Erik Serrano

An overview of failures in large-span timber structures

SP AR 2006:14
Bygg och Mekanik
Borås 2006

Abstract

A total of 18 failures (collapse, crack formation, excessive deformation etc) in Swedish timber structures from the period 1978 - 2005 were reviewed.

The most common reasons for failure was found to be the lack of design or poor design in relation to mechanical loading, which was the primary cause of failure for nine of the cases. In these cases the presence of notches and holes, at which the high stresses perpendicular to grain develop, was often the main cause of failure.

In six of the eighteen cases it was concluded that the failure was caused by errors done in the manufacturing of the product (mainly glulam). Examples of this category include, for example, gluing errors (lack of hardener, poor gluing pressure etc.)

The main conclusion from the investigation is that in none of the cases was the wood raw material the cause for failure. Instead, all failures were either due to the lack of knowledge (lack of design knowledge, lack of knowledge about sound production methods) or due to poor quality control procedures.

Key words: timber structure, failure, collapse, glulam, glued laminated timber, wood, wood products

SP Sveriges Provnings- och Forskningsinstitut
SP AR 2006:14
Borås 2006

SP Swedish National Testing and Research Institute
SP AR 2006:14

Postal address:
Box 857,
SE-501 15 BORÅS, Sweden
Telephone: +46 33 16 50 00
Telex: 36252 Testing S
Telefax: +46 33 13 55 02
E-mail: info@sp.se

Contents

Abstract		**3**
Contents		**5**
Preface		**7**
1	**Introduction**	**9**
1.1	Background and aim	9
1.2	Scope and limitations – selection of cases	9
1.3	Outline of the report	10
2	**Cases studied**	**11**
2.1	Case 1 – Glulam arched beams with notches	11
2.2	Case 2 – Failure of a roof beam at building site	12
2.3	Case 3 – Cracks in roof beams with holes	13
2.4	Case 4 – Delamination in roof beams	15
2.5	Case 5 – Glulam purlins failure	16
2.6	Case 6 – Failure in roof beam with notches	17
2.7	Case 7 – Failure of a roof structure during erection	19
2.8	Case 8 – Cracking in arch structure	21
2.9	Case 9 – Cracks in glulam roof structure	22
2.10	Case 10 – Failure in tapered glulam roof beam	24
2.11	Case 11 – Cracking in tapered glulam roof beams	25
2.12	Case 12 – Failure in beams at steel connections	26
2.13	Case 13 – Cracks in glulam arches	28
2.14	Case 14 – Cracks in glulam beams and columns	29
2.15	Case 15 – Delamination in glulam beams	31
2.16	Case 16 – Collapse of roof structure in a school	33
2.17	Case 17 – Buckling of roof structure	34
2.18	Case 18 – Cracking in glulam frame	37
3	**Discussion and concluding remarks**	**39**
3.1	Failure categories	39
3.2	Quantification of failure cases	40
3.3	Concluding remarks	42

Preface

This report is a part of an international survey of failures in timber structures, and is the SP contribution to sub-task C in the project "Innovative design, a new strength paradigm for joints, QA and reliability for long-span wood construction".

The project partners are:
VTT, Lund University, Växjö University, SP – Swedish National Testing and Research Institute, Casco Products AB, Exel Oyj, Fastening Systems, Finnforest, Late-Rakenteet Oy, Limträteknik i Falun AB, Svenskt Limträ AB, Skanska Teknik AB, Skogsindustrierna, SFS-Intec, Spu-Systems Oy, Versowood.

The project is jointly financed by TEKES – The national technology agency of Finland, VINNOVA – Swedish Agency for Innovation Systems, and by the industrial partners in both countries. The financial support from these organisations is gratefully acknowledged.

In my work with this report I have had the privilege to discuss old failure cases with several colleagues and friends, some of them officially involved in the project and others with special knowledge about specific failure cases. Instead of mentioning them all, I express a collective gratitude to all of them who have contributed with their knowledge about these cases.

Växjö 2006-06-27

Erik Serrano

1 Introduction

1.1 Background and aim

Wood and wood-based products are sometimes considered as having a lower status and being related with higher risks among practising engineers and other professionals in the building industry. The reason for this can probably be contributed to the fact that wood has a high variability in comparison with other materials. The building industry has also been more traditional when dealing with timber structures, than compared with steel and concrete, where well established quality assurance procedures are being used.

Two recent and highly noticed failures in large glulam structures (Jyväskylä (FIN) and Ballerup (DK)) have furthermore drawn the attention to the reliability of large timber structures, and especially the design methods for large scale dowel-type joints. With these two failures in mind, a joint Finnish-Swedish research project, “Innovative design, QA and reliabilty for long span wood construction”, was started in 2004.

The main objective of the project is to improve the competitiveness of timber structures. The project deals with design methods for large scale dowel-type joints, the development of new type of joints, and with the development of quality assurance (QA) procedures. The development of QA-procedures is furthermore divided into two main sub-tasks: an international survey of errors and failures in buildings with timber as a structural material and the development of a QA-program for the design and construction of demanding timber structures. This report includes the results from one of the surveys of failures performed, the contribution of project partner SP – Swedish National Testing and Research Institute. By performing such a study, the aim is to learn from failures in the past. By looking into the main failure reasons, and implementing QA-procedures to avoid such errors in the future it is believed that the reliability perception of large span timber structures can be improved.

1.2 Scope and limitations – selection of cases

In total 18 cases have been studied. These are failure cases that were available through the archives at SP dating back to 1978. The normal procedure at SP means that reports are archived at the central archive at least 10 years, but fortunately the Wood Technology department has a parallel set of documents for reference.

The information presented here was deliberately made as short as possible, mentioning only crucial details playing a part in the failure mechanisms. In this way it has been possible to present the data without mentioning any manufacturers, building contactors or other third parties involved. The cases are reported in random order.

The original reports written during the investigations performed have been studied and in some cases additional interviews were made with the people involved in the investigations.

Clearly the selection procedure chosen plays a crucial role when performing this type of study. The cases presented here are the ones known to SP in terms of SP being involved in the investigation. In what extent these cases are representative for all failures in long-span timber structures remains to be proven.

1.3 Outline of the report

Following this introduction, the cases studied are reported in Chapter 2. Here each case is described by giving a short description of the structure in question, a description of the failure and the original investigation performed by SP. Additional comments and possible new explanations for the cause of failure are given.

Chapter 3 gives a short overview of all cases, some statistics and discussion and general conclusions drawn from this study.

2 Cases studied

2.1 Case 1 – Glulam arched beams with notches

2.1.1 Description of structure

The structure consists of a number of glulam arched beams, 21,5 m in span. The building was erected during winter, and approximately one year after its inauguration, the failure occurred. During the erection of the beams, notches were cut to fit the support. The notches were approximately 160 mm deep, and the remaining depth at support approximately 500 mm. The cuts, which were done on site, thus correspond to about 24% of the beam depth.

2.1.2 Description of failure

One of the arched beams was found to have a 7-8 m long crack, which seemed to originate from the notch at the support. The crack path followed the main fibre direction in the beam and only on short parts did it follow any bond line.

2.1.3 Original investigation performed and conclusions

An on-site inspection was performed by SP. At the inspection, two more beams were found to have similar cracks, although of considerably shorter length. In addition to this bleeding from bond lines of the beams with the large crack was seen. The bleeding can indicate that a too a low amount of hardener had been used. Apart from the inspection on site, the moisture content of the beams was determined, samples were also taken in form of drill cores, and the type of failure at the drill locations was determined.

