1.Foreword

This handbook is a result of the Nordic Wood project   the Nordic Glulam Handbook. Nordic Wood is the name of the research and development programme of the Nordic timber industries, aimed to strengthen the competitiveness of timber. Nordic Wood was initiated by the Nordic Industrial Fund and the programme was carried out during the period 1993-2000 with a budget of 225 million Norwegian kroner. The programme was financed by the Nordic timber industry, the Nordic Industrial Fund and the national R&D organizations   Skov- og Naturstyrelse in Danmark, TEKES in Finland, Islands Forskningsråd, Norges Forskningsråd and NUTEK in Sweden.

In the project Nordisk Limträhandbok, P99024, the following companies have taken part: Finland: The Finnish Glulam Association/ WoodFocus Finland, Vierumäen Teollisuus Oy, Kuningaspalkki Oy, PRT-Wood Oy, Kestopalkki Oy, Jetlink Oy; Norway: Moelven Limtre AS; Sweden: The Swedish Glulam Manufacturers Association/ Svenskt Limträ, Långshytte Limträ AB, Martinsons Trä AB and Moelven Töreboda Limträ AB.

The main object of the handbook has been to produce a handbook which is easy to update, distribute and use. In order to achieve this, the handbook is available both in printed and electronic form. The handbook exists in four language versions   English, Finnish, Norwegian and Swedish. The contents in the different versions are adapted both to Eurocode 5 and the associated NAD (National Application Document), and to the timber building regulations in the country concerned.

The project work has been financed 50% by the Nordic Industrial Fund and the three countries' R&D organizations TEKES, NFR and NUTEK and 50% by the co-operating glulam manufacturers in Finland, Norway and Sweden.

The steering group has consisted of:

Lennart Axelsson, Svenskt Limträ AB and Moelven Töreboda Limträ AB; Petri Heino, Finnish Glulam Association/Wood Focus Finland; Åge Holmestad, Moelven Limtre AS.

The working group has consisted of:

Olle Carling, Olle Carling Ingenjörsbyrå AB; Holger Gross, Gross Production AB and Svenskt Limträ AB; Veijo Lehtonen, Late Rakenteet Oy; Åge Holmestad, Moelven Limtre AS; Rune Karlsson, Långshytte Limträ AB; Tomas Sundström, Consultec Byggteknik AB.

The handbook describes certified glulam, deals with important precautions and gives advice on the design of glulam structures. It also shows some of the numerous applications where glulam structures are used today. The chief author has been Olle Carling. A reference group consisting of user representatives from each country has taken part in various stages. The members of the group have been Mika Leivo, Mikko Viljakainen, Jussi Vepsäläinen, Finland; Sverre Wiborg, Norway and Dan Engström, Sweden. Parts of the contents have been kindly placed at the disposal of the project by Håkan Persson, Tyréns Byggkonsult AB and Martin Gustafsson, AB Trätek. The translation into English has been made by James Codrington, Transark. The Finnish translation and adaptation has been made by Heimo Pystynen and the Norwegian by Åge Holmestad and Harald Bjerke.

The data system for the electronic version has been developed by Consultec Byggteknik AB with a web-like interface. The system has been designed with different data bases to handle the various language versions. A number of interactive functions make it possible for the user to get the right glulam dimensions quickly and easily.

To be sure that one has the latest contents, the user is referred to the electronic version which will be updated regularly as the need arises.

Stockholm, August 2001

Holger Gross

Project co-ordinator,

Nordic Glulam Handbook

2.1. Glulam as a structural material

Glulam places no limits on the possibilities of timber building technology. Glulam is a structural material that optimises the technical properties of the renewable raw material timber. Glulam components consist of individual laminates of structural timber and provide an effective utilisation of material. The laminates are finger jointed to give greater lengths and are then glued together to produce the desired size. Thanks to the method of production, very large structural components can be made. With the aid of glulam, building owners, specifiers and builders can continue to enjoy the strength and versatility of large timber components.

Glulam has greater strength and stiffness than corresponding dimensions of structural timber. In comparison with its self-weight, glulam is stronger than steel. This means that glulam beams can span large distances with a minimal need of intermediate supports. It also means that architects and engineers have virtually unlimited opportunities of designing their own forms in glulam, regardless of whether the task is a small house, the roof of a department store or a road bridge.

If the aim is to optimise products from a well-managed source of raw material, glulam is one of the most resource-conserving ways of doing it. It is a structural material manufactured to satisfy the most exacting structural demands. Glulam is however by no means a new product.

The first glulam patents were issued in Germany around 1900. A German patent issued in 1906 Hetzer Binder was the real start of modern glulam technology. Some of the first glulam structures in Sweden are the concourses of the central, railway stations in Stockholm, Gothenburg and Malmö. They were supplied and built in the 1920s.

There are at least ten established glulam factories in the Nordic countries today. The manufacturing standards in the different countries are virtually identical thanks to a comprehensive cooperation between the building authorities. The coordination is organized and supervised by a common organization and, as a result, glulam from the nordic countries is marked in the same way: with the common L-mark (see figure 1.6a).

3.2. Structural systems

Glulam provides a wide variation of structural systems. In this chapter a number of basic designs of glulam structures for hall-type buildings are summarized from simple systems consisting of columns and beams to frame and shell structures, each one of which in different ways and to a greater or less extent utilises the opportunities provided by glulam.

The choice of structural system is above all influenced by the function of the building and by architectural considerations. Production and transport constraints can in some cases be critical (see 1.4).

Table 2.1 is a summary of the most common types of structure.

To make the choice easier, the table gives recommended span ranges and approximate heights for the various types of structure. They correspond to average values under normal conditions. Small loads or closely-spaced members reduce the height somewhat. The opposite also applies.

4.3. Structural Design

Design rules for load-bearing building structures aim primarily to minimise the risk that a failure in the structure should lead to serious personal injuries. It should feel safe to be in the building. In addition the aim is to ensure that the building works satisfactorily in normal use, e.g. by making demands on the stiffness of floors.

The building regulations stipulate approved verification methods. I.e. methods which show that a given requirement has been fulfilled. The regulations also state the conditions concerning loads, strength etc. which shall be used as a basis for design.

Glulam structures shall be calculated and designed in accordance with approved rules. Within the European Community Eurocode 5 Design of Timber Structures provide common structural design rules; in general supported by national annexes which safeguard the compliance with the national rules in force.

The Eurocodes, as well as most national codes within the EC, are based on the partial coefficient method. The method involves checking the construction in two states the ultimate limit state and the serviceability limit state. In the ultimate limit state the construction is checked for safety against failure. In the serviceability limit state the construction is checked for deformations sufficiently large to hazard the functional requirements on it. An example of deformation is deflection of floors. Design in accordance with EC5 presupposes in most countries a national adaptation document (NAD). NADs to EC5 have been prepared in all the Nordic countries.

Further, Eurocode 5 presupposes that

· The design is carried out by qualified and experienced persons

· Factories, workshops and building sites are subject to sufficient control

· Building materials and products are as specified in EC5 or relevant material or product specifications

· The building is maintained

· The building is used as foreseen in the design process.

5.4. Special considerations

Glulam differs from other modern structural materials in that it is, inter alia, a living material which moves both with variations in the moisture content and under long-term load. It is in addition extremely anisotropic, i.e. the properties of the material are markedly different in the direction of the fibres and transverse to them. The tensional strength is for example about fifty times as great parallel with the fibres as perpendicular to them. Special attention must therefore be paid to connections between two components and places where the flow of forces in the structure changes direction, e.g. at supports and at haunches of frames.

At an overall level this chapter deals with some of the points which deserve special consideration when designing a glulam structure.

6.5. Columns and Struts

Columns and struts are normally straight glulam components. They can be specially made or be standard beams from stock if one of the stock beam sizes is suitable. A column can easily be manufactured with a capital at the top to reduce the stress perpendicular to the grain in the supported beam, or with a larger cross-section at the base to take up large moments of fixture. Normally, columns are calculated as pinned at both ends or pinned at the top and fixed at the base.

Columns are normally given the same width as the beam or arch which they support, but they can also be designed with a greater width or as double, mechanically jointed columns.

7.6. Beams

Glulam beams can have a constant depth (straight beams) or a depth which varies along the length of the beam. Beams of varying depth include symmetrical double tapered and (exceptionally) asymmetrical double tapered beams, single tapered beams and roof beams in frames. Double tapered and single tapered beams are normally simply supported at two points.

Design values of moments and forces are calculated with the aid of equilibrium equations and linear elastic theory based on design load values and load combinations as given in current regulations.

Forces and deformations are calculated in accordance with the linear elastic theory for beams. Cross-sectional properties are thus determined on the basis of the net cross-sectional area, i.e. taking into account actual sectional size, notches and chases and with reductions for bolt holes and for nails and screws whose diameter exceeds 6 mm. If nothing else is stated it is assumed that the sections are rectangular with constant width. For beams whose sectional depth varies it is also assumed that the direction of grain is parallel with the underside of the beam.

8.7. Trusses and Space Frames

Trusses and space frames are systems of straight members meeting at nodal points and forming a load-bearing structure. If all the members are in the same plane this is called a truss. If not, it is usually called a space frame.

Trusses, the most usual form, are normally used like beams simply supported at two points. The remainder of this chapter deals with such trusses. Frames for stabilisation of buildings are dealt with in 12.2 Wind bracing.

Trusses have a number of advantages compared with solid structures:

- High material efficiency contributes to economy
- Great design freedom
- Can easily be made in several parts to facilitate transport
- Low self-weight leading to easy handling in factory and on site, with low transport costs.

9.8. Three-pin trusses

Three-pin trusses are normally designed as a propped construction with rafters consisting of glulam and ties either of glulam or steel. They are used for spans where ordinary timber trusses are insufficient. The roof can be designed with purlins, normally with the trusses at 6 - 8 m centres. The rafters can be augmented by steel ties to improve the material efficiency. The roof slope for three-pin trusses should be between 14 and 30 degrees and the span between 15 and 40 m. Roof trusses with steel tie rods can however span considerably more   50 m or longer.

10.9. Portal Frames

Frame structures of timber are today almost without exception executed in glulam. The haunch can be made curved with continuous laminates, finger jointed, jointed with steel dowels and slotted-in plates, or built-up (see figure 9.1). The form of the frame should follow the force line of the main load, as far as functional and aesthetic considerations permit. Curved or built-up haunches fulfil this desire most easily and are therefore the forms best suited to large spans. The roof slope should not be less than 14° due to, among other reasons, the wish to reduce the deflection of the ridge.

Three-pin portal frames are suitable for spans up to 30-40 m. If spans are greater, the two halves of the frame will be too large to transport in one piece. The connecting line between ridge and foot should thus not exceed 24 m, and the distance at right angles from this line to the outer edge of the haunch should not be more than 4 to 5 metres.

The two-pin portal frame provides a stiffer structure but generally means that the frame must be manufactured and transported in three or more parts which are jointed with rigid joints on the site. Joints can suitably be placed at positions in the structure with small moments. Rigid joints demand more complicated workmanship than hinges and therefore command a higher price. They are in addition often highly visible in an undesirable way. The parts of the frame are on the other hand smaller than those in a corresponding three-pin frame and therefore easier to transport.

Timber frames with one or no pins are not usually employed for load-bearing structures.

11.10. Arches

Arches are a type of construction very suitable for execution in glulam   a material which without a great increase in price can be produced in curved forms and with varying depth. As a rule, solid sections of constant depth are used, but composite sections of I or box form also occur, specially for large spans.

The form of the arch is chosen so that the moments are as small as possible. As a rule, this means that the arch follows the thrust line (equilibrium polygon) of the dominating loading combination. The influence of moments can however not be avoided completely, as several load combinations must be taken into account, each with its own thrust line. As a compromise a parabola is often chosen, or for small spans a circle. For functional reasons, e.g. to increase the headroom near the supports, an elliptical or other arch form can be preferable. The dividing line between frames and arches is fluid here. The same result can be achieved by placing the arch on columns, see figure 10.1. The horizontal support reactions caused by the arch must in this case be taken care of by a tie rod between the springing points of the arch. When the arch rests directly on the ground floor slab, e.g. as in figure 10.2, the horizontal forces can be taken up by the foundations if ground conditions permit, or by tie rods under the floor or cast into it. To limit the size of the horizontal reactions the rise of the arch should be equal or greater than 0.14 of its span. For a parabola this corresponds to an angle of spring of 30°.

The choice between two- and three-pin arches is made after similar considerations to those for frames (see chapter 9). Three-pin arches are thus preferable over spans of up to 60 70 metres, while larger spans usually demand that the arch is manufactured and transported in three

or more parts, which are joined rigidly on the site. Hinges and rigid joints should be placed as in figure 10.3.

