Glulam Trusses for Olympic Arenas, Norway

Erik Aasheim
Civil Eng., Norwegian Inst. of Wood Technology, Oslo, Norway

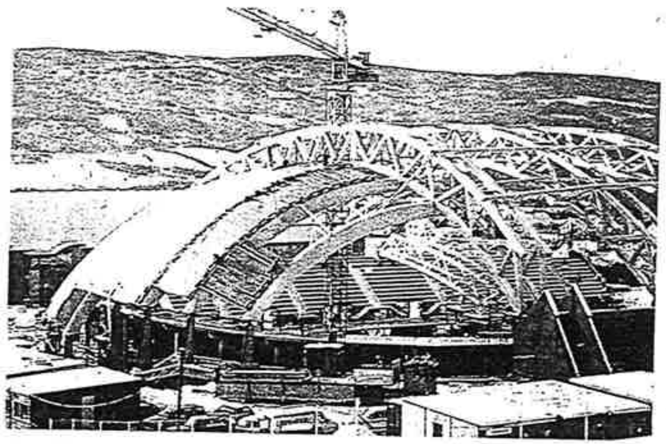

Fig. 1: Hakon's Indoor Stadium during erection

Introduction

In 1988, the city of Lillehammer, Norway, was chosen to host the 1994 Winter Olympics. In order to meet the needs of this event, several large halls had to be built, including facilities for ice hockey, figure skating and speed skating. A basic aim of the organisers was an environmentally friendly Olympics, with buildings that emphasised natural Norwegian wood products.

The Norwegian wood products industry, its engineers and producers, saw the Olympics as a unique opportunity to demonstrate the possibilities of timber construction techniques. At an early stage, the glued-laminated (glulam) truss was chosen as the principal load-carrying element for the new structures because of the following design and fabrication requirements that it satisfies:

- efficient use of a locally available raw material
- great flexibility in design
- simple prefabrication of elements
- light weight.

An intensive glulam truss development programme was undertaken with the following goals:

- capacity for clear spans of up to 120 m
- fire resistance of at least 60 minutes
- easy transport of structural elements
- quick and safe assembly and erection
- inspiring architectural and technical designs
- costs competitive with steel, concrete and aluminium structures.

The structures for the 1994 Lillehammer Olympic Games will provide a strong demonstration of the advantages of glued-laminated timber trusses in structures with large clear spans. Glulam trusses allow for interesting architectural design and create structures that are resource- and environment-friendly.

Structural System

A particular challenge was to develop a jointing system appropriate for the large members and large forces foreseen in the new buildings. A system of steel dowels and slotted-in steel plates was developed from a variety of tests and studies. The system utilises 12 mm dowels of high grade steel combined with 8 mm steel plates. Precision slots were sawn by specially developed equipment using a computer-controlled circular saw with the blade in a horizontal position.

Hakon's Indoor Stadium

This hall will be the main ice hockey arena during the Olympics, with a capacity of 10 000 spectators. The main structure consists of double arch glulam trusses with a maximum span of 85.8 m *(Fig. 1)*.

The stadium has an indoor length of 127 m and is, with its amphitheatre-like grandstands, an intimate arena despite its size. The gable structures are formed by curved glulam beams suspended from the main trusses. 1300 m³ of glulam have been used in total in this structure.

Architect:
Ostgaard Arkitekter A.S.

Contractor:
A.S. Veidekke

Glulam producer:
Moelven Limtre

Service date:
1993

Hamar Olympic Amphitheatre

This is the main arena for figure skating and short track speed skating, with a capacity of 6000 seats. The roof is supported by trussed girders with a maximum span of 70.8 m and a spacing of 13.2 m. The girders are simply supported on concrete columns.

The straight bottom line of the trusses places an extraordinary tension force of about 7500 kN on the splices of the 570 × 560 mm bottom chord. Transfer of this force required joints with 11 slotted-in steel plates and 70 dowels.