The investigation concluded that the failure was due to high stresses perpendicular to the grain at the notch. Since the crack had only followed the interlaminar bonds very locally, it was also concluded that the bond line strength had not been of any importance for the failure.

2.1.4 Additional conclusions and comments

The conclusions of the original investigation seem adequate. The failure can be classified as being due to bad practice on site (assuming that the notches were not in the original design), bad knowledge about design of notched beams (assuming the notches were included in the original design).

2.2 Case 2 – Failure of a roof beam at building site

2.2.1 Description of structure

The structure in this case was a roof beam, a double tapered beam with a span approximately 11 m. The beam depth was approximately 350 mm at the supports and 690 mm at mid-span. Two circular holes of diameter 300 mm were placed symmetrically, 500 mm from the mid-span. A schematic of the beam is shown below in Figure 1.

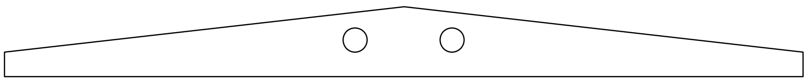

Figure 1. Schematic of the double tapered beam.

2.2.2 Description of failure

The failure occurred during construction of the building. The roof beams had been erected and covered with corrugated steel. Thermal insulation and roof decking materials were lifted up to the roof. The personnel on the building site had got instructions to place the material over the beams, and especially to place heavy pallets close to the support. Just as the first beam was covered with pallets, a sudden and catastrophic failure occurred. The beam fell to the ground.

2.2.3 Original investigation performed and conclusions

SP personnel made an on site inspection, and the failed beam was put together on the ground. The failure pattern is shown in the schematic of Figure 2. The first 1.2 m of the crack denoted A in Figure 2, ran in a interlaminar bond line, which was of very low quality. In the first 20 cm of crack A, measured from the support, only dried hardener was found. In the rest of the crack glue was found on both surfaces, but the bond line was light coloured. This indicates that the amount of adhesive applied had been small. The amount of wood failure in crack A was also very small. The beam had been connected to the column by means of nailing plates. The plates were placed such that one row of nails split the bond line.

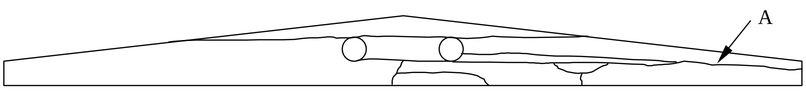

Figure 2. Failure pattern of the failed beam.

In addition to inspecting the failure surfaces, test samples were taken from the failed beam to perform delamination tests. The delamination was for all cases well below the stipulated margins according to ASTM 1101-59.

The conclusions of the original investigation were that the poor bond line acted as an initial crack, a situation that was worsened by the nailing of a row of nails into it.

2.2.4 Additional conclusions and comments

The conclusions of the original investigation seem adequate. The failure can be classified as being due poor manufacturing of the glulam. In addition to this, the nailing at the support worsened the situation, but a bond line of normal quality would not have been affected.

2.3 Case 3 – Cracks in roof beams with holes

2.3.1 Description of structure

The roof structure of the building consists of 6 double-tapered beams with a free span of 20 m placed at 6 m distance. The beam height was 765 mm at the supports and 1870 mm at the mid-span. Four of the beams had rectangular holes at the support. The holes were 146 mm deep and 400 mm wide, see Figure 3

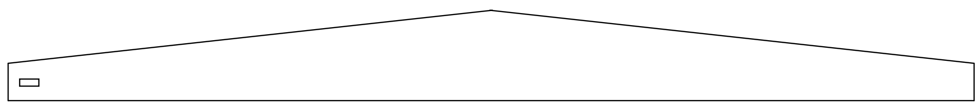

Figure 3. Schematic of roof beam with hole at the support

2.3.2 Description of failure

When replacing the lights in the building it was discovered that several large cracks had opened and an investigation by SP was commissioned.

2.3.3 Original investigation performed and conclusions

SP-personnel performed an on-site inspection. The beams were visually inspected and the moisture content of the beams was measured with a resistance meter. The beams with hole at the support had all cracks starting from the upper corner and running along the beam length. These cracks were from 65 to 750 mm long and 50-65 mm deep. One beam had cracks also at the lower corner of the hole, see Figure 4.

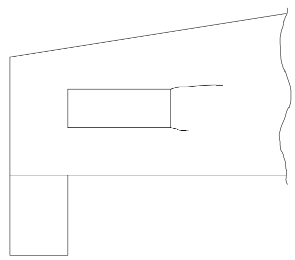

Figure 4. Schematic of cracks at hole.

In addition to the cracks at the support, two beams also showed considerable cracking at mid-span. One beam had a single 5 m long crack and one beam had two cracks 2 and 4 m long, respectively. The cracks were situated in the 10th bond line from the lower edge (5 and 4 m cracks) and at the 6th bond line from the lower edge of the beam (2 m crack). At the inspection it was also noted that the glulam was made from timber that in the outer parts of the beam did not fulfil the Nordic Glulam standard in terms of quality (knot sizes). Several finger-joints also showed a poor quality in terms of distance from knots and in terms of gaps between the fingers.

The investigation concluded that the cracks in the beams were due to mechanical loading, due to poor design of the holes. It was recommended that the holes be reinforced. It was also recommended that the moment capacity of the beams should be checked, taking into account the low quality of the timber used and the poor quality of the finger-joints.

2.3.4 Additional conclusions and comments

The conclusions of the original investigation seem adequate. The failure can be classified as being due to poor design of the beams, with inadequate consideration of the stress concentration at the holes. It is not known if the holes were manufactured with the proper corner radii, neither if the holes were made at the glulam manufacturer or on site. In addition to this, the poor quality of the glulam and finger-joints may have worsened the situation.

2.4 Case 4 – Delamination in roof beams

2.4.1 Description of structure

The structure is an old roof structure build with prismatic glulam beams. The year of erection is unknown, but probably the building was erected during the1960s. The building was extended in 1983.

2.4.2 Description of failure

In 1985 damages from water were found. The roof structure was inspected and extensive moisture induced damage was found, but also serious damage to the old glulam beams. The damage of the glulam beams led to extensive deflection of the beams, and since the damages increased rapidly in time, it was decided that the old roof beams would have to be replaced.

2.4.3 Original investigation performed and conclusions

SP was commissioned to perform an investigation to conclude the reason of the failure. An on site inspection was performed. The beams that had been replaced were inspected.

It was found that moist air was leaking from the new part of the building into the roof structure of the old part. When the moist air hit the cold parts of the structure it condensed, free water was also found. Microscopy studies of the bond lines was performed. The moisture content in the beams was found to be between 15 and 26 %

It was concluded that the old beams were manufactured with a cold-setting acid-curing adhesive. These types of adhesives were commonly used in the 1960s. In some cases it is possible that the acid from the adhesive is deposited if an excess of acid is available and if the bond line is subjected to high moisture contents. The acid in turn degrades the wood resulting, in time, in total loss of structural integrity.

2.4.4 Additional conclusions and comments

The conclusions of the original investigation seem adequate. The failure can be classified as being due to the use of an adhesive type that can be sensitive to high moisture contents. The failure is thus due to poor material in combination with building physics related issues. The cold setting type of adhesive used was banned some 35 years ago. SP has during the last 25 years performed a large amount of investigations related to the use of this type of adhesive.