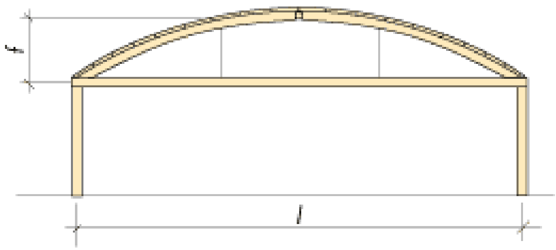


Figure 10.1 Arch with tie rod, on columns.

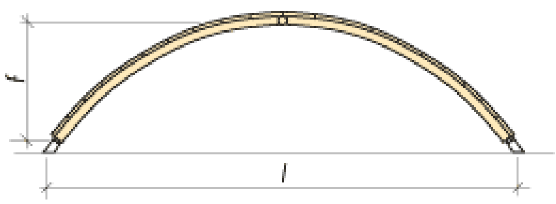


Figure 10.2 Arch springing from foundations.

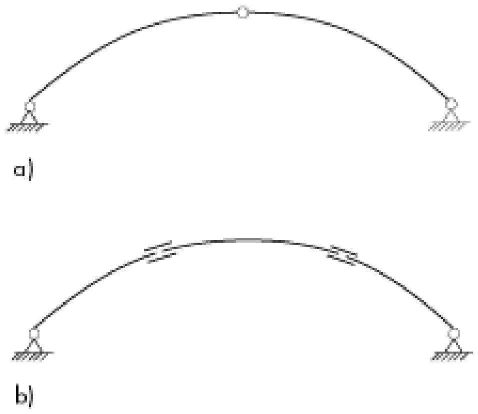


Figure 10.3 Suitable placing of joints in arch structures. a) Hinge b) Rigid joint.

12.11. Purlins

Purlins normally consist of straight beams of constant section. They can be simply supported on two supports and are then hung in between the primary beams, or alternatively be continuous beams with several supports, usually placed on top of the primary structure. Continuous purlin systems can in turn consist of unjointed purlins in two or more bays, purlins with rigid joints, often in the form of overlapping over supports, or with hinged joints in the Gerber System .

13.12. Bracing of Glulam Structures

An important stage in the design of a building structure is a check on the total stability. To be able to take up horizontal loads, e.g. wind loads, the structure must be stabilised in accordance with one of the following alternatives:

- The diaphragm action of the roof which transfers the horizontal load to wall diaphragms, normally placed in the gable walls.

- Wind trusses in the roof which transfer the horizontal load from columns to braced columns, normally placed in the gable walls as in figure 12.1.

- One or both of the columns is rigidly fixed to the foundations as in figure 12.2a.

- One or both of the columns is rigidly fixed to the beam as in figure 12.2b, forming a three- or two-pin frame respectively.

- The structure is complemented by diagonals as in figure 12.2c. For functional reasons this is usually only possible in gables.

- The structure is rigidly connected to a wall diaphragm in its own plane.

14.13. Fixing details

Strictly speaking, the art of building in timber is the art of correctly joining one piece of timber to another. Only with the aid of well thought-out and carefully made joints can the opportunities of the material be used to the full. Good design and workmanship are characterized by, amongst other things:

- The route of forces through the joint is short and well-defined
- Forces perpendicular to the grain are transferred over such a wide area that

strength is not exceeded (see 4.8)

- Moisture movement can take place without leading to splits or damage (see 4.9)

Water- and dirt-collecting pockets are avoided.

15.14. Designing for Fire Resistance

Timber is a combustible material and numberless fire catastrophes have occurred over the years, leaving their imprint on the building regulations in the form of restrictions on the use of timber in building.

Experience has however also shown that heavy timber members exposed to fire retain their carrying capacity for quite a long time. This experience is nowadays reflected both in the regulations, where glulam structures are allowed in buildings with high demands on fire safety, and in insurance rates where frames of glulam are generally equated with those of concrete. A positive attitude from the fire brigades is due amongst other things to the fact that firemen can easily assess how far charring has taken place and thus how much of the load bearing capacity remains.

16.15. Transmission poles

The networks for distribution of electric power and for telecom are normally in the form of overhead lines. Only in densely built-up areas is it normally economically defensible to bury the lines underground. The overhead networks therefore have a noticeable effect on the landscape, something which has received more attention in recent years and to which great importance is attached in the decision-making process of the authorities when proposals for new lines are examined.

The power network in the Nordic countries consists mainly of natural timber poles. For lines under 130 kV the dominance is total. An increasing part of new construction consists however of glulam poles. When old timber poles are replaced, glulam is often the only choice as the substantial poles of the old days are now in short supply.

Glulam poles for power lines have the same good characteristics as roundwood poles:

- long life, low maintenance
- simple foundations
- small energy loss through the pole
- aesthetically attractive and environment-friendly
- low bending stiffness enabling loading redistribution between spans, e.g. in connection with non-uniform ice loading.

Compared with roundwood poles, glulam offers additional advantages:

- shorter delivery times
- lower weight for same load capacity
- less risk of woodpecker damage.

Today there is fifty years of experience of glulam poles in the USA and elsewhere. In the Nordic countries it is Norway which has the longest experience; since 1980 more than 6000 poles have been erected. Swedish and Finnish glulam manufacturers also have glulam poles on the production programme today.

17.16. Glulam bridges

The first bridges were probably fords which had been improved by laying out some extra, strategically placed stones. From there it was a short step to laying a tree trunk between the stones thus inventing the (albeit primitive) bridge. The needs of transport (not least of moving troops) pushed development forward, and the Roman Empire marked an early high point.

As early as the spring of 55 B.C., Julius Caesar´s soldiers built a 140 m long stock bridge over the Rhine near Koblenz. The roadway was about 5 m wide and the erection time reputedly ten days. The advantage of this type of bridge was that it could obviously be built very quickly and with relatively simple means. Amongst the disadvantages were the disturbance to boat traffic and the comparatively short life-span, especially that of the poles. At the most important sites this led to a change to bridge columns of stone, placed further apart than the yokes consisting of poles. The simple timber beams had therefore often to be replaced by more complicated structures such as truss systems and strut-frame systems. The timber structure was now entirely above the high water mark and was better protected from damp, ice and running water. In Central Europe, where there is a long tradition of timber bridge building, the roadway was often roofed to shield the timber and the users from rain and snow. In the Nordic countries there is also long experience with timber bridges (though seldom roofed) but few are preserved. One of the oldest preserved examples is Lejonströmsbron in Skellefteå, Sweden which was built in 1737 and which, after some rebuilding, is still open for traffic.

18.Annex

19.Pictures

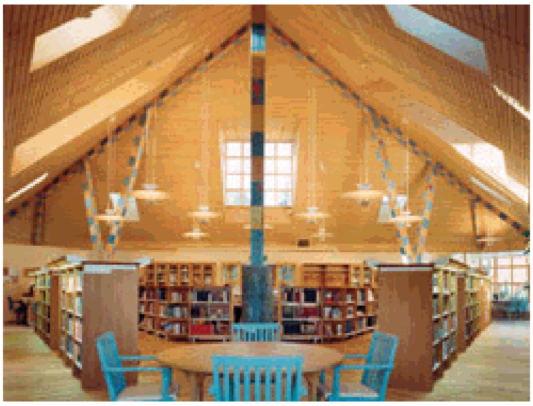
Central Station, Stockholm, Sweden. Architect: Folke Zetterwall, 1925.

Fredells Home Center, Stockholm, Sweden. 1998.

Mälardalens University, Västerås, Sweden. Architect: Dag Cavallius, Nyréns arkitektkontor, 1998.

Håkons Hall, Lillehammer, Norway. Architect: Ostgaard Arkitekter A/S, 1994.

Hedmark Museum, Hamar, Norway. Architect: Sverre Fehn, 1969.



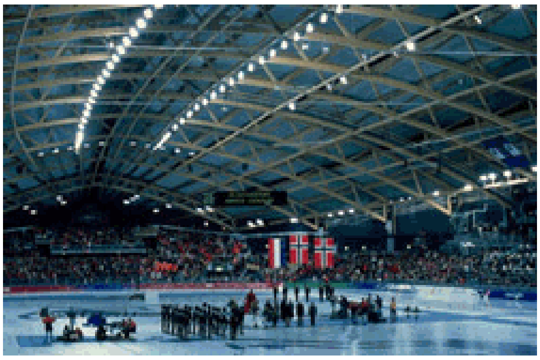
Visingsö Library, Visingsö, Sweden. Architect: Carl Nyrén, 2000.

Library, Linköping, Sweden. Architect: Johan Nyrén, 1999.

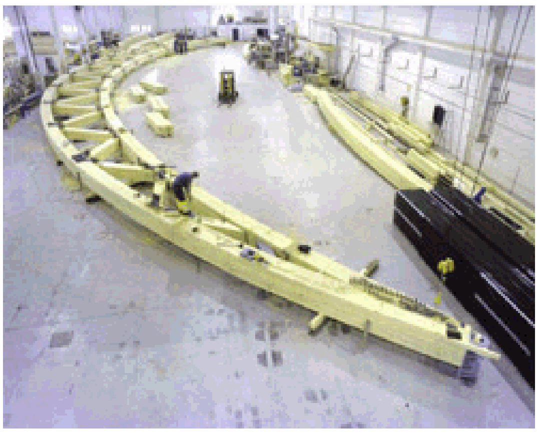
Sibelius Hall, Lahti, Finland. Architect: H. Tikka and K. Lintula, 1999.

Barn, Rödåsel, Sweden. 1999.

Riding School, Vrena, Sweden. Architect: Johan Mörling, FFNS, 1999.



The Viking Ship, Hamar, Norway. Architect: Biong & Biong/N. Torp A/S, 1994.



Glulam Factory Interior, Moelven, Norway.

Store Building for pellets, Skelleftehamn, Sweden.

Music University, Gothenburg, Sweden. Architect: Nyréns arkitektkontor, 1998.

Håkons Hall, Lillehammer, Norway. Architect: Ostgaard Arkitekter A/S, 1994.

Fire Station, Mikkeli Paloasema, Finland. Architect: ArkIdea, JP Tuomainen, P. Spoanen, I. Svärd, T. Sinkko, 1999.

Lightening posts, Östersund, Sweden. Architect: J. Grönvik Ark.kontor.

Poles for Overhead Lines, Sör/Tröndelag, Norway.

Bridge over Vihantasalmi, Finland. Architect: Ranta-Kokko & Co Oy, 1999.

2.1.1.1 Introduction

Glulam technology began to develop in Germany at the end of the nineteenth century and came, via Norway, to Scandinavia at the beginning of the twentieth.

Up to the beginning of the sixties, production was fairly small, but since then it has increased continually and the total production in the Nordic countries is now over 200.000 m^3, of which roughly half is exported.

Most of the glulam sold in the Nordic countries goes to the building sector, principally to industrial buildings, schools, day nurseries and housing. Together these account for about 60% of consumption. Glulam is however a material with many uses and has over the years been used in the most wide-ranging uses, from shuttering , scaffolding and playground equipment to bridge building, multi-storey car parks, ski slopes and electricity pylons.

Modern gluing technology in combination with the good strength qualities of timber makes glulam a highly qualified structural material with a unique series of characteristics:

- appearance which appeals to most people and which acts as a valuable addition to the interior and exterior environment

- high strength/weight ratio, enabling wide spans

- small manufacturing tolerances and good form stability within normal temperature and moisture conditions

- high resistance to fire   often a requirement in public buildings

- good heat insulating characteristics, reducing the effect of cold bridges and the risk of condensation

- low weight, resulting in low transport and erection costs and favourably affecting the cost of foundations

- long life in chemically aggressive environments

- flexible production, enabling curved structural components to be produced more cheaply than in other materials

Glulam structures are characterized by speedy and simple erection of prefabricated units. The parts are assembled by nailing, screwing or bolting,, unaffected by the time of year or the weather, and any

adjustments can be made with simple hand tools. Timber building is a dry building method and a glulam frame can carry its full load immediately after erection.

Timber is a well-tried material which, correctly used has extremely good durability. In the Nordic countries there are examples of timber buildings over a thousand years old!

Glulam manufacture uses little energy. The raw material is constantly renewed. It is taken from the natural life cycle and can be returned after use without negatively affecting the environment.

2.2.1.2 Glulam in the life cycle

Products originating from nature shall in a durable manner be used, reclaimed, recycled or finally taken care of with a minimum use of resources and without affecting the environment.