The structures were prefabricated in three sections at the factory and assembled at the building site *(Fig. 2)*. A total of 750 m³ of glulam wa d. The outer walls of the amphith e were constructed of glulam columns and prefabricated wall elements of timber and glass. The internal surface consists of fire-resistant impregnated timber cladding.

Architect:
HRTB A.S., Hovde Arkitekter

Glulam producer:
Moelven Limtre

Contractor:
Martin M. Bakken A.S.

Service date:
1993

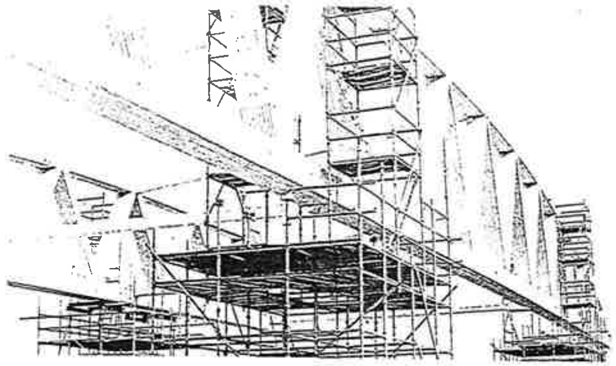

Fig. 2: Main girder Hamar Olympic Amphitheatre

Hamar Olympic Indoor Stadium
The Viking Ship

This indoor stadium has, with its unusual architectural form of a upturned Viking ship, already become a regional landmark. *Fig. 3* shows the "ship" during erection. The Olympic speed skating venue, the Hamar Stadium is a multi-purpose building and will also house the 1993 world cycling championships. Other options for utilisation include indoor football and concerts. Making use of the floor area can raise the capacity from 10 000 to 20 000 seats.

The main structure consists of arched trusses with spans varying between 30.0 m and 96.4 m, spaced at 12 m centres *(Fig. 4)*. The roof construction consists of corrugated steel sheets with insulation and a roofing felt covering, supported by glulam purlins. The stadium has an inside length of 260 m, a width of 96 m and a highest point above the floor of 35 m. The total glulam consumption was 2000 cubic metres, and 40 000 dowels were used in the connections.

Architect:
Biong & Biong/Niels Torp A.S.

Contractor:
Ole K. Karlsen A.S.

Glulam producer:
Moelven Limtre

Service date:
1993

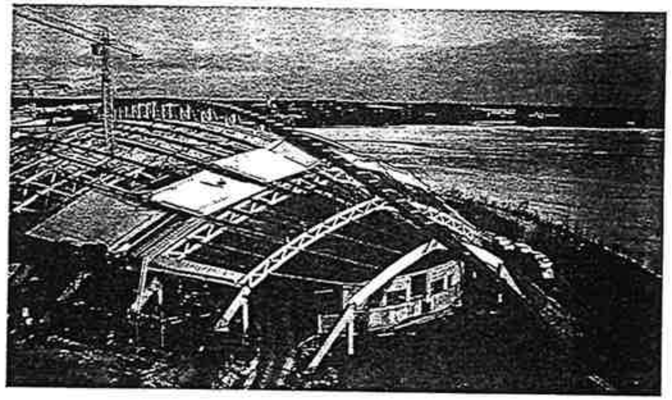

Fig. 3: Hamar Olympic Indoor Stadium during erection

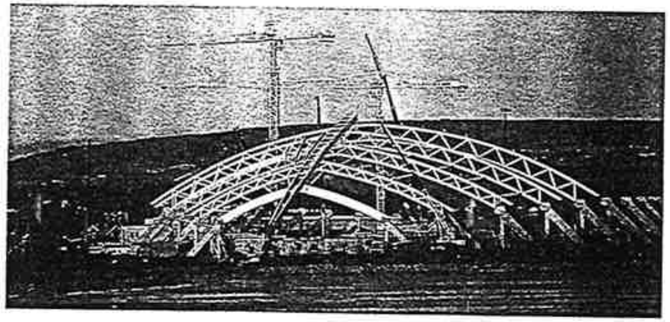

Fig. 4: Arched glulam trusses

Experiences

Accuracy was paramount throughout the construction of these facilities and the results have been very positive. The dowel and steel plate connection system has proven to be a good one. No problems with driving the dowels through the holes were encountered, even at the building sites.