2.5 Case 5 – Glulam purlins failure

2.5.1 Description of structure

This structure is a roof structure consisting of large double tapered glulam beams, with a secondary structure (purlins) also made from glulam. The purlins were 6 m long.

2.5.2 Description of failure

The failure occurred in early spring following a winter with heavy snowing. The failure was a tensile bending failure in one of the purlins. The purlin did however not fall to the ground, but due to excessive deformation it acted in almost pure tension.

2.5.3 Original investigation performed and conclusions

At failure, a local consultant inspected the building. The consultant was contacted in 2004 and interviewed for further information not available in the original report. From the interview it was found that the snow load on the roof, had been measured and was found to be approximately 400 kg/m^2. In addition to the snow there was about 5-10 cm of ice below the snow. The roof had a pitch of 14°, and the failed purlin was situated on the leeside. The snow load had become extremely unsymmetrical, due to wind. At the roof top only a few centimetres of snow was found. The failure of the purlin seemed to be initiated at a finger-joint.

The failed beam was sent to SP for investigation. The failed finger joint was inspected. It was found that the failure had probably been initiated at a finger-joint in the outermost lamination, and a crack had propagated upwards through a joint in the second lamination. An intact finger-joint of the beam was tested in bending and was found to be of excellent quality. No indications were found of adhesive failures/poor gluing, rot or other biological degradation. The wood had good strength properties. The production journals of the manufacturer of the glulam were surveyed, and no irregularities were found. The conclusion was that the beam material, including finger-joints and interlaminar bonding had no part in the failure.

2.5.4 Additional conclusions and comments

The conclusions of the original investigation seem adequate. The failure was not caused by bad design or erroneous materials. From the additional information obtained in the interview it can instead be concluded that the failure was caused by the heavy loading on the roof. The local snow load on the most loaded part of the roof exceeded the design load according to the code, which at the time was 2.0 kN/m^2 (exceptional load case).

2.6 Case 6 – Failure in roof beam with notches

2.6.1 Description of structure

This structure was a large hall, with a roof consisting of double tapered beams, resting on columns and/or wall beams. At the support, notches had been cut and reinforced with bonded-in rods. The roof beams were 31,5 m in span, approximately 2 m deep at mid-span and 1100 mm deep at the supports. The notches cut at the support were 26 and 35 % of the beam depth respectively (calculated at the inner side of the notches). A schematic of the roof beam is shown in Figure 5.

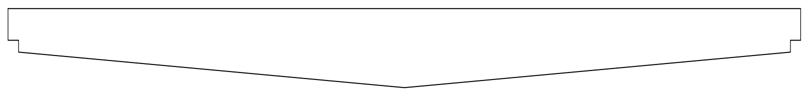

Figure 5. Schematic of roof beam (not to scale).

2.6.2 Description of failure

A sudden failure occurred, producing a loud noise, and resulting in a crack running from the corner of the notch at one of the supports. The crack was through the width of the beam, measuring 12 m on one side and 9 m on the other. It initial width, or crack opening, was 50 mm. The vertical side of the notched was displaced approximately 15 mm. After the failure it was decided that immediate actions would have to be taken to reinforce the structure, since there was an imminent risk of total collapse. Temporary columns were placed underneath the roof beams, close to the supports.

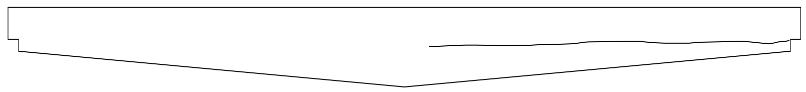

Figure 6. Schematic of roof beam (not to scale).

2.6.3 Original investigation performed and conclusions

An extensive investigation of the building was performed by SP. The investigation included on site inspection, measurements of beam geometries, deflections and what was found to be initial deformations. Samples were also taken to test bond-line strength and moisture content.

A very large amount of more or less serious errors were found. Here only the most serious ones are reported.

The roof beams had been manufactured to a mid-span depth less than the final depth of the beam using traditional technique. After this, two additional laminations had been nail-glued to the beam in order to achieve the final depth. The nail-glued laminations were not end-joined at the apex.

At erection the roof beams had not been stabilised properly (or not all) until all roof beams were put in place. This lead to an initial deformation of the beams, which were leaning and also had a horizontal deflection. For one beam, the total horizontal deformation at mid-span was estimated to 110 mm.

The wind-stabilising trusses had not been properly post-tensioned, leading to an more or less non-active structure in terms of wind stabilisation.

The roof beams had been designed without accounting for the deformation of the wall beam. This lead to non-uniform load sharing, thus roof beams resting on columns taking greater load than the roof beams resting on wall beams.

The design of the notches was inadequate. The formal shear strength at the support was approximately twice the design value. A separate fracture-mechanics based calculation mad in the investigation, concluded that the average failure load was in the range of the actual load at failure. The failure load was estimated to approximately 120-140 kN shear force, and the actual load was estimated to 140 kN.

It was concluded in the investigation that the failure was caused by the high stresses perpendicular to the grain at the notch. The situation was worsened by many other factors, such as the initial deformation and the non-uniform load sharing of the roof beams, but it was estimated that the roof eventually would have failed irrespective of these anomalies.

2.6.4 Additional conclusions and comments

The conclusions of the original investigation seem adequate. The failure can be classified as being due to poor design of the beams, with inadequate consideration of the stress concentration at the holes. Even if the building code used at the time of design did not correctly handle the notched beam design, the code values of permitted stresses were exceeded.

2.7 Case 7 – Failure of a roof structure during erection

2.7.1 Description of structure

The roof structure of a complete building was assembled on the ground. The structure consisted of four trusses according to Figure 7. The trusses were joined by purlins, no additional lateral bracing was present. The total span of the trusses was approximately 42 m and the distance between the trusses was approximately 7,2 m.

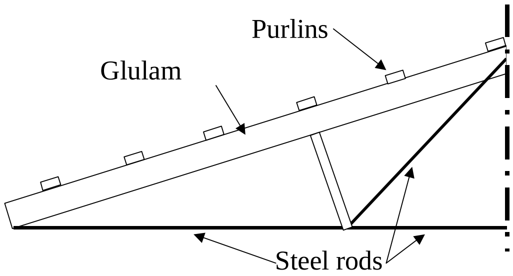

Figure 7. Combined glulam-tension rod truss (not to scale).

2.7.2 Description of failure

After finalising the roof structure with four trusses and purlins, a crane was used to raise it in place on top of supporting columns. The straps used during rising of the structure were placed according to Figure 8. The forces in the straps lead to a compressive force in the glulam parts, as indicated below. In addition, the purlins are also subjected to compressive forces, se Figure 9. Just before putting the roof structure down on its supports, the structure collapsed. According to witnesses, one of the outermost trusses tilted and the structure failed.

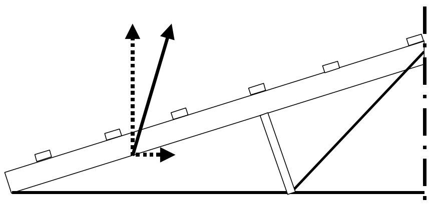

Figure 8. Strap placement.