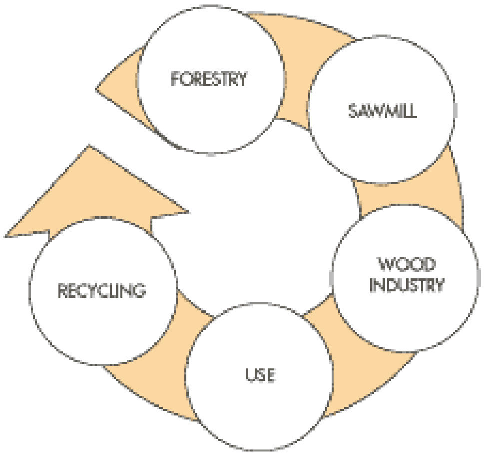


Figure 1.1 Life cycle of timber products

Glulam is made of timber laminates glued to one another under controlled conditions. They do not affect the environment during their life cycle and can easily be used, reclaimed or used for energy production.

The production of glulam is a process which demands little energy. The raw material is Baltic spruce (picea abies), and a synthetic glue. The glue is made from non-renewable raw materials, which affects the environmental profile negatively. The amount of glue per unit of volume is however so small (less than 1% weight) that the effect on the final product is negligible. During manufacture, some process-related emissions occur in the form of water used for cleaning the gluing equipment, hardened excess glue and small amounts of esoteric material during hardening.

The products are supplied at a moisture content of 12%. The drying process is largely fuelled with sawdust and other bi-products. This reduces the use of electricity.

Since glulam is often tailored for the project it does not cause significant building rubbish on the site. Wrappings consist of material which can be reclaimed.

During its lifetime, glulam has no negative environmental effects of any importance. It can be maintained using traditional methods. It can easily be repaired   parts of a glulam component can if necessary easily be replaced. It can if needed be worked afterwards in various ways, e.g. by abrasive treatment or rubbing down. Within limits it is possible, after making strength calculations, to take up holes and notches.

Glulam products can be re-used if their strength class and loading history are known. The quality controller must check the condition of the glulam and assess the conditions for re-use in each particular case.

Like all timber, glulam is combustible and can, if used unsuitably or in poorly detailed construction be subject to biological decay. The energy content is the same as that of solid softwood.

Most Nordic glulam producers have presented environmental statements in accordance with a common layout. These show the environmental impact of the product during the part of the life cycle which can be controlled by the producer, that is from the taking out of the raw material to the point where the finished product leaves the factory. Environmental statements can be ordered free of charge from the glulam producers.

For those who wish to assess the glulam product's environmental impact over the complete life cycle, the swedish glulam producers have produced a common building product statement which is reproduced below.

(the file is on the Swedish Glulam Manufacturers Association's website www.svensktlimtra.se)

2.3.1.3 Glulam manufacture

The term glulam in the text which follows covers structural components consisting of glued boards or planks with the grain in the longitudinal direction of the component and the gluelines parallel with the width (normally the smaller face of the beam).

2.4.1.4 Glulam components. Sizes and form

Glulam technology makes it possible to vary the cross-sectional form, the geometry and the size of structural components. The limits are set by practical considerations such as the size of the production area, the capacity of the mechanical equipment and the possibilities of transport, etc. Some of these limiting factors are commented on below.

Rectangular cross-sections are usual, but other cross-sections can be manufactured, e.g. I, T and L sections or hollow sections, rectangular or 12 sided (see figure 1.10). The lastnamed are mainly used as electricity pylons but can be used with advantage as columns in buildings.

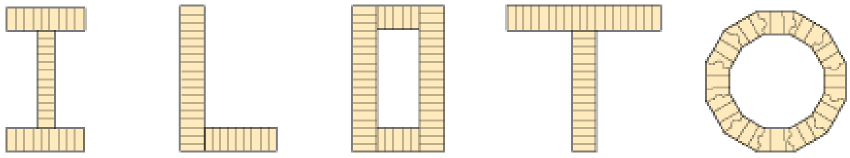


Figure 1.10 Examples of composite glulam sections.

2.5.1.5 Appearance and surface finish

Glulam is in the first place a structural material whose strength, stiffness and durability are generally the most important properties.

Glulam components do not normally have the timber quality and surface finish which are normally demanded in joinery and furniture. However, in the great majority of cases standard products will fulfil normal appearance requirements provided they are treated with suitable care during transport and on the site.

2.6.1.6 Transport and erection

Transport and erection are the last operations in the building of a glulam structure and it can be thought that they are of minor importance. However, they demand the same care as the previous operations, since they can have a decisive effect not only on the design but also on the planning and the economy of the project.

2.7.1.7 Ordering and delivery

On drawings and in the specification, the following are stated:

- Identification code (e.g. B1, P3 etc)
- Type of component (e.g. as in table 2.1 or refer to drawing)
- Nominal size (width x depth at left support/maximum depth/depth at right support x length in mm). For constant-depth beams or columns only one depth is given. For special component types e.g. asymmetrical double-pitch beam sizes should be given on a drawing.
- Strength class.
- Glue type (I or II in accordance with EN 301).
- Surface processing (state if necessary which sides are visible).
- Surface treatment (if desired).
- Camber (if desired).
- Timber species (other than spruce, e.g. pressure treated pine)
- Permissible deviations.

3.1.2.1 Beam and Column systems

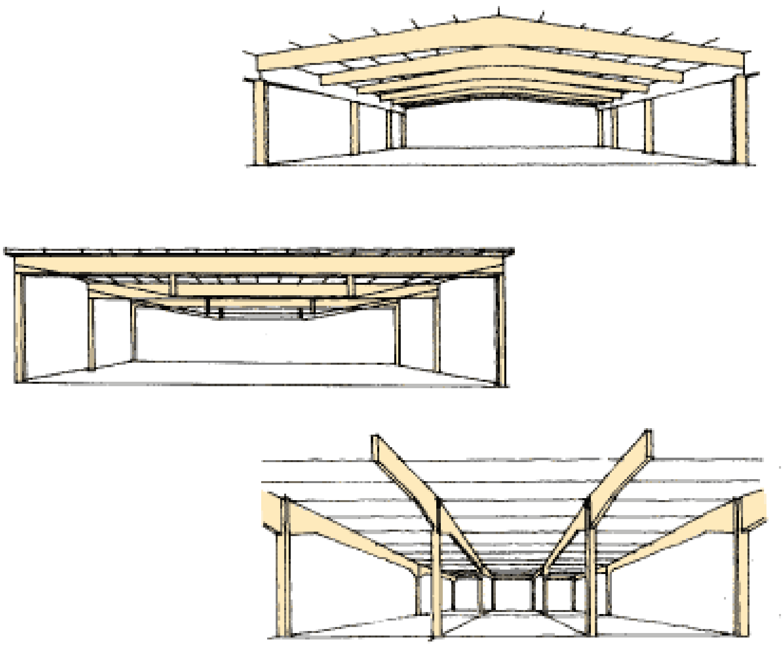


Figure 2.1 Beam and column system.

In its simplest and most common form the glulam structure consists of beams freely supported at each end by columns. For small spans beams of constant section are often preferable, while for larger spans it

may be worthwhile economically to allow the sectional height to vary with the forces in the beam.

An example of this is the symmetrical double pitched beam, where the depth is at a maximum in the middle where the bending moment is greatest.

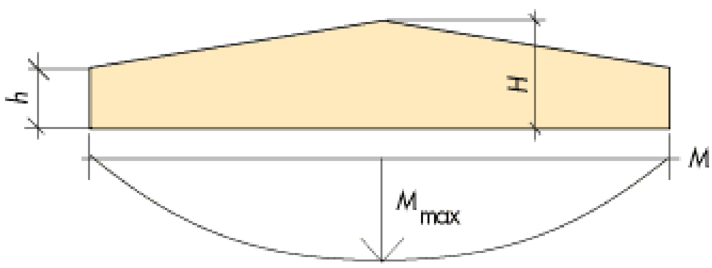


Figure 2.2. The shape of a symmetrical double pitched beam approximates to the moment diagram of a beam freely supported at each end. It is therefore more economic in material than a constant-depth beam.

It is often deformation, i.e. the greatest permitted deflection that can be accepted, rather than load-bearing capacity, which is crucial for the lowest structural depth that can be chosen.

Glulam beams are normally designed with a straight underside but they can also for aesthetic or functional reasons be given a more or less pronounced curve. A popular form is the pitched cambered beam whose form resembles a boomerang a symmetrical double pitched beam with a curved underside (figure 2.1 upper).

Services form an important part of beam's function and affect the architectural impact to a high degree. A question which often arises is therefore if it is possible to make holes and notches in glulam components. In a normal beam the whole of the cross-section is used to take up lateral forces. These are greatest at the supports and it is therefore usually unsuitable to make holes or notches there. Further, holes should be placed in the centre of the cross-section as bending moments are least there. Figure 2.3 illustrates the area in a freely supported beam where any hole should preferably be placed.

The above argument deals with principles. Detailed instructions on how holes and notches should be designed are given in Chapter 4, Special considerations.

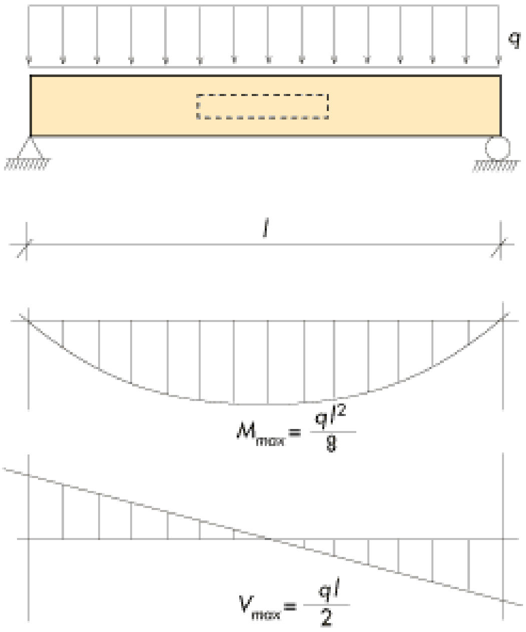


Figure 2.3 External moments and shear forces in a beam freely supported at two points with an evenly distributed load.

Glulam columns normally have good load-bearing capacity. A clamped column, free at the top end has a buckling length of approximately twice its height. In a normal column, which is hinged at the top and the base, the buckling length is equal to its height.

It is normal that the design of the building enables the columns to be restrained at the top, e.g. by being connected to the roof construction. In low buildings up to 3 - 4 m high it is usually economic to fix the columns in the foundations to ensure stability. The foundations must then be designed for the resulting moments. In higher buildings it is usually advantageous to arrange diagonal bracing or a wind girder. See further chapter 12, Structural stabilisation.

3.2.2.2 Continuous beams

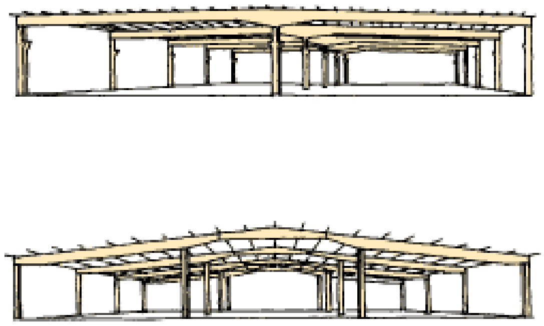


Figure 2.4 Continous beams.

Beams with several supports or beams with a cantilever make possible a better use of material than that which can be achieved with beams freely supported at two points. The degree of utilisation can be further increased by increasing the depth of the beam at the inner supports (see figure 2.4, lower).

Continuous beams can with advantage be designed on what is known as the Gerber System. The joints are then designed as hinges, and placed so as to achieve a favourable distribution of moments and suitable lengths for transport.

Continuous beam systems are particularly suitable for roofs, e.g. as secondary beams (purlins).

3.3.2.3 Solid Decking

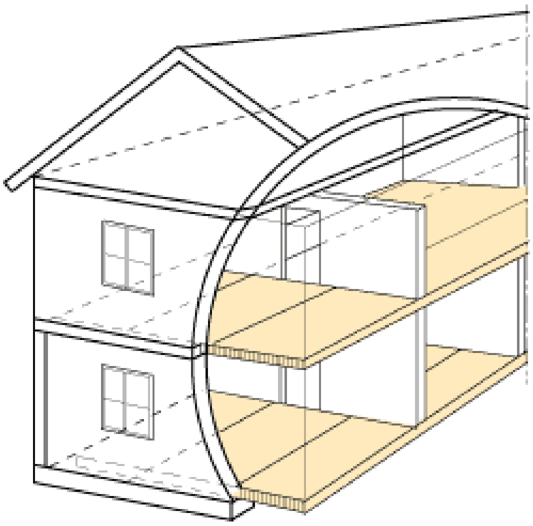


Figure 2.5 Solid decking.