As for the future, glulam designers see no reason why arched glulam trusses of up to 130 m should not be possible and competitive. This type of structural timber system might also be applied to other structures, for example, to road bridges.

THE USE OF WOOD AND GLULAM IN THE XVII OLYMPIC WINTER GAMES

Challenges in production and assembly

Introduction

When it was decided in September 1988 that Lillehammer was to host the XVII Olympic Winter Games in 1994 we knew that several large sports halls were to be built, halls for ice hockey, figure skating and speed skating. And we knew that we had no competitive solution for such large halls. We also knew that the media would pay special attention to these projects and that the use of glulam in these halls would have great impact on the future use of glulam as load-bearing system in large structures.

On this background we decided that at least one large sports hall was to be built with glulam as the main load-bearing system. At this time we also knew that some halls had to have load-bearing structures with a free span of approx. 100 m, which was well above what we had supplied earlier.

For Moelven Limtre A.S, the supply of glulam structures for halls with large spans had been limited by production and transport such as:

- max. length of laminations approx. 37 m
- max. cross-section thickness approx. 300 mm
- max. cross-section height approx. 2000 mm
- problems of transporting both large lengths and large widths

In addition to the production and transport aspects, financial matters have resulted in the fact that the type of structures shown in figure 1 have been most relevant for the use of glulam in load-bearing systems. The figure also indicates the most common span ranges for the structures. The type of structure that may cover the largest span is the three-hinged arch which in principle is only limited by how large the size of each half is allowed to be. The figure indicates a limit of approx. 80 m, which means a lamination length of 42-43 m. For Moelven Limtre A.S with a maximum lamination length of 37 m, the largest span will be in the range of 65-70 m depending on the transport possibilities.

The requirements

Our aim was to supply load-bearing systems for the sports halls for the Winter Olympics in 1994 and we decided that the following requirements had to be met:

- Free span up to 120 m;
- Fire resistance of 60 minutes;
- Simple transport and assembly;
- Competitive in price in relation to steel, concrete and aluminium load-bearing systems;
- To show that the arch could be used although the requirements for good design were strict.

In the following I will demonstrate how the various requirements were met.

The reason for this last requirement is that just before September 1988 Lillehammer built a new ishockey hall, called Kristins Hall. The load-bearing system for this hall is a (three-hinged) arch, and many architects said that it was awful (ugly) because of the arch, and that they did not want such constructions for the Olympic Halls.

Why use the arch?

Constructors have always known that the arch is definitely the most favourable for use in structures with large spans. Figure 2 illustrates this fact.

A straight glulam beam with dimensions 190 x 1200 mm and normal roof loads will have a free span of approx. 17 m. If we take the same roof loads and the same cross-section and give the beam a curvature to make a three-hinged arch, the arch may now have a free span of approx. 50 m, or approx. three times as much as a simple beam.

A simple beam of 50 m (if it had been possible to produce such a beam) would have had a cross-section of at least 300 x 3000 mm. It would have had a volume of glulam almost four times as big as the arch and would consequently be considerably more expensive and could not compete with steel systems.

This proves that the arch is the most cost-effective type of structure for construction of halls with large, free spans.

For glulam, the arch is particularly favourable because the cost of producing large curved glulam beams is practically the same as for the production of straight beams. For other construction materials, curved elements will mean considerable extra costs.

I leave the question of whether the arch can be combined with good design to you, who have visited the Viking Ship today and who will be visiting the Håkon Hall tomorrow evening.