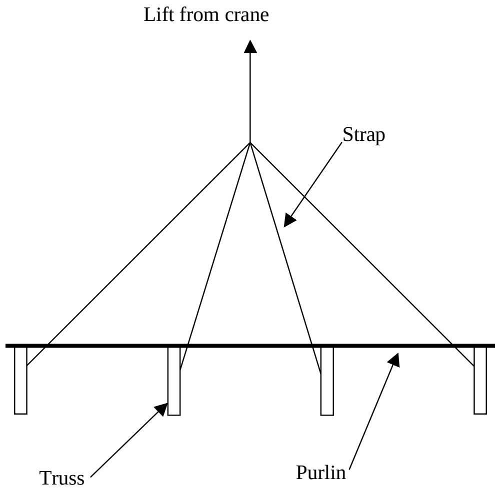

Figure 9. Strap placement.

2.7.3 Original investigation performed and conclusions

SP personnel conducted an inspection on site. The beams were investigated with regards to failure surfaces, amount of glue failure in finger joints, delaminations etc.

The conclusion from the original investigation was that buckling of the purlins caused the failure, due to an excessive compressive force being applied. A simple hand calculation of the forces was made showing that the normal force in the mostly stressed purlin exceeded the design value by a factor of 3. The positions of the straps during the raise were possible to determine due to local compressive failure of the wood.

The investigation also showed that the glulam had several defects, including defect finger-joints. It was, however, concluded that these had no influence on the failure.

2.7.4 Additional conclusions and comments

The conclusions from the original investigation seem adequate. The failure can be classified as being due to poor handling during erection. The roof structure should have been further stabilised during the raising. The local crushing of the wood at the straps should alone be a reason for stopping the raise.

2.8 Case 8 – Cracking in arch structure

2.8.1 Description of structure

The structure was built in 1934 and consists of six parallel arches, which act as statically indeterminate load-bearing frames. The arches have I-shaped cross-sections and web stiffeners at every 4 m. The flanges are held together with bolts going through the beam at the stiffeners. The structure was originally used as an outdoor theatre arena. In 1983 the building was subject to renovation and insulation was placed in the roof, in order to make the theatre an indoor facility. The insulation was placed in between the arches, not on top of the existing roof.

2.8.2 Description of failure

When additional restoration work was conducted in 1988, a large crack was found in one of the arches. No data on when the crack had developed was available. SP was commissioned to perform an investigation of the structure.

2.8.3 Original investigation performed and conclusions

An on-site inspection was performed. The crack that had led to the call for an inspection was measured, and is schematically shown in Figure 10.

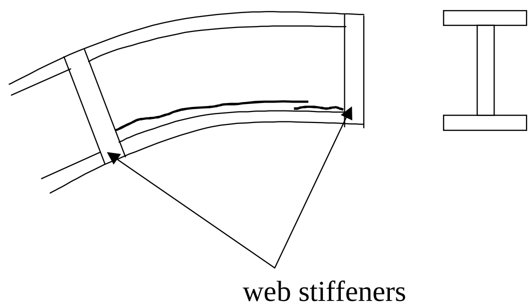

Figure 10. Schematic of arch apex with cracks. The crack length was about 4 m.

In addition to the cracks it was also noticed that the bolts holding the flanges together had loose fittings. Distances of about 5 mm were measured between the nuts and washers.

The original investigation concluded that the failure was due to the changing climate conditions after insulation was added to the roof. By placing the insulation in between the arches instead of on top of the existing roof, a temperature and thus a moisture gradient built up. The drying of the wood induced the perpendicular to grain failure. Calculations showed that the loading by self-weight resulted in approximately 0.2 MPa perpendicular to grain stresses, which is about the allowable stress. The moisture gradient induced stresses were estimated to be also about 0.2 MPa, and thus the strength of the material was reached. The moisture variation was assumed to be linear over the beam depth, with a difference of 6% MC between the inner and outer parts of the arch.

2.8.4 Additional conclusions and comments

The conclusions from the original investigation seem adequate. The failure can be classified as being due to poor knowledge about moisture induced stresses in wood. This failure shows the importance of accounting for other loads than mechanical ones when designing load-bearing structures.

2.9 Case 9 – Cracks in glulam roof structure

2.9.1 Description of structure

This structure is a glulam roof structure, built in the early or mid 1960s. A schematic of the structure supporting the hipped roof is shown below. The roof beams had support along the walls and in the middle of the building, and at the supports indicated in Figure 11 the beams had notches. The glulam beams marked A and B had cross-sections of 115 by 697 mm^2 the remaining beams were of cross-section of about 190 by 700 to 1000 mm^2.

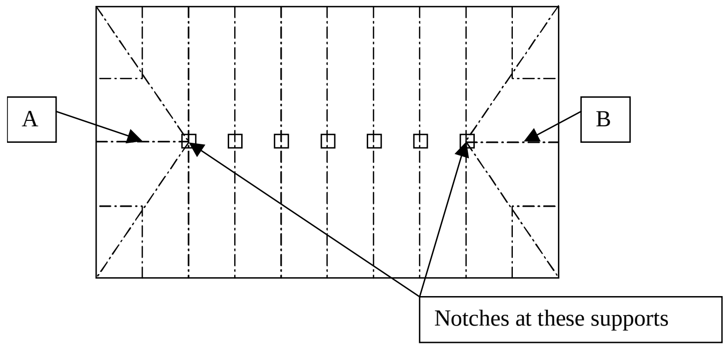

Figure 11. Schematic of roof structure

2.9.2 Description of failure

During winter beam A failed in shear, with a crack originating from the support where a notch had been cut. The crack originated from the mid-depth of the beam. The beam was lifted, a new steel support put in place and the beam was then reinforced with steel holders.

2.9.3 Original investigation performed and conclusions

Approximately one year after the failure the building was inspected by SP-personnel, who conducted a visual inspection, tested bond line strengths from drill cores (both from interlaminar bond lines and from finger-joints. The MC of the beams was measured using a resistance meter and was found to be below 11%. A metal detector was used to look for steel reinforcements, but no indication of such were found.

Several of the beams, apart from beam A that is, were found to be reinforced with steel hangers and in some cases with steel rods through the beams, fastened by nuts on either side of the beams. It is not known exactly when these reinforcements were made, but probably in the early 1970s. At that time, the beams had also been exposed to weather, being unprotected during the work.

The inspection showed that in beam A, the crack did originate from the notch at the support. Also beam B had a crack originating from the notch corner.

A total of 9 of the beams were inspected and were all found to have cracks. In some cases the cracks were severe, leading to delamination at the end of the outermost lamella. Another beam showed a combination of shear and tensile failure close to the support at the wall.

At the reinforcements by rods fastened with nuts it was clear that substantial drying deformations had occurred. The nuts were not tightly fixed to the wood surface.

The conclusions of the initial investigation do not mention anything about a probable cause of failure. It was concluded, however, that the damage was so severe that the beams at the gable should be replaced immediately.

2.9.4 Additional conclusions and comments

It is unclear exactly when all the beams had failed, except for beam A. The cause for the failures is most likely due to over-loading/bad design of the notches at the supports. The drying deformations found indicate that also drying stresses could have worsened the situation.

2.10 Case 10 – Failure in tapered glulam roof beam

2.10.1 Description of structure

The building was erected in 1963-64. The structure is a sports hall with 8 parallel glulam double tapered roof beams. The beam depth at mid span is approximately 1250 mm. About 1,5 m from the support, a 100 mm circular hole has been cut to provide space for tubes.