Solid decking of glulam components is a good alternative to concrete or to other types of light floor construction, particularly in multi-storey timber buildings. The decking consists of glulam components laid on their sides. Individual components can with advantage be connected by post-tensioned steel rods perpendicular to the direction of the laminates. The transverse stress gives a certain amount of diaphragm effect which improves stiffness and enables the floor to be used as a stabilising component. It also reduces lateral moisture movement and safeguards the good fire and sound insulating properties of the floor. When there are requirements on sound insulation it must however as a rule be complemented with a sub-floor or a false ceiling.

3.4.2.4 Trusses

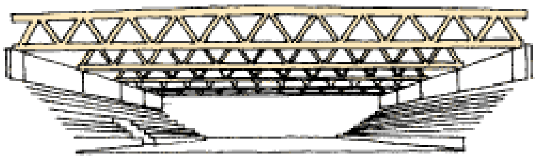


Figure 2.6 Trusses.

Over large spans, where solid beams tend to be too clumsy and to use too much material, some type of truss can be a viable alternative. This applies particularly where a low roof slope is required and where the construction height is fairly ample.

Among the advantages is the fact that they can be made in the factory in suitable sections for transport, which are assembled on the building site. Among the disadvantages are numerous, sometimes complicated, nodes. It is desirable that the architect should take part in the design of the truss, especially the nodes and other details.

Services can in many cases be placed near the top member or above the bottom member, and need not give the impression of being a visible obstruction in the volume. The volume can be seen as following:

the top member and the underside of the insulated roof. Compression members are designed in glulam, while tension members can be in steel. The constructional height is in this case the distance between the centres of the top and bottom members. This often gives the architect a great deal of freedom to influence the constructional height.

3.5.2.5 Three-pin trusses

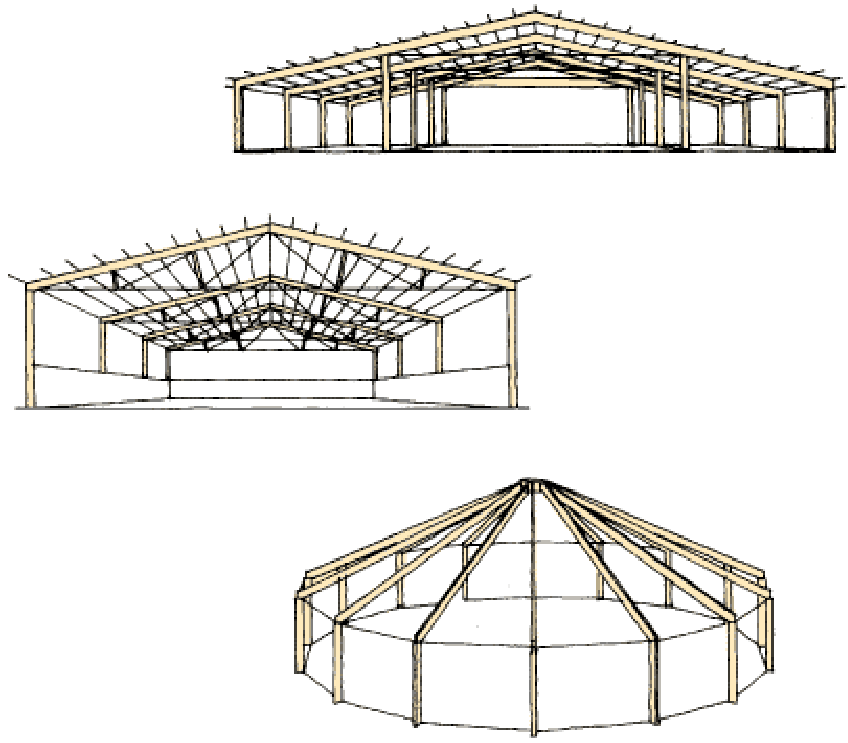


Figure 2.7 Three-pin trusses.

Three-pin trusses and other truss systems can provide a solution where demands on span exclude solid beams and where arches or other frames cannot be used for various reasons.

In its simplest form, the three-pin truss consists of two beams leaning against each other with a hinge connection at the ridge. The bottom ends are similarly hinged to the foundations, or connected to each other with a tension member, often of steel (figure 2.8 upper). In the latter case the truss is normally supported by columns, but the tension member can also be cast into the floor slab. The beams are usually straight and of constant depth, but variations can also occur here.

The bottom-tensioned beam can be regarded as a transitional form between solid beams and trusses. The joints are however fewer and simpler in design than in a pure truss. Today there exist firms which produce purpose-made steel parts such as tension members and joints.

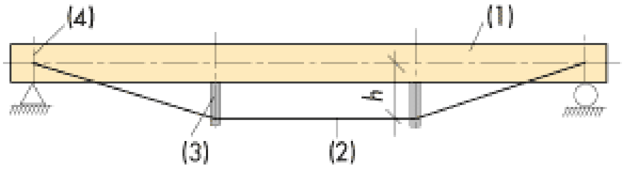


Figure 2.8. Example of bottom-tensioned beam with a constructional height of h. (1) beam. (2) tension member. (3) compression member (4) recessed or external steel fixtures.

Three-pin trusses can with advantage be designed as space frames. The roof beams are then arranged to radiate from a summit and the tension members replaced by a polygonal tension ring, linking the bottom ends of the beam round the edge of the roof (lower figure).

3.6.2.6 Arches

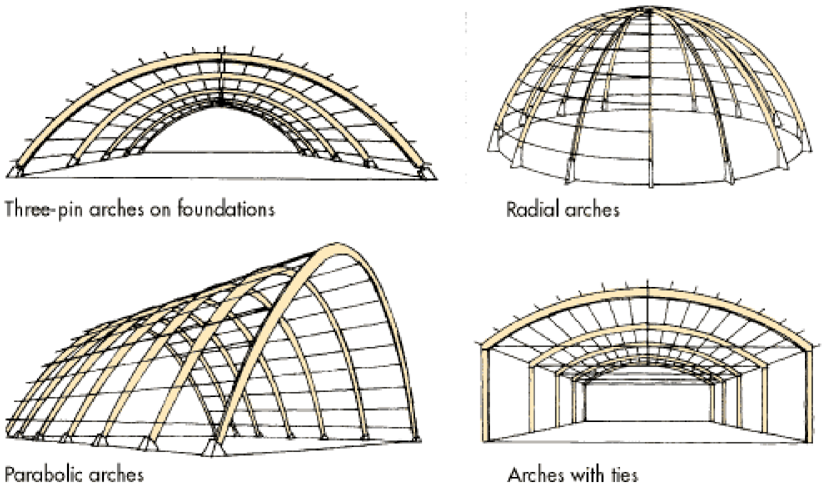


Figure 2.9 Arches.

Glulam is an exciting constructional material, partly because of the ease with which curved forms such as arches, frames etc. can be built. For each type of loading the most functional form the compression line can be chosen. An arch which follows the compression line and which is subjected only to vertical loads will be loaded in pure compression throughout its whole length. If the load is equally distributed the load line will be a parabola; with point loads it will be a polygon.

Thanks to the fact that the material is better utilised in an arch, the constructional height will be only 1/3 of that in a beam of the same span and loading. The difference in the way a beam functions, compared to an arch, is illustrated in figure 2.11.

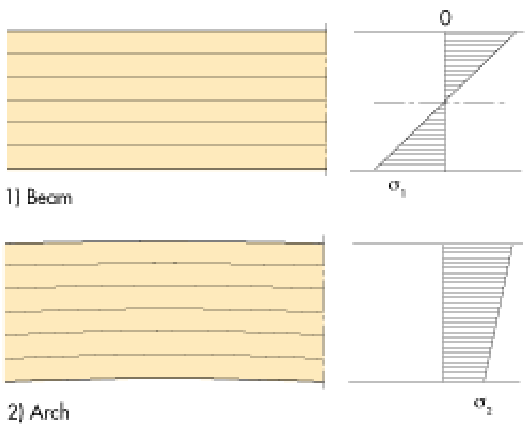


Figure 2.10 Bending stresses in a loaded beam (1) vary through the depth. Compression at the top and tension at the bottom. At the centre of the cros section the stress = 0. An arch (2) on the other hand has compression throughout its cross-section.

The design options, together with the high strength, mean that glulam construction is particularly viable over large spans. Free spans of over 100 m have been built.

Circular arches are the most usual form. For large spans, however, parabolic arches can be more economical. To increase the headroom near the supports an elliptical or other arch form can be chosen. Another method of increasing headroom is to place the arch on columns.

An arch demands stable supports, which can be provided by an adjoining structure, by the foundations or by special tension members. These can be visible, or in hall-type buildings under the floor slab.

Arches are normally built with hinged fixtures at the supports and (usually) a hinged joint at the ridge. In larger spans, more joints can be desirable for transport reasons. These are rigid and placed in areas where the moments are small.

The three-pin arch is statically determinate, which means simple calculations and insensitivity to subsidence of the supports. It is also stable in its own plane and does not give any bending moments in the foundations.

Arches radially arranged give a dome-like form. A genuine dome also utilises the shell effect, which demands special tangential design of the structure. If spans are large, and in particular if the area to be covered extends in several directions, a dome can be an interesting solution. In Tacoma, Washington there is a glulam dome with a span of over 160 m.

3.7.2.7 Portal Frames

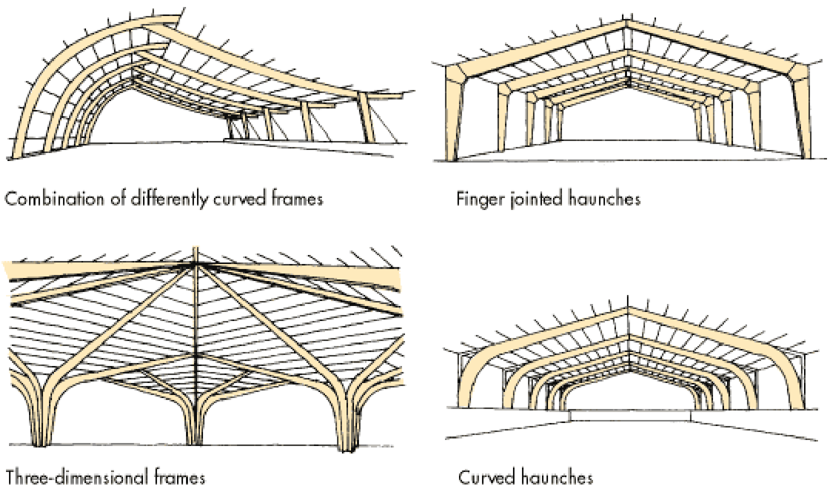


Figure 2.11 Portal frames.

For functional, aesthetic or economical reasons another type of arch than the material-saving parabola or the circular arch may be preferable. Requirements on headroom throughout the whole area of the building often lead to the characteristic glulam form of a three-hinge frame with curved haunches or, if demands on utilisation of the whole area are extreme, sharp-cornered haunches.

The function of the building is improved in both cases at the cost of somewhat lower utilisation of material. In other respects the three-pin frame has the same advantages as the three-pin arch   simple design and foundations. It is specially suitable on poor subsoils as it does not transfer any bending moment to the foundations.

The traditional form is symmetrical on plan, but interesting volumes can be achieved through combination with other constructive elements   curved or straight   or by three-dimensional arrangements of half-frames.

3.8.2.8 Cantilevers

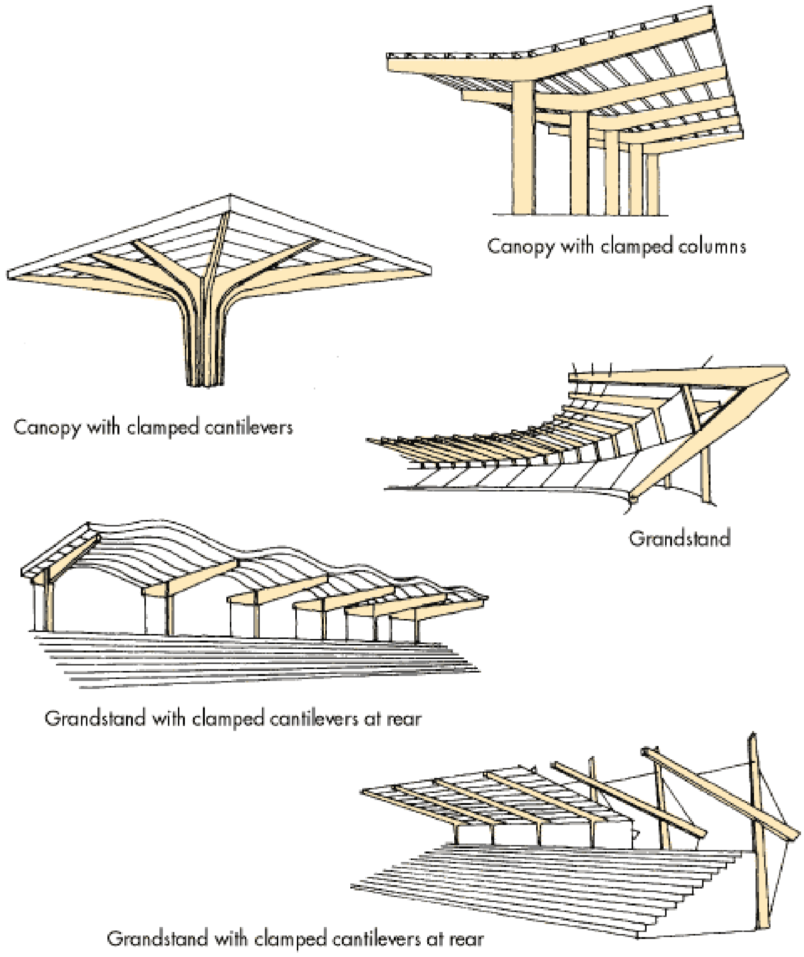


Figure 2.12 Cantilevers.