The solution

For the large spans and loads as in this case, it was natural to consider a triangulated truss in addition to the arch. Some decisive factors for this decision were:

- The wood is correctly located to give the optimal effect;
- It may be prefabricated in parts;
- Light structure which is favourable with regard to production, transport and assembly;
- Great flexibility with regard to structural configuration.

The greatest challenge was to find a connector capable of transferring the great forces existing in a triangulated truss with a span of up to 120 m. In Norway, the most common connectors were bolts and Bulldogs. These joints require a large area to transfer the forces, and the exterior steel parts did not meet the architectural requirements.

We had seen that the development in Europe was to increase the use of small diameter dowels and slotted steel plates and we decided to develop our own system for the triangulated trusses for the large Olympic sports halls.

Eventually, we chose a solution which involved 8 mm steel plates that were slotted into wood and anchored with 12 mm steel dowels of medium steel quality.

Figure 3 shows how a joint between the chord and two diagonals has been made. This example shows two slotted steel plates, but the number may vary depending on the size of the forces and the cross-section. The optimal distance between the steel plates will be approx. 70 mm, which means that both the compressive strength of the wood and the strength of the dowels are fully exploited. By reducing the distance between the steel plates the stiffness of the connection is considerably increased.

The first project in which we had the chance to use the combination arch - triangulated truss -joint solution was in the Hamar Olympic Hall (the Viking Ship) which you visited today and which you see on this picture. Figure 4 shows a section in the middle of the hall where the arch has the largest span. The free span between the arch footings is 96,4 m and the height of the arch is approx. 14 m. The arch is a three-hinged arch - hinges in both the arch footings and on the lower chord at the apex. The joint in the upper chord cannot transfer forces in the longitudinal direction of the chord. This joint would have been very simple to make, thereby making a two-hinged arch. However, we wanted a three-hinged arch because:

- The design load actions are nearly the same for a two-hinged arch as for a three-hinged arch;

- We wanted to avoid the additional stresses due to small movements of the arch footings of the two-hinged arch. The foundations are supported by 25-30 m long concrete poles on rock, and the height from the top of the pole to the arch footing is approx. 13 m. A horizontal force of 5000 kN in the arch footing will quickly cause deformations, making the two-hinged arch less favourable than the statically determined three-hinged arch.

Furthermore, we wanted the arch footings to be located on the upper chord because most of the compressive force would then have been in the upper chord which is laterally braced in the roof. However, the contractor did not want to move the horizontal force further up than necessary. This resulted in considerable compressive forces in the lower chord, which therefore had to be laterally braced.

Documentation of strength

Having demonstrated the result of the development project I will return to the requirements we decided to meet, and the documentation of the joint strength was the first working item. Theoretical calculations and assessments were carried out, but in order to control ourselves and to be able to sell these structures to sceptical clients/contractors, we decided to carry out a full-scale load test of a joint as shown in figure 3. The test was carried out at the Swedish National Testing and Research Institute in Borås in Sweden, and is shown in figure 4. The same tension and compressive forces were applied in the two diagonals, giving a pure compressive force in the chord. At collapse the diagonals had a force of 500 kN which was in agreement with the theoretically calculated collapse values. We were therefore sure that the connection was safe for use in the Viking Ship.

However, new challenges turned up.

At Hamar, a figure skating hall was also to be built. The load-bearing system for this hall is shown in figure 5, and we were asked to offer a glulam structure. A quick calculation proved that the lower chord had to be jointed for a tensile force of 7000 kN, and we said straight away that it was not possible. After a few days however we started to construct the splice and decided that it was possible after all. New loading tests were necessary. The actual triangulated truss structure consists of three elements (total width 780 mm) and only a third of the structure was tested each time.