2.10.2 Description of failure

The investigation performed by SP personnel took place about 10 years after the failure, since the hall was to be renovated. There was a concern that the actions taken at the previous failure had not been enough at also that the beams might have been manufactured using acid-hardening adhesive.

The failure, which was not investigated by SP, was detected since it made it impossible to use some of the gymnastics equipment. One of the roof beams had a crack approximately 200-300 mm from the lower side of the beam, starting from the support and reaching beyond mid-span. The failed beam was replaced, and four of the remaining beams were strengthened with plywood, partially or along the entire beam length. The plywood was screwed and glued to the beam sides.

2.10.3 Original investigation performed and conclusions

At the investigation performed by SP, as mentioned 10 years after the failure, moisture content, cracking, geometrical data, adhesive bond line tests and tests for sulphur content were performed.

The conclusions were that there was no reason to take any further actions relating to the adhesive bond line quality, and that the adhesives used not were based on acid hardening systems.

It was suggested that the holes made at the supports were to be further investigated for possible cracking, since these parts of the beams were not possible to inspect at that time.

2.10.4 Additional conclusions and comments

Data regarding geometry is lacking. However, it is not unlikely that the failure can be due to the holes at the support.

2.11 Case 11 – Cracking in tapered glulam roof beams

2.11.1 Description of structure

The building is a sports hall, erected in 1988. The roof structure consists of 7 double tapered roof beams, approximately 655 mm deep at the supports and 1300 mm deep at mid-span. The beam width was 185 mm. The beams have a span of approximately 20 m, and are placed with a spacing of 5 m. The roof is made from self-supporting corrugated sheet steel.

2.11.2 Description of failure

At a building inspection it was noticed that extensive cracking had occurred in several beams. This lead the inspecting consultant to recommend an extended inspection to be performed by SP.

2.11.3 Original investigation performed and conclusions

The beams are visible from the inside of the building and were inspected by SP personnel. Apart from geometrical data of the beams the moisture content was measured, and found to be in all cases below 10%. The beams were manufactured with a dark coloured adhesive. All beams were inspected for cracking, although not all parts of all beams were inspected. The parts inspected were selected amongst those parts showing the most severe cracking, as was visible from the floor level. This means that the parts not inspected, are likely to have less severe cracking.

The original investigation gives a rather detailed view of the cracks. The conclusions were that, apart drying cracks which are normal in glulam, there were about 10 cracks 1-8 m long and 30-70 mm deep, which could not be explained by drying alone. The most severe crack of these was one starting at the support and reaching about 6-7 m, running all the way in an interlaminar bond line. This crack was found on both sides of the beam, and on one side it had been filled with adhesive at the production. The crack depth was measured to be more than 70 mm from one side and at least 10 mm on the other side (the depth of the repair was 10 mm). In several cracks there was evidence that the adhesive had started to cure too much before appropriate pressure had been applied.

The conclusion was that the severe cracks were due to manufacturing errors (too long open assembly time), and that the shear capacity of the beams should be re-evaluated, taking into account the crack depths and lengths.

2.11.4 Additional conclusions and comments

The conclusions from the original investigation seem adequate. The failure can be classified as being due to manufacturing error, related to a too long open assembly time.

2.12 Case 12 – Failure in beams at steel connections

2.12.1 Description of structure

The roof structure of this library and lecture hall consists of primary load bearing glulam beams of depth between 855 and 1350 mm. The beam width is 215 mm. The beams are connected at supports with a steel connector, schematically shown in Figure 12. The structure was erected during March – April.

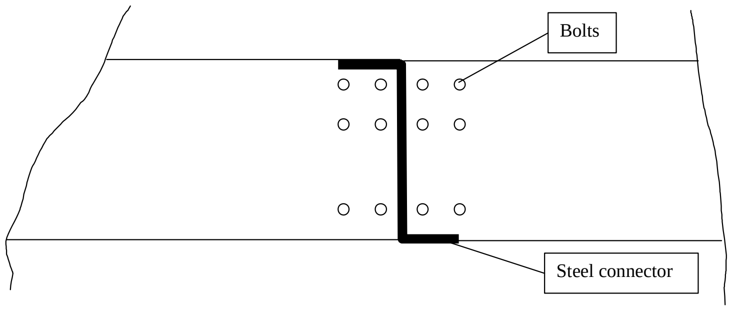

Figure 12. Scematic of beam connection.

2.12.2 Description of failure

Approximately six months after the completion of the roof structure, severe cracking occurred in the connections. Another four months after this, in February, a shear failure in one of the beams occurred. The failed beam had been monitored prior to the shear failure, and it was concluded that its deflection had increased from 37 mm to 121 mm under constant loading conditions, from December to February.

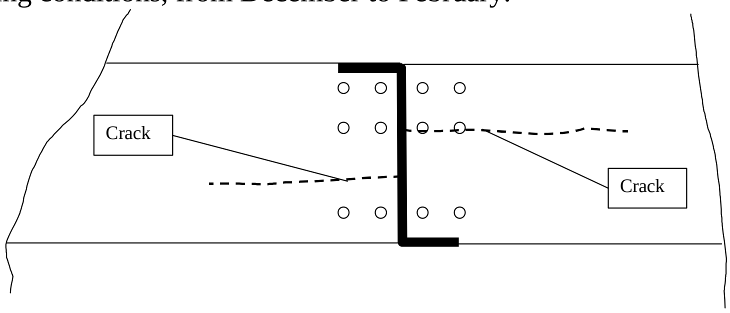

Figure 13. Scematic of beam connection and cracks found.

2.12.3 Original investigation performed and conclusions

The original investigation made by SP personnel included two inspections on site. The inspections included measurements of crack depths, beam geometrical data and moisture contents at different depths.

The moisture content in the beam was found to vary from approximately 9% at the surface to 10% in the inner parts of the beam 530 mm from the connection, and to approximately 12% at beam mid span.

The conclusion of the original investigation was that the failure was due to a poor connection design. The steel parts had been fitted without taking into account possible moisture induced deformations, leading to cracking when the beams dried. The moisture content at delivery to the site was probably in the range of 10-12%, as was confirmed by the protocol from the internal control of the manufacturer. The investigation concluded that the shrinkage was equivalent to an 8% reduction in moisture content (from approx. 18 to 10%). If this is correct, the only possible explanation is poor handling of the beams during construction. Although the beams were delivered with plastic covers, it is still likely that the ends of the beams have been subjected to water at the connections. Being fitted in a state of high moisture content, the subsequent drying lead to the cracks since the connection design does not allow for any deformation.

2.12.4 Additional conclusions and comments

The conclusions from the original investigation seem adequate. The failure can be classified as being due to poor design of the connections, not taking into account the moisture induced deformations of the wood material. The handling of the beams at the building site also played a role here.

It is unclear if the connection design would have caused the same problems if the beams had been handled correctly, and drying from 10-12% moisture content to approximately 7% at the wood surface. Whatever the case, this type of rigid connection should be avoided.

2.13 Case 13 – Cracks in glulam arches

2.13.1 Description of structure

The structure is an arched glulam roof structure. The 36 arches are supported at on the exterior concrete walls of the building with steel hangers. Although visible from the inside of the building, the arches are difficult to inspect. The arches are made from 30-31 mm thick laminations, 187 mm wide. The beam depth varies.