In many situations it is a requirement that one or both the long sides of a building are open and free from columns. Examples are open-air stages, platform roofs and grandstands.

In such cases glulam offers solutions in the form of cantilevered, straight beams or curved brackets half frames. In both cases large fixing moments are transferred to connecting structures, which must be designed accordingly.

3.9.2.9 Grids

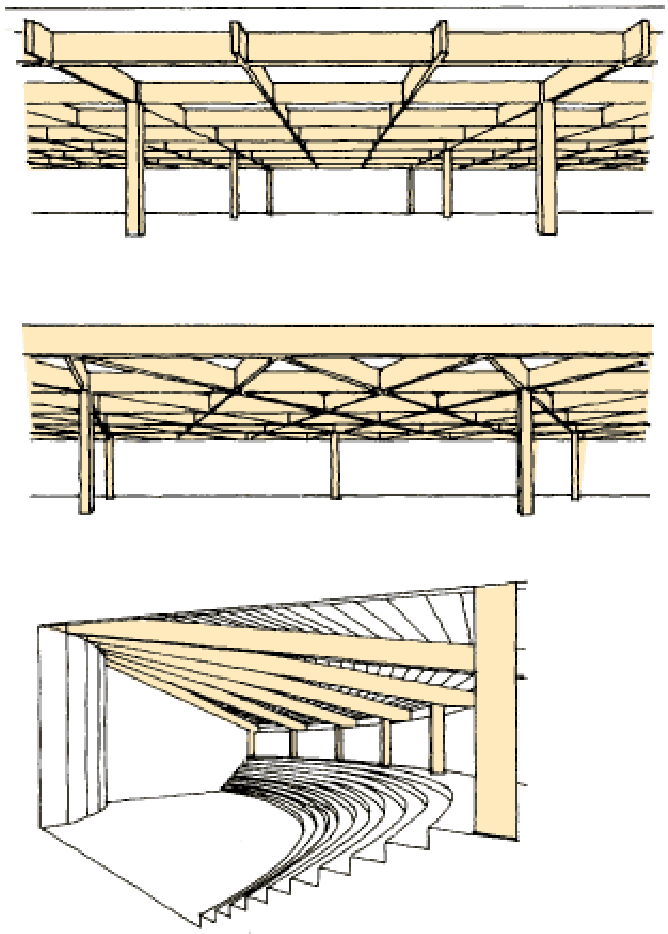


Figure 2.13 Grids.

The beams can be arranged radially or in the form of a grid. The latter type of structure spans in several directions and therefore allows a lower depth than traditional beam systems. On the other hand it burdens the budget with a large number of relatively expensive intersections. The advantages of the system can best be used if the span and distance between columns are the same in all directions.

4.1.3.1 General

Since structures often function differently in normal use and when nearing failure it is usual to distinguish between design in the *serviceability limit state*and in the *ultimate limit state*. Limit state means the state in which a structure or a part of the structure just fulfils the given requirements.

For design in the *ultimate limit state*the structure shall have adequate safety against failure as long as it is used in the intended way. What can be regarded as adequate is stipulated in the current regulations.

For design in the *serviceability limit state*no compulsory requirements are usually made in the regulations. Recommendations are considered sufficient, and it is left to the building owner, or his adviser the designer, to decide what is acceptable. In this way it is possible to avoid a situation where inflexible requirements lead to unrealistic and uneconomic consequences.

The risk of reaching a certain limit state, e.g. a failure, in the structure depends on how uncertain the conditions are on which the calculation is based. i.e.

- the likelihood that the assumed loads are exceeded

  the likelihood that the calculated loading capacity is not reached

The risk that a structural failure will lead to personal injuries depends in turn on

- the likelihood that there will be people in the building when failure occurs

Normally, both the external influences of loading etc. and the ability of the structure to resist these influences are regarded as stochastic (random) variables. If the distribution factors for these variables are known it is therefore possible to calculate the risk of failure using methods based on probability theory. Knowledge on these distribution functions is however incomplete; in particular this applies to the outermost parts, the tails which are critical for the result. Therefore, various standard functions are used instead such as normal distribution and Weibull distribution. The risk of failure calculated in this way is purely theoretical and the method is therefore not usable for practical design. That is to say, not with the present level of knowledge. This is compounded with the fact that the calculation process itself is complicated.

Even though probability theory methods are thus not usable for design in individual cases they can with advantage be used for comparisons, e.g. between different materials or different types of construction.

Probability theory methods are therefore important as tools for calibrating other, simplified methods, for example the partial coefficient method.

4.2.3.2 The Partial Coefficient Method

A verification method which is more suitable for practical use than strict probability theory methods is the partial coefficient method. The partial coefficient method is deterministic, i.e. it is based on the assumption that the parameters on which it is based are not stochastic (random) but can be given definite values. The values of the parameters, such as partial coefficients, values of load and values of strength, are however determined on the basis of probability-theory calculations.

The partial coefficient method is today generally, internationally accepted and is used in the common European rules for timber, Eurocode 5 Design of timber structures (EC 5).

The method uses several different safety factors   partial coefficients   each of which takes into account the various types of uncertainty affecting the calculations. For example

· loading assumptions (γ_f)

· calculated loading capacity (γ_m)

· consequences of failure (γ_n)

In the rules, the partial coefficients and the characteristic values of loads and strength of material etc. are given as a basis for design. With these starting points the critical action effect $S_d$and the critical resistance $R_d$are calculated. In the ultimate limit state the following critical condition applies:

$$R_d \geq S_d$$

Formula 3.1

In the serviceability limit state the critical condition is usually formulated in such a way that the calculated deflection, the width of cracks etc shall be less than an absolute or relative requirement value.

The application of the partial coefficient method can vary from one country to another. Within the Nordic countries the various consequences of a failure are treated by placing various types of building or parts of a building in different safety classes (see below), a concept which does not exist in the Eurocodes..

4.3.3.3 Recommendations on camber and limitation of deflections

From the appearance point of view there is reason to limit the deflection of, for example, roof beams. Deflections as small as 1/300 of the span are visible, especially if horizontal reference lines exist. Such visual requirements should however be formulated differently for different types of building   for example lower in a storage building than in an exhibition hall.

5.1.4.1 Size effect

The bending strength of large glulam beams is, under certain circumstances, lower than that of small beams. This size effect (also called volume effect or depth effect) is fairly well documented for short-term loads and there is also a striking theoretical explanation of the phenomenon in Weibull´s weakest link theory. Since the relationship between the length and depth of beams, like that between width and depth, varies within quite narrow limits the size effect in bending and in tension parallel with the fibres can equally well be perceived as a depth and/or length effect. With this background, North American and other regulations have for many years had a depth factor to correct the bending strength of glulam. In new American regulations the factor is a function of the length, width and depth of the beam, and of its species and the shape of the moment diagram.

Both the experimental documentation and the theoretical explanation of the size effect apply to short-term loading and constant climatic conditions, when timber behaves elastically and has a markedly brittle fracture. Building structures on the other hand are subject to major variations in moisture content and the loads which are usually critical have a duration of several weeks or months, e.g. snow load. In these, more realistic, conditions, the effects of creep in the timber are not inconsiderable and some redistribution of forces can be expected to occur within the critical volume of the beam. In addition, the experimental basis is altogether too meagre to permit conclusions other than those concerning mean values. New studies seem to indicate that the size effect is considerably smaller when characteristic values, rather than mean values, are compared. It has therefore been questioned whether one should pay attention to the size effect with regard to bending, in the present state of knowledge.

Several modern regulations, among them EC 5, have taken up the size effect for glulam whilst it has been ignored in others, e.g. the Norwegian and German regulations.

In bending and in tension parallel to the direction of the fibres it is usual to take the size effect into account by multiplying the characteristic strength value with a modification factor:

$$k_h = \left(\frac{h_0}{h}\right)^{\frac{1}{m}}$$

Formula 4.1

where

h_o= depth of a reference beam

h= depth of the beam in question

m= constant determined by testing

In EC 5, m= 5

In contrast to the case of bending and tension parallel with the fibres, there is general agreement that the size effect shall be taken into account when considering tension perpendicular to the grain (see 4.8).

5.2.4.2 Lateral buckling

Deep and narrow structural components loaded with bending moments about the horizontal axis can   if they are not restrained from turning or sideways bending   buckle under a load which is lower than the bending failure load. Safety against buckling can, in accordance with EC 5, be checked by showing that

$$M_S \le k_{crit} \cdot M_R$$

Formula 4.2

where

M_S= design bending moment in the ultimate limit state

M_R= design carrying capacity in bending, in appropriate cases reduced on account of bending of laminates (see 4.7)

$$k_{crit} = \begin{cases} 1,0 & \text{för} \quad \lambda_m \le 0,75 \\ 1,56 - 0,75\lambda_m & \text{för } 0,75 < \lambda_m < 1,40 \\ 1/\lambda_m^2 & \text{för } 1,40 \le \lambda_m \end{cases}$$

Formula 4.3

$$\lambda_m = \sqrt{\frac{f_m}{\sigma_{m,crit}}} = \sqrt{\frac{bh^2 f_m}{6M_{crit}}}$$

Formula 4.4

f_m= design bending strength, in appropriate cases reduced on account of bending of laminates (see 4.7)

M_{crit}= buckling moment, calculated in accordance with elasticity theory and with moduli of elasticity and shear for carrying capacity calculations.

5.3.4.3 Contact pressure

Compressive forces are often transferred by direct contact. If the direction of the force is *parallel* with the fibres, the whole strength f_{c0} can usually be utilised. In cases of end to end pressure there is however a risk of penetration of the contact surfaces, in roughly the same way as when two scrubbing brushes are pressed together. To prevent this it can be suitable to design the connection with an insert of hardboard, steel sheet or similar. Otherwise the critical value should be reduced to $0{,}6f_c$.

In local pressure *perpendicular* to the fibres the non-loaded parts prevent the loaded parts being pressed together. If the loaded length (measured in the direction of the fibres) is short it is therefore possible under certain circumstances to base calculations on an effective loaded length l_{eff} which is longer than the real one. Some regulations increase the value of the compression strength instead.

For a glulam member under compression perpendicular to the grain, the following condition shall be satisfied:

$$\sigma_{c,90,d} \leq k_{c90} \cdot f_{c,90,d}$$

Formula 4.11

where

$$k_{c,90} = 1 + \frac{a \cdot (150 - l)}{17000} \leq 1{,}8$$

Formula 4.12

a is the distance in mm from the end of the beam to the nearest edge of the support. The value must not exceed 100.

l is the support length in mm. The value must not exceed 150. (See figure 4.4 below).

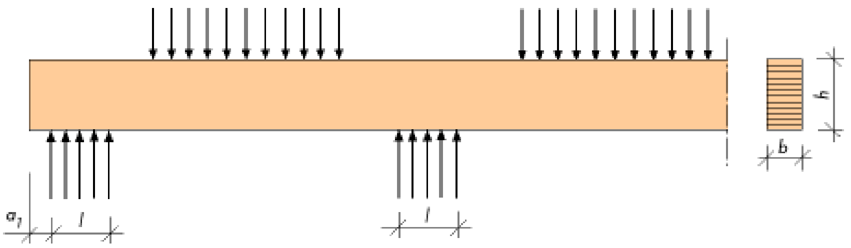


Figure 4.4 Beam on supports.