Figure 6 shows one of the splices in the big tension test machine at the Swedish National Testing and Research Institute. This connection had a short time strength of 3350 kN. Figure 7 shows the load and deformation in the connection. It is shown that for service loads, the deformation in the connection is less than 1 mm and for collapse 2-3 mm. This is a very rigid connection, which is mainly due to the fact that the wood has a thickness of only 63 mm between the steel plates.

Having carried out these tests we felt sure about our jointing technique and figure 8 shows the final structure during assembly.

Fire resistance time

We now had a joint capable of handling the great loads but one crucial point remained unanswered - did the system had the sufficient fire resistance? In Norwegian sports halls of this type, it is required that the load-bearing system has a fire resistance of 60 min. The good fire characteristics of glulam are known and well documentated. With the large cross-sections of the structure the fire resistance of the actual glulam would by far exceed the requirement of 60 min. The question was how the joint would resist to fire. From other tests we knew that the steel plates slotted into the glulam with sufficient cover, are exposed only to modest heating before the wood on the outside is combusted. We therefore concentrated on registrating the temperature development in the dowels. These tests were carried out at SINTEF (The Foundation for Scientific and Industrial Research at the Norwegian Institute of Technology) in Trondheim and figure 9 shows how a test piece appeared after having been exposed to 60 min. standard fire. An average of 45 mm was combusted, which corresponds well with the new CEN rules.

Based on the knowledge on the heating process, we reckoned that the distance from the dowels to the fire-exposed surface would have great consequence on the temperature development in the dowels. We therefore assembled dowels with various distances to the fire-exposed surface.

Figure 10 shows the temperature development at a specific location (56 mm from the exposed surface) for a dowel at 5 mm distance from the exposed surface and for a dowel at 25 mm distance from the exposed surface. The diagram indicate that when the dowels end 25 mm from the exposed surface, the temperature is 200° C after 60 minutes of fire and more than 100° C lower than if the dowels end 5 mm from the exposed surface. This difference in temperature will be significant for the load-bearing capacity of the connection.

Production

The greatest challenge during production was the lack of time, because the structures had to be delivered within a very short time. The technical challenge was primarly how the slotting of wood and how the drilling should be carried out. For the slotting it was necessary to develope a separate equipment capable of slotting to the depth and the accuracy we required, and with the necessary working capacity. The solution was a computer controlled saw which you will see in the video that we will show you in a moment.

As for the drilling, the steel plates are predrilled. The requirements of such dowel connections are that the dimension of the hole in the steel plate shall not be more than 1 mm larger than the diameter of the dowel. For a dowel with diameter 12 mm this means a hole diameter of maximum13 mm. After a few drilling tests we found it unnecessary to have such wide tolerances and therefore we use a hole diameter of the steel plates of 12,5 mm, which means a better and more rigid connection.

As to the final erection the structure was preassembled and predrilled such that we were completely sure that everything fitted together, as shown in figure 11 showing one of the triangulated trusses in the Hamar Olympic Amphitheatre. This figure also illustrates the size of the structure, because, I can assure you, that the man standing at the end of the beam is of normal size.

We do not wish to reveal more of the production process details, because the techniques we have developed are the advantage we have compare to our competing companies wanting to offer the same product.

Assembly

The assembly of the huge arch triangulated trusses was carried out in a way that you will be able to see in the video we will show you and as illustrated on figure 12. Each half consists of two parts. the footing is constructed such that it is capable of handling great rotations, and one of the halfs is placed in the footing and on a support below the splice. The height of the support is such that when the next part is fitted the top of the arch will be at floor level. When the dowels in the joints have been fitted the two halfs are hoisted by means of four mobile cranes and the upper end is connected. The structure is braced with wires and wind bracings members are assembled.

As an illustration of the challenges we encountered during the assembly I will show you a photo taken from the assembly of the Håkons Hall where, in addition to our glulam structures, there were six mobile cranes and two fixed building site cranes in an area just a little bit larger than an ice hockey rink.