2.13.2 Description of failure

One and a half to two years after construction cracks in the arches were discovered. These lead to the inspection of the building.

2.13.3 Original investigation performed and conclusions

The arches were inspected with regards to geometry, moisture content and cracks. The moisture content of the arches was 11-12% at the surface and 12-13% 30 mm below the surface. Six arches were inspected. Cracks with lengths up to almost half the span were found. The description of the structure, its steel hangers and crack locations is difficult to follow. However, it seems like all cracks investigated had started from the supporting hangers or close to these.

The investigation concluded that several cracks were due to weakened bond lines. The bond lines were probably weakened due to premature hardening of the adhesive and/or due to insufficient pressure during curing.

2.13.4 Additional conclusions and comments

The conclusions from the original investigation seem adequate. The failure can be classified as being due to weakened bond lines in the glulam emanating from production errors, such as premature hardening of the adhesive and inadequate pressure.

Since the exact design of the structure and especially the design of the hangers is unknown it is difficult to draw any other conclusions. One could suspect that if the arches were rigidly attached to the hangers, the cracking could be due to restrained shrinking of the glulam.

2.14 Case 14 – Cracks in glulam beams and columns

2.14.1 Description of structure

The structure is the main load bearing part of a sports hall. It was erected in 1985-86. The roof structure consists of 6 straight glulam beams with rectangular cross-section, supported by rectangular glulam columns. The beam cross-section was 140 by 450 mm and the columns were 140 by 405 mm. The columns supporting the beams were of different height, resulting in a roof pitch of 1:4. The beams were slightly tapered at the lower support, reducing their depth. At the upper support there was a notch cut, approximately 100 mm deep.

2.14.2 Description of failure

No information is available regarding the reason for the inspection, but obviously there was a general concern of the integrity of the structure, which had numerous cracks.

2.14.3 Original investigation performed and conclusions

The inspection included geometrical data, moisture content measurements and measurements of the cracks.

The beams had some cracks in both the bond lines and in the laminations. The length of the cracks was approximately 1,5 m, the cracks depths were not measured.

The columns had rather severe cracking, on all sides. Crack depths of up to 71 mm were measured. Traces of surface treatment (paint or lacquer) were found in most of the cracks.

The connection of the wind stabilising structure (probably steel rods) showed a gap between bolt and nut of 4 mm.

At the upper beam support (the one with the notch) a gap of 2-4 mm was found between the column and the beam. There were no signs of crushing of the wood material perpendicular to the grain at the supports. In 5 of the beams cracks were found. The cracks started from the notch and ran in the direction of the notch (along the beam axis). The largest notch crack was found to be 740 mm long with a depth of up to 75 mm.

It was concluded that the cracks in the columns were due to moisture-induced deformations. The gap between bolt and nut was taken as evidence for this. Consequently, it was concluded that the moisture content in the columns had been as high as 15% at construction. These cracks were not judged as being likely to cause any structural degradation. It was recommended that the cracking of the columns should be monitored in the future.

It was furthermore concluded that the cracks in the beams with notches was due to stress concentrations in the notch. It was recommended to perform new design calculations for the notches and possibly to reinforce them using glued-in rods, to avoid that the cracks would grow further.

2.14.4 Additional conclusions and comments

Although difficult to make a detailed analysis of this case, due to lack of information, the conclusions of the original investigation seem adequate. The failure was due to poor design of the notched support. The cracks in the columns are not to be regarded as a failure, since these were not so severe so as to cause any structural degradation.

2.15 Case 15 – Delamination in glulam beams

2.15.1 Description of structure

This structure is a roof structure of an ice-hall. The structure is a three-hinged roof truss with a tensile rod as depicted in Figure 14. A total of 11 roof trusses at a spacing of 6 m make up a total of 60 metres length of the building. The building was erected in 1964.

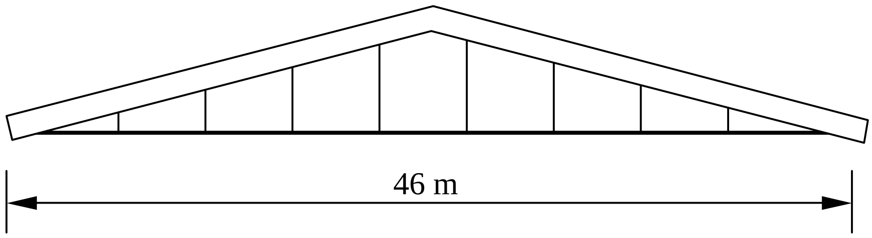

Figure 14. Roof structure.

Each of the 190 mm wide roof beams was made up from two laminations, edgewise glued to each other. The two laminations had widths of 45 and 145 mm respectively. This is shown in Figure 15.

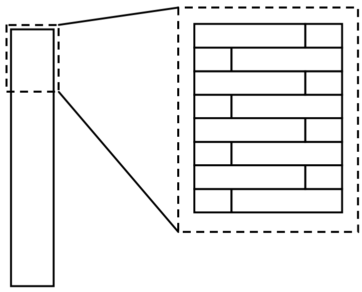

Figure 15. Beam cross-section built up from edgewise glued laminations.

2.15.2 Description of failure

After two summers of severe moisture conditions in the hall, extensive delamination was seen in three of the trusses. Being built in 1964, the structure had been inspected for the possible occurrence of acid hardening adhesives. No such adhesive could be detected.

2.15.3 Original investigation performed and conclusions

The beams were inspected on site and were photographed. The delamination depth was measured and drilling core samples were taken for shear testing and chemical analysis.

The original investigation concluded that the two beams that had been damaged were different from the others. During the inspection information was provided indicating that a failure had occurred during the erection of the building. This would explain that two of the trusses were different.

The failure was concluded to be due to the moist conditions, which had lead to the severe delamination. It was concluded that the laminations had probably not been edgewise glued together and planed before forming the beam cross-section. Instead it is likely that the laminations had been put in place all at once, to form the cross-section. This means that even a very small difference in lamination thickness between the 45 and 145 mm laminations will result in the gluing pressure only being adequate parts of the interlaminar bond line.

It was concluded that the beams had not been glued with acid hardening adhesive.

The delamination had lead to a reduction of the active beam cross-section to about half the original. As a result it was recommended that these beams be immediately reinforced.

2.15.4 Additional conclusions and comments

The conclusions from the original investigation seem adequate. The failure can be classified as being due to weakened bond lines in the glulam emanating from production errors, leading to an inadequate gluing pressure.

2.16 Case 16 – Collapse of roof structure in a school

2.16.1 Description of structure

The roof structure of the school building was made from single tapered beams, made from solid wood flanges glued to a plywood web. The flanges were glued-nailed to the web using a phenol or possibly a phenol-resorcinol adhesive. The flanges consisted of two timber members, one glued to each side of the web. The web had joints, which were reinforced by vertical wood members, also glued-nailed to the web. The flanges were also lengthwise joined by glued-nailed, overlapping wood members. The flange joint was located at the same section for both timber members in the flange. The roof beam is schematically depicted in Figure 16. The span of the roof beams was approximately 7,9 m.