If the contact pressure is applied at an acute angle to the grain, e.g. as in figure 4.5, the strength can be determined by using the following formula (usually called Hankinson's formula):

$$f_{c,\alpha} = \frac{f_{c,0}}{\frac{f_{c,0}}{f_{c,90}} \sin^2 \alpha + \cos^2 \alpha}$$

Formula 4.13

where

α is the acute angle between the direction of the force and the direction of the grain.

Which values of $f_{c,0}$ and $f_{c,90}$ are to be used depends on the layup of the cross-section, i.e. if it is homogeneous or combined (see 1.3.1) and in the latter   and most common   case also the size of the contact surface and its position in the cross-section. For combined glulam the material values of the next lower strength class shall be used if the contact pressure is mainly in the middle two thirds of the beam, where the timber in the laminates is of somewhat lower quality. In direct contact between the ends of two components the strength parallel to the grain is reduced to $0{,}6f_{c,0}$ according to the above.

Fixtures and supports should be designed so that load-carrying fibres are supported by nearby, non-loaded fibres. Contact pressure must not be applied too near the free edge, specially at sharp corners, see figure 4.5.

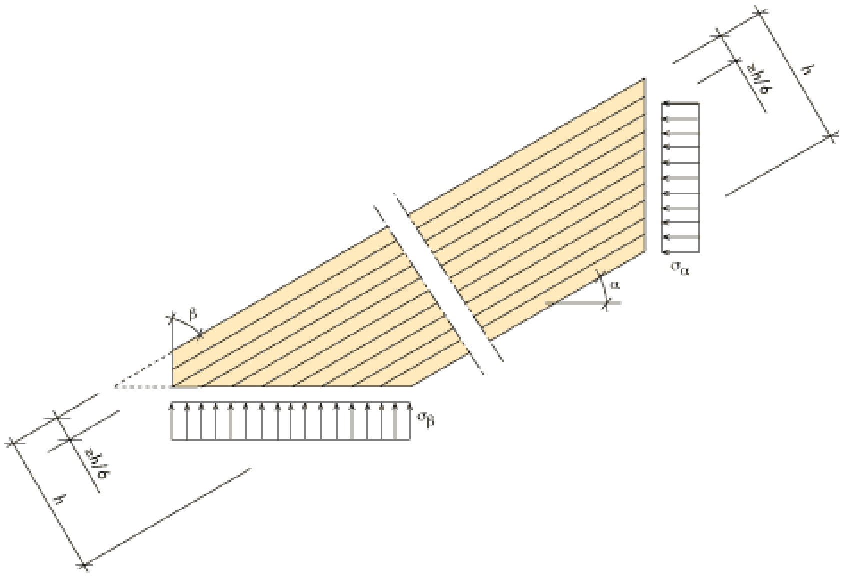


Figure 4.5. Contact pressure at sharp corners.

5.4.4.4 Notches in beam ends

Notches in beam ends should be avoided, as even small notches form the embryo of a potential split which reduces the carrying capacity considerably. Special care should be taken with structures where there is a risk of major variations in the moisture content (see also 4.8 and 4.9). If notches cannot be avoided they should, at least at the bottom, be chamfered (see figure 4.6b). All surfaces in a notch shall be surface-treated. Right angled notches should be given a 25 mm radius curve. Larger notches than 0,5h or 500 mm should not be allowed without reinforcement.

5.5.4.5 Holes

Large holes mean sudden changes in the cross-section which impede the flow of forces in a structure. This leads to noticeable lateral tension and shear stresses near the hole. Together with the drying fissures which naturally occur as a result of end timber being revealed in the sides of the hole these extra stresses can seriously reduce the carrying capacity of the component. Special care is necessary with such types as double pitched beams and monopitch beams where the geometrical form itself causes lateral tension stresses. In curved structural members such as frame haunches and pitched cambered beams holes should not be permitted at all. On account of the risk of splitting holes are not permitted in outdoor structures or elsewhere in places where there is a risk of large variations in the moisture content.

If it is impossible to avoid holes they should be placed centrally in the neutral zone (see figure 4.10) though a deviation of up to 0.1$h$is acceptable. Holes where h_0> 0,5$h$or a> 3h_0 are not permitted. The distance between the edges of two holes shall at least be equal to the depth of the beam h. Rectangular holes shall have a radius of at least 25 mm in the corners. The sides of the hole shall be surface treated to reduce variation in the moisture content and to reduce the risk of splitting. Hot pipes and ducts passing through holes shall be insulated.

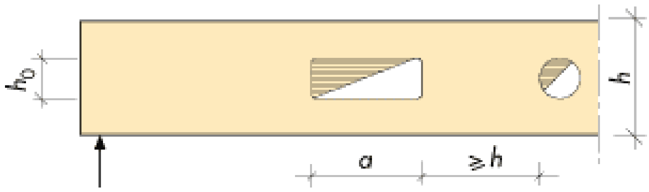


Figure 4.10. Holes in glulam beams. Symbols.

5.6.4.6 Glulam with tapered laminates

Structural glulam components are often designed with a varying cross-sectional depth, e.g. symmetrical double pitched beams, continuous beams with a deeper section over the intermediate supports, or frames.

As a rule, variations in the cross-section are achieved by tapering the laminates along one edge whilst those on the opposite edge are parallel. Since the bending stresses along the tapered edge are parallel with the edge and thus form an angle $\alpha$with the grain, the bending strength is reduced. The distribution of the bending stresses also differs compared with a beam of constant sectional depth (see below under double pitched beam and pitched cambered beam). The reduction in strength differs depending on whether the taper is on the compression or the tension edge. Design bending strength is obtained by multiplying the bending strength parallel with the grain f_m, by a reduction factor $k_k\alpha$/sub>.

$$k_\alpha = \frac{1}{\frac{f_m}{f_{90}} \sin^2 \alpha + \cos^2 \alpha}$$

Formula 4.28

where

f_{90}= $f_{t,90}$if the taper is on the tension side and

f_{90}= $f_{c,90}$if the taper is on the compression side.

Taper with an angle exceeding 15° on the compression side or 5° on the tension side should be avoided. To reduce the risk of splitting, a cover board can be glue nailed to the tapered surface. This arrangement is recommended for all components in climate class 2 and 3 and for components in climate class 1 if the angle of taper exceeds 7°.

5.7.4.7 Glulam with curved laminates

Amongst the most prominent advantages of glulam is the possibility of designing curved structural members. During manufacture, the individual laminates are bent to the desired form before the glue has hardened, which means that certain inherent stresses are built into the material. The creep characteristics of timber mean, however, that these are very largely evened out during gluing, which is of course carried out with added heat and moisture. These inherent stresses can therefore usually be ignored in designing. The effect on bending strength of small bending radii must however be taken into account. This can be done by multiplying the basic value of bending strength by a reduction factor k_r

$$k_r = \begin{cases} 1{,}0 & \text{for } r_m/t \geq 240 \\ 0{,}76 + 0{,}001 \cdot r_m/t & \text{for } r_m/t < 240 \end{cases}$$

Formula 4.29

where

t= thickness of the laminate

r_m= average radius of curvature of the glulam component.

According to EN 386 (Timber structures Glulam Functional and Production Requirements) the maximum permitted laminate thickness (t) in curved structural components is

$$\max t = \frac{r}{250}\left(1 + \frac{f_{m,k}}{80}\right)$$

Formula 4.30

where

r= radius of the curvature in the laminate (mm)

$f_{m,k}$= characteristic bending strength of laminate joints (N/mm^2)

In the Nordic countries. glulam is usually manufactured using 40 - 45 mm thick laminates, which corresponds to a minimum bending radius of about 7 m for strength class GL 32. If the bending radius is reduced, the laminate thickness must also be reduced, which increases the cost.

Tension stresses perpendicular to the grain often arise in curved structural members. Tension strength is treated below and calculation of stresses in the section on pitched cambered beams and curved beams.

5.8.4.8 Tension strength perpendicular to the grain

Timber has its lowest strength in tension perpendicular to the grain. It is affected to a high degree by fissures, knots and growth features, specially in early wood, the light, softer part of annual rings. Tension strength perpendicular to the grain is utilised only in areas where stresses arise as a result of secondary effects, e.g. at holes and notches, within the ridge zone of double pitched beams and in curved, moment-loaded structural members.

Various studies have shown that tension strength perpendicular to the grain is highly dependent on the stressed volume of the timber, i.e. on type of loading and geometrical form of a structure (see also 4.1 Size effect). The basic value given in the regulations must therefore be modified, e.g. by multiplying it by a modification factor k_{vol}.

$$k_{vol} = k_{dis}\left(\frac{V_0}{V}\right)^{\frac{1}{m}}$$

Formula 4.31

where

m= constant based on testing. In EC 5, m= 5

V_0= reference volume. In EC 5, V_0= 0,01 m3

V= characteristic volume determined with regard to the geometry of the member

k_{dis}= modification factor with regard to stress distribution in the beam

Values of $k_{dis}$and $V$for evenly loaded beams as in figure 4.15 can be taken from table 4.4. Vneed however not be taken as more than 2/3V_b, where $V_b$is the total volume of the beam.

For calculation of tension stresses perpendicular to the grain, see section on double pitched and curved cambered beams.

Beam type	k_{dis}	V
Double pitched	1,4	$b(h_{apex})^2$
Pitch cambered	1,7	Curved part as in fig 4.15
Curved	1,4	As in fig 4.15

Table 4.4

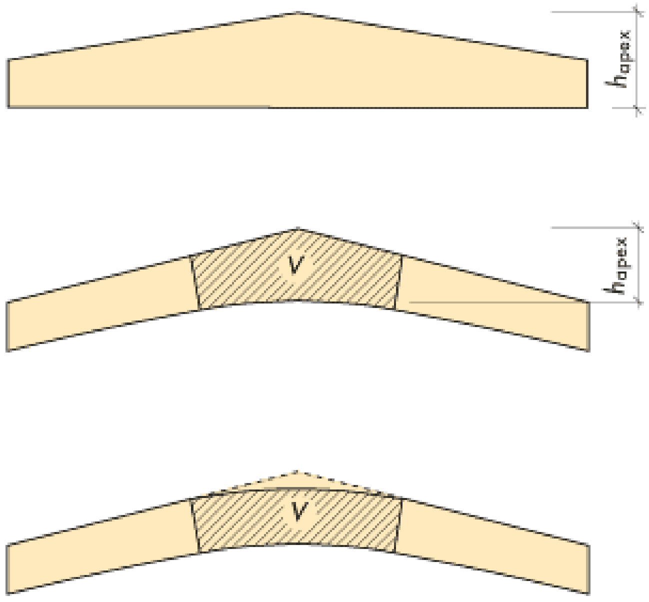


Figure 4.15 Double tapered beam, pitched cambered beam and curved beam with constant depth.

5.9.4.9 Moisture movement

Glulam components are normally supplied at a moisture content of 12%, which roughly corresponds to the equilibrium moisture content in service class 1. Under different climatic conditions the moisture content will in time adjust itself to the surrounding relative vapour pressure and to the temperature. The average year-round moisture contents for glulam structures in various service classes is approximately

- 12% in service class 1
- 16% in service class 2

No average value can be given for structures in service class 3.

As a result of seasonal changes in the climate, the moisture content in a structure will vary endlessly. The variation is 4 - 5% units for indoor members and 8 - 10% units for outdoor members. Timber indoors is usually driest in the winter, while outdoor structures are driest in the summer.

Glulam, like other timber material, swells when the moisture content increases and shrinks when it decreases. Movement is many times greater perpendicular to the grain than parallel, 0,2% and 0,01% respectively for each per cent of the change in moisture content. Experience has shown that maximum moisture movement of glulam in service classes 1 and 2 is as follows:

- Perpendicular to the grain approx 10 mm/m
- Parallel with the grain approx 0,5 mm/m

If moisture movement perpendicular to the grain is restrained due to internal or external constraints the tension strength perpendicular to the grain can be exceeded, causing the timber to be crushed or to split. Fixtures and connections must therefore be designed so that normal moisture movement is restrained as little as possible. One should also be aware that stiffness and carrying capacity in bolted connections are impaired if they are not properly tightened. If possible, at least the vital bolted connections should be tightened again when the timber has dried out.

Length changes are in general so small that they can be disregarded, except in very large structures. Details where the moisture content is unevenly distributed across the cross-section, e.g. beams and columns within a layer of insulation, can suffer from considerable deformation due to differing moisture movement on the cold and on the warm sides. During the winter one side is in a warm and dry climate while the other is in contact with the outdoor air, whose moisture content is high. The outside swells and becomes longer than the inside, which is the reason why roofs and outside walls tend to bend outwards during the winter. With a pinned support and unrestrained moisture movement the outward bending can be calculated using the following formula

$$u = \frac{l \cdot \Delta l}{8h}$$

Formula 4.32

where

l = span

Δl = difference in length between outer face and inner face due to swelling or shrinking

h = depth of the member.

