevance

The two cases described serve as examples of different design strategies for large span timber structures and their consequence for robustness. It is demonstrated that robustness is not a straight forward concept because the best strategy depends on the cause of the failure - which is obviously not known during planning and design.

The structures and their failures

Siemens Arena

The cycling arena was built in 2001. The main roof structure consisted of 12 trusses, each truss composed of two glulam timber arches with vertical connectors, see Figure 1. The upper arch was mainly exposed to compression, the lower to tension. The horizontal component of the tension and compression forces were neutralised at the corner connections, realized with concealed steel plates which were connected to both arches by embedded dowels and a few bolts, see Figure 2. The structure appeared as an elegant slim construction with a free span of 73 metres across the arena. The distance between the trusses was 12 metres. The secondary structure consisted of simply supported purlins.

Two of the trusses collapsed without warning at a time with almost no wind and only a few millimetres of snow. The partial collapse happened just a few months after the inauguration of the arena. No people were present in the arena during the collapse.



Figure 1: The Siemens Arena roof structure after the collapse of two trusses. An intact truss can be seen on the right



Figure 2: Corner connection with concealed steel plates, connecting the timber parts through bolts (visible) and dowels (not visible)



Figure 3: Rupture at the critical cross section in the corner connection. Note the dowels and steel plates

An investigation showed that the cause of the failure could be localised to one critical cross-section in the tension arch near the support, where the strength was found to be between 25 and 30% of the required strength, see Figure 3. By mistake, this cross-section was not considered at all in the design. Three critical design errors were identified:

- A 48% too high design strength was used for the timber part

- The reduced height of the cross section near the ends of the arches, see Figure 2, was not considered
- The reduction of the timber cross section due to steel plates, bolts and dowels, see Figure 3, was not considered

The expected short term load-carrying capacity at the critical cross section happened to be only slightly larger than the loads from the self weight of the structure. Because the strength of timber is reduced over time when it is loaded (the k_{mod} -effect) it is likely that the collapse took place when the strength was reduced to the stresses caused by the self weight. According to Eurocode 5 and confirmed by Hoffmeyer and Sørensen the reduction factor for medium duration loads (1 week to 6 months) is $k_{\text{mod}} = 0,8$. Such a reduction is enough to explain how the collapse could take place at a time with no special external load.

The investigation also revealed that the stability of the trusses was not ensured sufficiently and that the quality of the gluing of the glulam was not as specified. These problems, nevertheless, did not contribute to the actual failure.

The collapse did not occur due to an unknown phenomenon. The design of the trusses was not checked by the engineer responsible for the entire structure due to unclear specification of the responsibility and duties of that engineer. This might explain why such a vital error could pass the quality assessment of the design. The demands to the quality assessment of such structures in the Building Regulations have been increased after the incident. An independent third party control is now required.

Bad Reichenhall Ice-Arena

The contents of the following are derived from: Winter, S., Kreuzinger, H., 2008 “The Bad Reichenhall Ice-Arena collapse and the necessary consequences for wide span timber structures”.

The Bad Reichenhall Ice-Arena, built in 1971/1972, was a structure of approx. 75 m in length and approx. 48 m in width, see Figure 4. The roof was supported by 2.87 m high main girders, which were produced as timber box-girders, see Figure 7. The box-girders featured upper and lower glulam members and lateral web boards made from so-called “Kämpf web boards” - a type of cross laminated timber. The 48 m long girders were produced from three 16 m long sections, which were joined with general finger joints.

The secondary system was fixed to the sides of the girders and acted both as purlins and as lateral bracing (K-bracing) for the main box-girders, see Figure 6. This enabled the roof structure to redistribute loads between the girders.

On the 2nd of January 2006, the entire roof collapsed without warning during a period of significant snowfall, see Figure 5. However, the snow load was not above the characteristic snow load used in the design.

The design and construction of the girders were identified as one cause for the failure. There were several contributions to the failure:

- The review of the structural calculations revealed two important errors. For the assessment of the load-bearing capacity of the box girder, the bending strength of the glulam was applied, rather than the tensile and compressive strength of the lower and upper girder. In addition, the weakening of the cross-sections due to the general finger-joints between the girder parts as well as the web-boards was also not taken into account in the structural calculations. Comparative calculations, based on the technical rules applicable at the time of the construction of the Bad Reichenhall Ice-Arena, have shown that the safety factor was only of a magnitude of around 1.5, whereas the required factor was 2.0.
- For the box girder with Kämpf web-boards, a general technical approval was available, which however, limited the height of the web-girders to 1.20 m. Therefore, a so-called “Approval for an Individual Case” by the Supreme Building Authority would have been necessary for executing this special structure. According to the findings to date, such an approval was not applied for. An application in 1971 to extend the general approval to larger heights was not granted by the German Institute for Building Technology. Further, the production of the vertical general finger joints of the web-boards must be regarded as difficult and not very robust. The quality of the glue lines in these finger joints differed significantly.
- The box girders were produced using urea-formaldehyde glue. The technical rules both then and now allow the use of this type of glue for load-bearing components only in a dry ambient climate because that glue is not permanently moisture-resistant. Today it is known that unheated and non-air-conditioned ice-arenas represent a particularly critical climate for moisture-sensitive components. Besides a high relative humidity, the thermal radiation between the ice surface and the roof parts facing the ice surface leads to cooling and thereby increased condensation on these parts of the roof structure. Since that knowledge did not exist in 1972, the use of urea-formaldehyde glue for bonding the load-bearing components did not generally violate the state-of-the-art of technology at that time. However, the general technical rules for using the Kämpf web-boards required the connections between the glulam-girders and the web boards to be carried out with the significantly more elastic resorcinol glue (RF), due to the thick bonding gaps.
- The gluing of the blocking between girders and web-boards at the supports did not correspond to the recognised rules at the time as gluing secured by nails was and is limited to a board thickness of max. 35 – 50 mm.



Figure 4: The Ice-Arena in Bad Reichenhall



Figure 5: Partial view of the collapsed roof structure

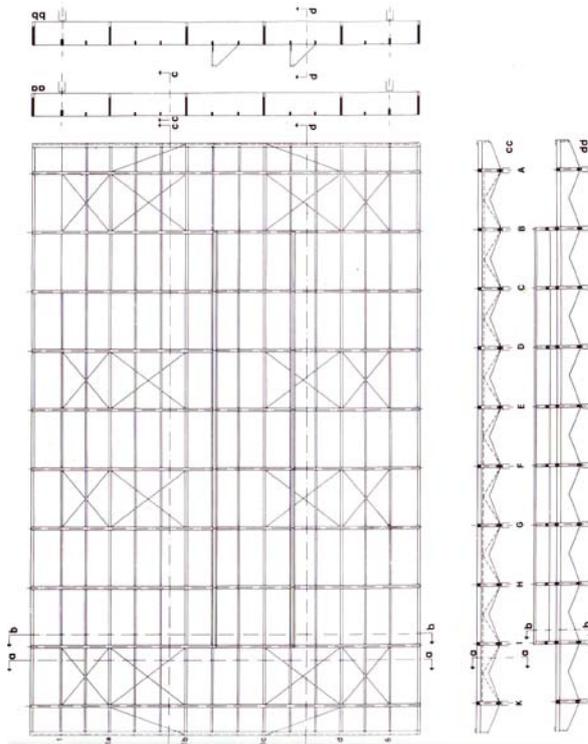
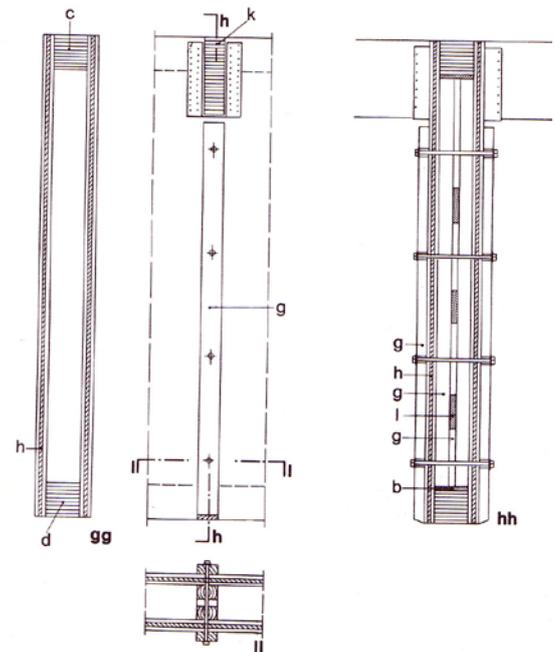


Figure 6: Layout of the roof of the Ice-Arena and side view of the girders (Schmidt, H.J. et al. (1974))



c-upper girder, d-lower-girder, h-Kämpf web-board, k-purlin, g-reinforcement (to avoid buckling)

Figure 7: Cross-section of the box-girders in the Ice-Arena (Schmidt, H.J. et al. (1974))

Due to the humidity exposure over the years, the glue-lines and finger joints were significantly damaged. This primarily affected the general finger joints in the lower girder and the bonding between the girders and the web-boards. It was found, that in some cases, there was no adhesive effect to a depth of 50 mm to 80 mm in the lower girders. In combination with the errors in the structural calculations, this represents the most significant cause for the collapse of the Bad Reichenhall Ice-Arena.