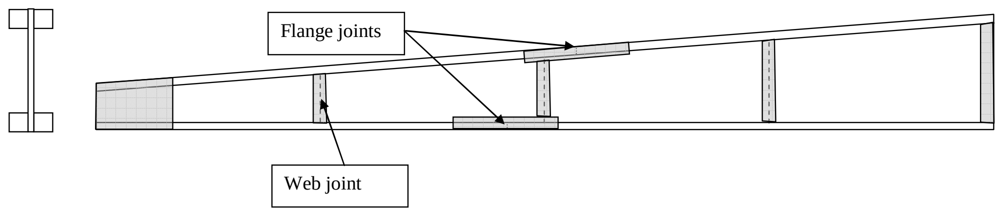

Figure 16. Beam with joint reinforcements.

2.16.2 Description of failure

In early spring, fortunately on a Sunday afternoon, all nine roof beams collapsed. The failure was complete and all beams had fallen down, failing at mid-span.

At the collapse, the snow load on the roof was high. The snow depth was measured the same day and was found to be between 56 and 65 cm. The snow density was also measured this day, and was found to be approximately 300 kg/m^3, and thus the snow load was equivalent to about 1,8 kN/m^2.

2.16.3 Original investigation performed and conclusions

Approximately two weeks after the failure, SP personnel performed an on-site inspection of the building. Apart from obtaining geometrical data of the beams, the failure modes and the quality of the bond lines were investigated.

The conclusions of the investigation performed were that the failure had started as a tensile failure in the lower flanges. This in turn was initiated due to the overlap joints of the flanges being of poor quality. It turned out that the production of the beams was done by applying adhesive on only one surface, and that no pressure was applied apart from the pressure from the nails. Using only the nails for obtaining glue pressure is surely not enough, since large areas in between nails do not experience any considerable pressure.

As an example of the poor overlap joints, in one beam the active bond line area was estimated to be only about 10% of the theoretical one. In other beams the bond line between the flange and the web was also poor, with up to one metre long parts without any adhesive on one part of the joint.

2.16.4 Additional conclusions and comments

The conclusions from the original investigation seem adequate. The failure can be classified as being due to poor production methods when manufacturing the beams.

2.17 Case 17 – Buckling of roof structure

2.17.1 Description of structure

A large number of similar structures were built in Sweden for a chain of stores. The structure is based on nail-plate connected roof trusses. The battens, for support of the roof tiles, are nailed directly to the upper chord of the roof trusses. The roof structure also includes a reinforced plastic sheeting or a hard fibre board, covering the roof trusses, for draining of rain water. At least 26 cases have been documented, were the same principle was used for the load bearing structure.The buildings are relatively large, with free spans of 20-25 m and lengths of up to 65-70 m.

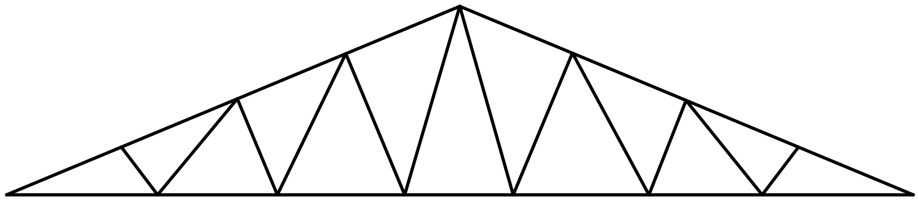

Figure 17. Exemple of roof truss

Sometimes, the trusses have been manufactured in two parts, for transportability, see Figure 18

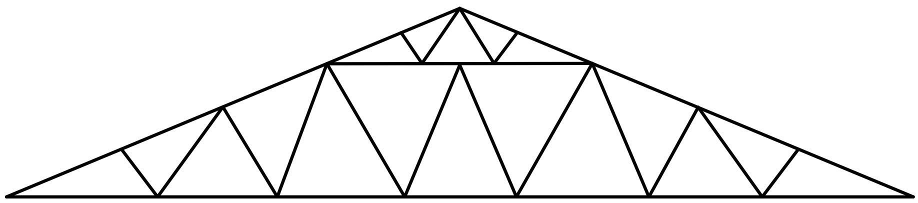

Figure 18. Truss divided in two parts for transportability.

2.17.2 Description of failure

In at least four cases similar failures have been observed (several failures have been reported, including one in Norway and one in the Czech republic). The failure was due to lateral displacements in the compressed parts of the truss, deformations were measured in one case to be as large as 400-500 mm, see Figure 19.

The failure led to the battens falling down in between the roof trusses in several cases. The battens were often joined at the same truss, leading to a more flexible structure with larger deformations. It is noticeable that there was no snow on the roof when failure occurred.

The risk of total collapse of the buildings was estimated as being very high in some of the cases.

Figure 19. Example of deformations at failure.

2.17.3 Original investigation performed and conclusions

Several investigations have been performed on the current buildings, the first in 2003. Although the problem was known at that time, new buildings were constructed the following years. In 2005 SP wrote a formal letter of warning to Boverket. This resulted in a commission to perform an overall investigation of all known failure cases and the reasons for these. Also a more general investigation of how this large amount of buildings with potentially unsafe design could have been erected without any reactions from building authorities.

The primary cause of failure is that the compressed parts of the trusses have been lacking lateral constraints to avoid buckling. The nailed battens do not provide enough rigid connections to avoid buckling due to initial deformations. As an example, consider the design rules of EC5, were an edge distance of 15d to end grain and 7d otherwise is required. From it is then clear that the truss should have a thickness of 44d (=190 mm for 4,3 mm nails), which should be compared with the actual width of 45 mm (in some cases 70 mm width was used for the truss. In conclusion, there is no possibility of transferring large tensile forces in the battens in order to stabilise the trusses.

An important question is how this large amount of buildings could have been erected with such poor designs. The investigation performed at SP concluded that:

- There exists a lack of knowledge among Swedish timber engineers relating to basic design of timber structures
- The education of timber engineers has been neglected
- There is a need for *complete* design examples for Swedish engineers. The Swedish code, BKR, contains only partial descriptions. Complete examples exist in a Swedish translation of the ENV-version of EC5, but this is probably not well-known by practising engineers, since the use of EC5 is not well established.
- The building process is highly diversified, with a large number of subcontractors taking responsibility only of their part. Thus, the stability of the trusses tends to fall in between different areas of responsibility
- Price is more driving than quality

The internal control, which is supposed to stop this kind of poor designs is not working properly.

2.17.4 Additional conclusions and comments

The conclusions from the original investigation seem adequate. The failure can be classified as being due to poor design.

2.18 Case 18 – Cracking in glulam frame

2.18.1 Description of structure

This structure is a tennis hall built from 3-hinged glulam frames, placed at 9 m distance. The columns are curved, and attached to the ground by steel parts, see Figure 20.

Figure 20. Arched column with steel parts (in white).

2.18.2 Description of failure

One of the columns showed a large crack, which stopped at a length of 2-2,5 m. The crack started from the steel part, where a notch had been made. The hall was closed pending an investigation and proper actions to be taken.

Figure 21. Crack at column fitting to the steel part..

2.18.3 Original investigation performed and conclusions

The day after the cracking, representatives from the glulam manufacturer visited the hall. From the inspection it was concluded that:

- The amount of snow on the roof was less than 200 mm.
- No deformations nor misalignments were visible.
- The crack was approximately 10 mm wide and 2-2,5 m long. The crack had gone through the complete beam width. The crack path crosses thre laminations.
- The crack starts at a notch made for the steel part added to transfer horizontal loads.
- The steel part is not according to the original drawings. The steel part was placed about 100 mm from its original position, and a notch making this possible has been cut.
- There is evidence of leakage from the roof (miscouloring of the glulam). Thus, a possible moistening of the timber followed by drying could be the reason for crack formation.