6.1.5.1 Compression due to axial loading

Axially loaded glulam components shall satisfy the condition

$$\sigma_c = \frac{N}{A} \leq k_c f_c$$

Formula 5.1

where

k_c= a reduction factor taking into account the risk of buckling in the plane

f_c= critical compression stress

A= total cross-sectional area of column

The reduction factor $k_c$is determined with respect to the slenderness ratio $\lambda$of the column or strut $\lambda = l_c / i$where $l_c$is the buckling length and $i=(I/A)^{0,5}$its radius of gyration in the direction of buckling. For rectangular columns the following applies

$$\lambda = \frac{l_c \sqrt{12}}{d}$$

Formula 5.2

where

l_c= the buckling length of the column in the direction of buckling

d= the width of the column in the direction of buckling

Slenderness is often expressed as a relative slenderness ratio

$$\lambda_{rel} = \sqrt{\frac{f_{c,k}}{\sigma_{c,crit}}} = \frac{\lambda}{\pi} \sqrt{\frac{f_{c,k}}{E_{0,05}}}$$

Formula 5.3

where

$f_{c,k}$= characteristic short-term value of compression strength in the direction of the grain

$\sigma_{c,\,crit}$= theoretical buckling stress

$E_{0,05}$= characteristic short-term value of the modulus of elasticity in the direction of the grain

Note that the modulus of elasticity shall be inserted with the value used when calculating carrying capacity, which generally differs from the value used for calculating deflections.

Slenderness usually differs in different directions, since stiffness in bending and fixing conditions are different. For columns with a constant cross-section the theoretical buckling length lc in relation to the actual column height L with various types of fixity can be seen in table 5.1. The various types of fixity   Euler´s four cases of buckling   are shown in figure 5.1.

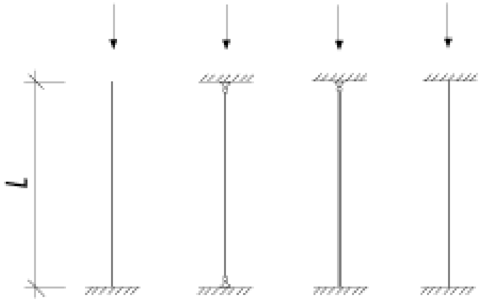


Figure 5.1 Various end conditions in buckling

Case of buckling	1	2	3	4
Theoretical buckling length	2,0	1,0	0,7	0,5
Buckling length in practice	2,25	1,0	0,85	0,7

Table 5.1 Theoretical and practically applicable buckling lengths in various end conditions.

Rigid fixity can never be achieved in practice due to deformations in steel fixtures, anchor bolts etc. Where fixing of glulam columns is carried out using mechanical timber connections, the values in the lower line of table 5.1 should be used.

For columns with a linear variation of the cross-sectional depth, the buckling length can be determined with the aid of figure 5.2.

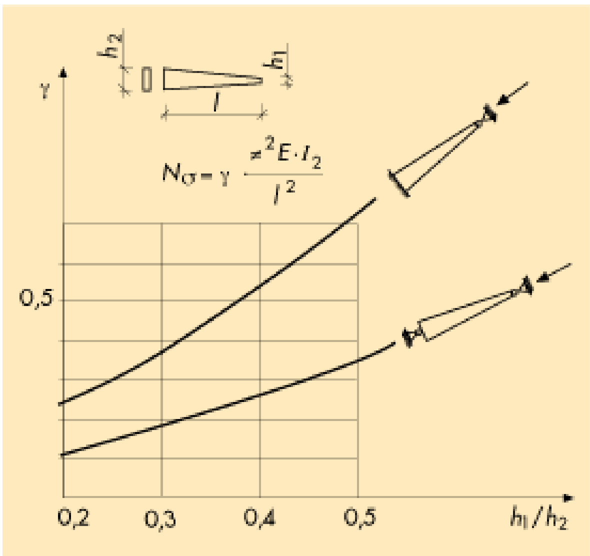


Figure 5.2 Diagram for determining the buckling load of columns whose cross-sectional depth has a linear variation.

Columns in external walls are often designed as being restrained from buckling in the weak direction (which means that fixing of studs and the wall construction in general must be checked for bracing forces, see chapter 12) while internal columns are normally unrestrained in their whole length.

At the top and bottom of columns, and at other points where the cross-section is weakened by screw holes etc, it must be shown that

$$\sigma_c = \frac{N}{A_{net}} \leq f_c$$

Formula 5.4

where

A_{net}= the net cross-sectional area of the column

The connection between the reduction factor $k_c$and the relative slenderness ratio $\lambda_{rel}$is given in the structural regulations and differs insignificantly between different codes. In EC 5 the following applies:

$$k_c = \begin{cases} 1 & \text{for } \lambda_{rel} \leq 0{,}5 \\ \dfrac{1}{k + \sqrt{k^2 - \lambda_{rel}^2}} & \text{for } \lambda_{rel} > 0{,}5 \end{cases}$$

Formula 5.5

where

$$k = 0{,}5\left(1 + 0{,}1\left(\lambda_{rel} - 0{,}5\right) + \lambda_{rel}^2\right)$$

Formula 5.6

The connection according to EC 5 is shown in figure 5.3:

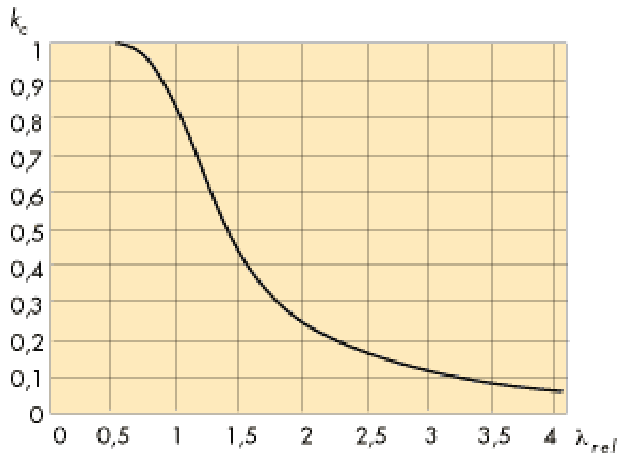


Figure 5.3 Reduction factor $k_c$as a function of the relative slenderness ratio according to EC5.

6.2.5.2 Combined bending and compression

Design conditions where compression and bending occur at the same time usually take the form of interaction conditions , which according to EC 5 are:

$$\left(\frac{\sigma_c}{k_c f_c}\right)^n + \frac{\sigma_{m,y}}{f_{m,y}} + k_m \frac{\sigma_{m,z}}{f_{m,z}} \leq 1$$

Formula 5.7

$$\left(\frac{\sigma_c}{k_c f_c}\right)^n + k_m \frac{\sigma_{m,y}}{f_{m,y}} + \frac{\sigma_{m,z}}{f_{m,z}} \leq 1$$

Formula 5.8

where

n= 2 if there is no risk of buckling ($\lambda_{rel} \leq 0,5$), n= 1 in other cases ($\lambda_{rel} > 0,5$)

σ_c= normal stress due to axial loading

σ_{my},σ_{mz}= bending stress due to lateral loading in y- and z-directions

f_c, f_{my}, f_{mz}= design compression and bending strengths respectively

k_c= reduction factor with regard to buckling (according to 5.1)

k_m= 0,7 for rectangular sections, 1,0 for other cross-sections

The reduction factor $k_c$is determined from the slenderness ratio for buckling in the least favourable direction, regardless of the direction

in which the moment acts. Thus, columns in external walls restrained by horizontal studs it must be checked if the slenderness in lateral buckling between the studs is greater than in buckling outwards from the wall.

6.3.5.3 Mechanically jointed columns

Since the maximum available width of individual glulam components is normally about 200 mm, slenderness in the weak axis of the column is often greater than λ= 170, which is the usual maximum in normal building practice. To overcome this the column can be made up of several shafts (e.g. see figure 5.4) connected by nails, bolts, glue or a combination of them.

If the column is continuously glued, full interaction can be assumed and the column designed as in the previous section. Otherwise it is necessary to take into account that the shafts move in relation to one another when the column is loaded. Bolted connections always involve considerable movement and must always be combined with connectors of indentation type, e.g. toothed indentation connectors. The design of a continuously bolted column is described below. Design rules for other types of composite columns are given in EC 5.

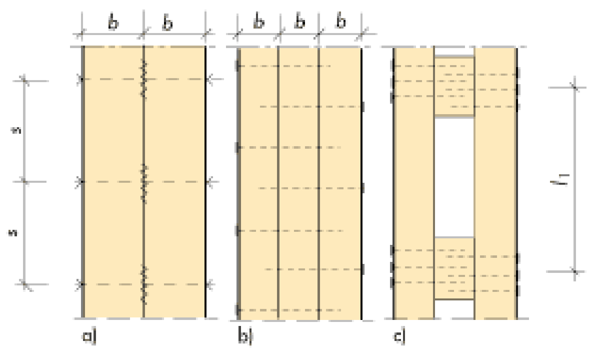


Figure 5.4 Section through mechanically jointed column. a) Continuously bolted. b) Continuously nailed. c) With spacers.

6.4.5.4 Column systems

Single storey hall buildings are often designed as simple systems of columns and beams where one or more columns are rigidly fixed to the foundations and brace the other columns which are designed as columns pinned at both ends (see figure 5.5).

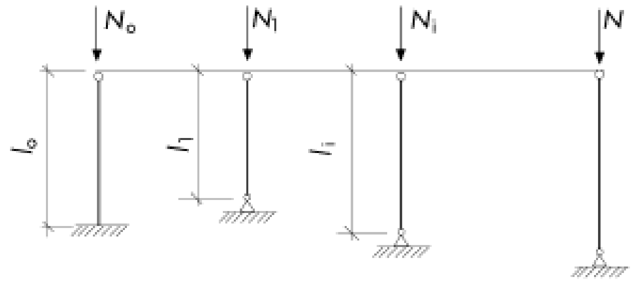


Figure 5.5 Column system with one rigidly fixed column.

7.1.6.1 Straight beams of constant depth

7.2.6.2 Straight beams with linear depth variation

Beams whose depth varies occur e.g. as symmetrical or, exceptionally, unsymmetrical double tapered beams, single tapered beams and roof beams in frame structures. Double tapered and single tapered beams usually rest freely on two supports.

In the text which follows it is assumed that the grain of the timber is parallel with the underside of the beam.

7.3.6.3 Curved beams

Glulam beams are often manufactured with curved forms. The sectional depth within the curved part can vary or be constant (see figure 6.9). If the depth is constant, the desired pitched roof form can be achieved with a separate ridge piece which is nailed or screwed to the beam afterwards.

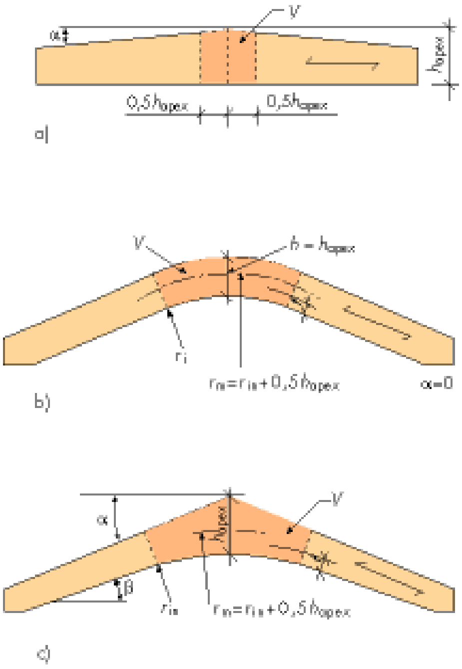


Figure 6.7. Curved beams with constant and varying depths. The characteristic volume V is marked in the figure.

When using curved beams it is important to note that at least one of the supports permits horizontal movement, since the functioning of the structure, and thus the design conditions, will otherwise be radically altered.

Steeper roof slopes than 10° should be avoided.

For the straight parts of the beam, the same design conditions apply regarding shear forces, bearing strength at supports and bending as for straight beams with constant or varying depth (see 6.1 and 6.2). For the curved part of the beam, bending stresses and tension stresses perpendicular to the grain must be checked. Design bending strength must here be reduced with regard to the laminate curvature (see 4.7)

7.4.6.4 Design diagrams

Straight beams supported at two points can be designed with the aid of the diagrams in figure 6.10 (carrying capacity) and 6.11 (stiffness).