In addition, there were repeated cases of water penetration as a result of leaks in the roof membrane and in the area of the roof drainage. These were not permanently rectified and the roof structure did not receive renovation paint during the use of the arena. Whether such measures would have significantly delayed the damage to the glue lines of the roof structure cannot be answered.

The structural calculations for the roof structure appear not to have been examined by a check engineer, even though this was obligatory for special buildings, e.g. assembly or sport halls. Further it is not documented that any professional examination of the structural integrity has been carried out as part of the maintenance of the building.

The arena was initially open on two sides. It is not believed that there were any disadvantageous effects from the subsequent enclosure. Neither is a settlement of the concrete structure thought to have caused significant impact on the roof structure.

Robustness considerations

Siemens Arena

During design it was decided that the 12 m long purlins between the trusses should only be moderately fastened to the trusses, such that a failure of one truss should not initiate progressive collapse. This strategy proved to work fairly well as 'only' two of the 12 trusses collapsed. Considering that all trusses had a much lower strength than required it might be fair to conclude that the extent of the collapse was *not* disproportionate to the cause. Each truss becomes a key element.

Another, and perhaps more expensive, strategy against progressive collapse could have been to design the trusses, the purlins and their connections such that a failed truss and the roof could hang in the purlins and transfer the load to the neighbouring trusses (when considered an accidental load case). Had this strategy against progressive collapse been chosen it is most likely that progressive collapse would have occurred because the neighbour trusses could not have withstood the extra load from the truss failing first. In this case the trusses are not key elements.

But had the cause of the failure been a huge load on one truss or a lone standing mistake in one truss, the second strategy would have been preferable because it significantly reduces the risk of injuries. That strategy might also have worked if e.g. a leaking roof had degraded one truss because it is likely that the other trusses remain unharmed. In that case large deformations would occur, giving a warning of possible failure.

There were two bracing systems in the longitudinal direction, one at each gable. This ensures stability of the remnant part of the building when one truss has failed. This strategy also proved successful, even though there was no wind or snow to call for big demands to the bracing system.

Bad Reichenhall Ice-Arena

Robustness has not been considered, neither during design nor during the lifetime of the building.

The investigation showed that the first failure occurred in one of the three main girders on the east side. Due to the stiff cross bracing, the loads were shifted from the girder that failed to the neighbouring girders. Because these girders suffered from the same mistakes and degradation processes as the girder failing first, they could not sustain the additional load. Consequently, this developed into a progressive collapse.

The ability to redistribute loads, often called redundancy, is generally regarded as favourable for the robustness of a structure because a random local failure will not cause total collapse. This also means that the box-girders are not key elements in the usual meaning of the term. But since the secondary structure was not only strong but also very stiff, a weak girder would transfer its loads to the adjacent girders without any warning from large deformations. This means that e.g. some general finger joints could have lost their strength long ago.

A more robust system could have been achieved in various ways:

- A strong but softer secondary system could give warning about redistribution of load taking place due to increasing deformations. Since the secondary structure also had to fulfil the purpose of bracing against lateral-torsional buckling of the main girders, it needed to be stiff. If both requirements should have been fulfilled, a different bracing system would have been needed.
- A statically determinate secondary system with connections, which would allow for one girder to collapse, without increasing the load on the adjacent girders.

Discussion

It should be noticed that neither of the projects for the structures had undergone an independent checking of the design and calculations.

The Siemens Arena was a statically determined structure, whereas the Bad Reichenhall Ice-Arena was a highly statically undetermined structure, with a very stiff secondary structural system.

In the Siemens Arena the weakness was present from the beginning and quite similar in all trusses. The collapse might have been initialized just by the strength reduction over time. The chosen robustness strategy with weak purlins limited the collapse. The design error was so large that is unlikely that any robustness strategy would have been able to prevent such a collapse; other strategies might even have caused a total collapse. Had it been a local incident which had caused a failure the collapse might have been restricted to one truss. But nearly 2000 m² of the roof falling down might in that case have been considered as disproportional to the cause.

In the Bad Reichenhall Ice-Arena the weakness was initially much smaller but developed over time, and presumably at different rate in different areas of the roof. The collapse was eventually initialized by a severe but nowhere exceptional snow load. Local damages have most likely been present for a long time, but the stiff structure was able to transfer the load to other parts, without significant deformations. A local incident causing a local failure might therefore never have been revealed. A strategy with a less stiff secondary system would presumably have issued a warning both about the general degradation of the strength as well as damage on a local level.

Outlook

The lessons learned from the two cases are included in the discussion in the Fact Sheet “Robustness design of timber structures – secondary structures” by Philipp Dietsch.

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