To assure that no further cracking would occur, and as a temporary remedy, steel fixings were placed on the column, at the crack end.

Figure 22. Temporary steel fixings attached to the column.

2.18.4 Additional conclusions and comments

The conclusions from the original investigation seem adequate. The failure can be classified as being due to alterations on site and due to poor maintenance of the building.

3 Discussion and concluding remarks

3.1 Failure categories

Below is an attempt to summarise the failure cases reported herein, and to categorise by the primary failure reasons. The following failure cases are used:

1. Wood material performance:
   By this is meant that the materials used in the product have been of poor quality in relation to practice. An example is larger knots than permitted in glulam laminations.
2. Manufacturing errors in factory:
   This relates to manufacturing errors in the factory, errors which should have been detected at production according to practice and internal quality control.
3. Poor manufacturing principles:
   This means that the basic principle used for manufacturing the product has was poor. However, the poor principle has been used as intended.
4. On site alterations:
   Here alterations of the structure, has been made on site. These alterations have led to the failure. Note that it is often difficult to know whether these alterations were intended from start, and made on site for practical reasons.
5. Poor design/lack of design, mechanical loading:
   This means that the failure was due to the strength design of the structure was poor. In this category only mechanical loading is considered.
6. Poor design/lack of design environmental loads:
   This means that the failure was due to the strength design of the structure was poor. In this category only failures due to mechanical loading in combination with environmental loading (drying cracks) are considered.
7. Poor principles during erection:
   Failures, which are due to poor handling at the erection of the structure are grouped in this category.
8. Overload in relation to building regulations
9. Other/unkown reasons

3.2 Quantification of failure cases

Below is shown an attempt to quantify the failure reasons. For each case, the failure reasons are indicated by indices, ranging from 0-1. The sum of the indices for each case equals one. The importance of the different failure cases is calculated both by the number of failures belonging to each cause, and by calculating the average index.

Case number	**Material**	**Failure reasons[1] and index**								
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
1	Glulam				0,5	0,5				
2	Glulam		1,0							
3	Glulam		0,2			0,8				
4	Glulam			1,0						
5	Glulam								1,0	
6	Glulam			0,4		0,4		0,2		
7	Glulam							1,0		
8	Glulam						1,0			
9	Glulam					0,8	0,2			
10	Glulam					1,0				
11	Glulam		1,0							
12	Glulam						0,8	0,2		
13	Glulam		0,8				0,2			
14	Glulam					1,0				
15	Glulam			1,0						
16	Composite			1,0						
17	Timber					1,0				
18	Glulam				0.8					0.2
No. of failures		0	4	4	2	7	4	3	1	1
Average index.		0	0,17	0,19	0,07	0,30	0,12	0,08	0,06	0,01

Thus the failure causes in this investigation, with any of the above principles for quantification, can be ranked as follows

- Poor design/lack of design
- Poor manufacturing principles or manufacturing errors in factory
- Poor principles during erection and on-site alterations
- Overload in relation to building regulations

[1] 1.Wood material performance, 2.Manufacturing errors in factory, 3.Poor manufacturing principles, 4.On site alterations, 5.Poor design/lack of design, mechanical loading, 6.Poor design/lack of design environmental loads, 7.Poor principles during erection, 8.Overload in relation to building regulations, 9.Other/unknown reasons

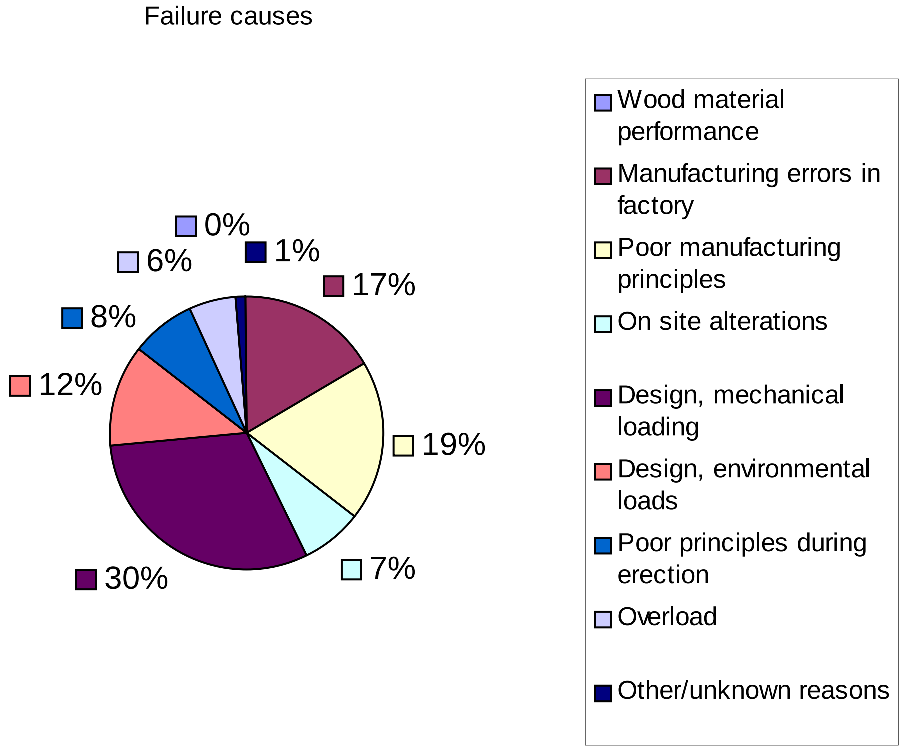

Figure 23. Frequency of failure causes, based on average failure index.

3.3 Concluding remarks

A number of failures in structures have been reviewed. The cases included are such cases where SP has been commissioned to perform an investigation in order to clarify the reasons for failure. Only cases involving structural damage (collapse, crack formation, excessive deformation etc) are reported, and thus, for example, moisture induced damage resulting in mould growth has not been included. A total of 18 cases from the period of 1978 – 2005 have been found in the archives.

From the cases studied it can be concluded that the most common reason for failure is the lack of design or poor design in relation to mechanical loading. This was the primary cause of failure for nine (50%) of the cases. In six out of these nine cases did the structure include notches or holes, at which the high stresses perpendicular to the grain were the main cause of failure. In the remaining two cases the failure was also due to fracture perpendicular to the grain, although induced, not by mechanical loading, but by moisture variations.

In six of the eighteen cases (33%) it was concluded that the failure was primarily caused by errors done in the manufacturing of the product (mainly glulam). The most common type of error is related to the gluing process and includes errors such as the use of too small amount of hardener and inadequate gluing pressure.

The remaining failures were due to poor principles during erection of the building and due to the snow load exceeding the code value, respectively.

The main conclusion from the investigation is that in none of the cases was the wood raw material the cause for failure. Instead, all failures were either due to the lack of knowledge (lack of design knowledge, lack of knowledge about sound production methods) or due to poor quality control procedures. As a consequence, improving the knowledge within the areas of solid and structural mechanics among timber engineers as well as within the areas of quality assurance among producers and contractors is of utmost importance. In no cases did the wood material performance have any influence on the failures.