Starter values in the diagrams are obtained by calculating the load per metre of beam (kN/m) and dividing this value with the width of the beam (m) and by the design value (N/mm^2) of the bending strength (figure 6.10) or the module of elasticity in bending (figure 6.11). The design value of bending strength is determined with regard to service class, duration of load and in appropriate cases also safety class in accordance with the regulations applying in the individual case (see e.g. Annex 2). In diagram 6.11, the design value of the modulus of elasticity which is used shall be that which is used for calculating deformations. This is normally the mean value.

Key to the diagrams below

1. Determine the load combination and the design value of the load (q_d)

2. Determine the span.

3. Choose type of beam (straight, double tapered, single tapered)

4. Choose starter values for beam depth and width that seem reasonable, e.g. using table 2.1.

5. Determine the design bending strength (f_{md}) with regard to the chosen depth, (safety class), service class and duration of the shortest-time loading in the loading combination. See e.g. Annex 2a.

6. Choose a starter value of $q_d/b/f_{md}$ using the units $kN/m/m/N/mm^2$ and find the value on the horizontal axis in diagram 6.10.

7. Draw a vertical line through this point and mark where the line crosses the curve corresponding to the chosen type of beam (A, B or C).

8. Draw a horizontal line through the intersection and read the value of the quotient h/l where the line crosses the vertical axis.

9. Calculate the beam depth h and compare it with the starter value.

10. Correct, if necessary, the starter value and repeat points 6 to 9 until the values of assumed and calculated beam depth agree.

11. Determine in the same way required beam depth with regard to deflection limits. Diagram 6.11 is based on the requirement that the deflection at mid-span shall not exceed 1/200 of the span.

Calculate the starter value $q_d/b/E$ with the loading combination and the value of the Modulus of Elasticity (E) which apply when designing in the serviceability limit state. See e.g. Annex 2b for design values of the Modulus of Elasticity.

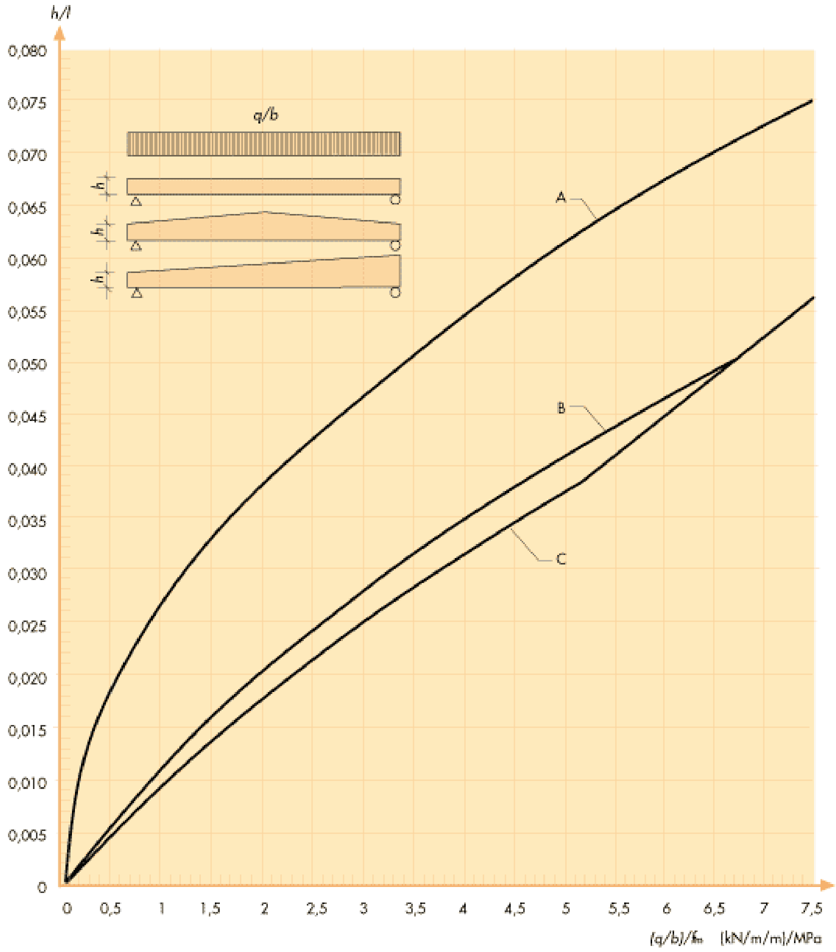


Figure 6.10 Diagram for design of beams supported at two points with regard to carrying capacity. Straight, double tapered and single tapered beams. The beams are assumed to be restrained against lateral buckling. Curve A = straight beam, Curve B = double tapered or single tapered beam 1:20, Curve C = double tapered or single tapered beam 1:16

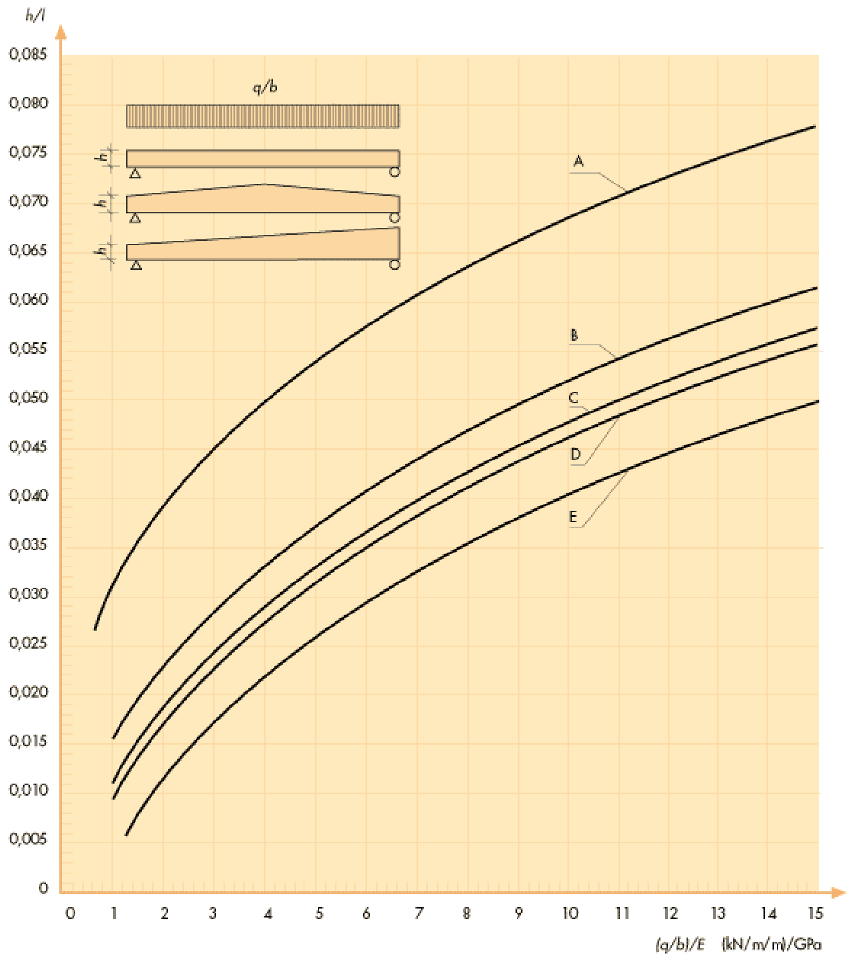


Figure 6.11 Diagram for design of beams supported at two points with regard to deflection in the serviceability limit state. Straight beams, double tapered beams and single tapered beams. The beams are assumed to be restrained against lateral buckling. Design conditions: max deflection w = l/200. For other conditions e.g. w = l/n, multiply load q by n/200. Curve A = straight beam, Curve B = double tapered beam 1:20, Curve C = double tapered beam 1:16, Curve D = single tapered beam 1:20, Curve E = single tapered beam 1:16.

8.1.7.1 Geometry

Timber trusses for short spans are usually triangular with stress graded timber members and nailplates at nodes. However, for longer spans glulam is the dominating material. A common glulam structure, which can be regarded as a simple truss, is the three-pin truss. This is dealt with in its own chapter, 8 Three-pin trusses. Other glulam trusses are often designed as parallel frames, though other forms such as trapezoid or bow frames also occur (see figure 7.1).

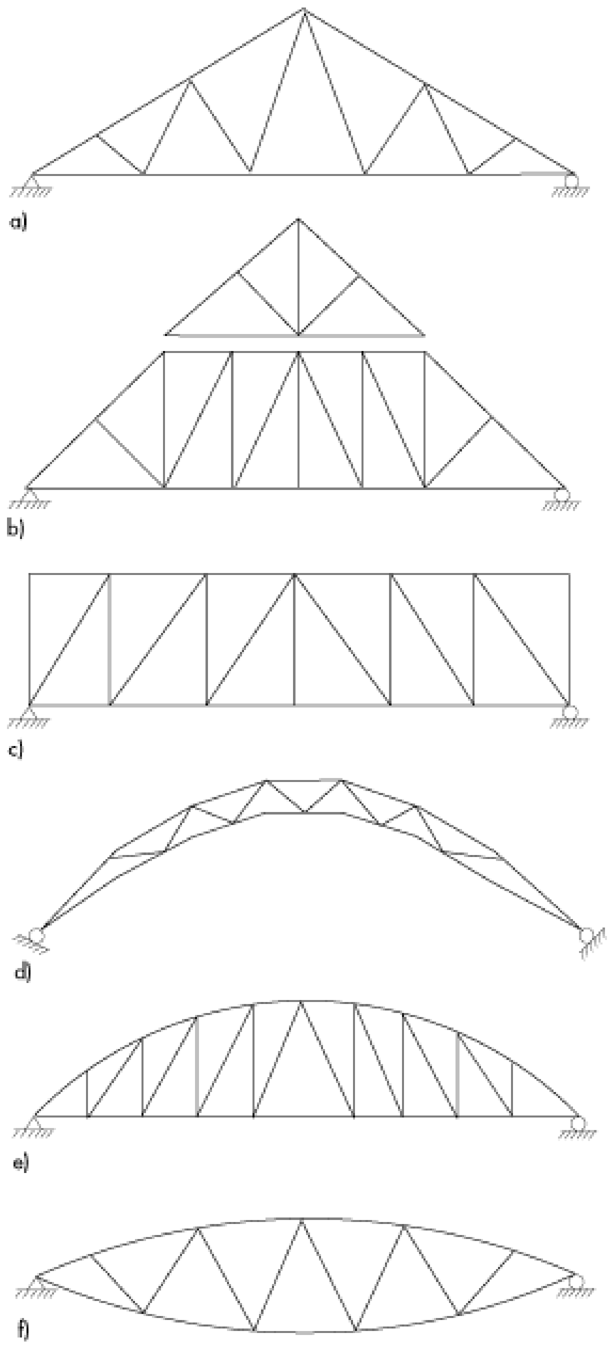


Figure 7.1 Types of truss. a)Triangular, b) Ditto, in parts for transport, c) Parallel, d) Two hinge arch, e) Bowstring, f) Fish belly

The top chord usually follows the shape of the roof while the shape of the bottom member is determined by requirements on free headroom in the building, the necessary constructional depth, the wishes of the architect, and the design of any false ceiling, etc.

The top and bottom chords are connected by intermediate vertical and/or diagonal members, forming a series of triangles. The truss is thus geometrically stable. It is also statically determinate, i.e. the forces in the members can be calculated using equilibrium equations, if the number of members n fulfils the condition

$$n = 2m - r$$

Formula 7.1

where

m= number of joints

r = number of external support reactions.

For a truss simply supported at two points, carrying horizontal and vertical loads, r= 3 and the condition can be expressed as

n= 2m- 3

If the number of members is greater, the truss is statically indeterminate and must be calculated with regard both to equilibrium and deformation conditions. If the number of members is less, the truss is geometrically unstable and unsuitable for carrying loads.

The intermediate members are also designed so that:

- the nodes coincide if possible with the points where external loads act, thus avoiding moments in the members forming the top chord.
- services such as ventilation ducts can pass through
- the truss has a harmonious appearance, e.g. symmetry, diagonals at the same angle
- members in compression are as short as possible so that they do not need to be braced
- the truss can be divided into sections for transport.

The construction depth should be adequate. About 1/12 of the span is suitable in parallel trusses if the forces in the members are to be kept within reasonable limits

8.2.7.2 Calculation of forces in members

Timber trusses can be calculated as elastic frame structures. Stiffness and deformation in members and joints shall be taken into account, together with the effects of eccentricity at joints and at supports. Calculations are best carried out by computer.

EC 5 lays down the following guiding principles as the basis of a choice of calculation model:

- The truss shall be designed as a system of beam elements placed along system lines and connected at nodes (as in figure 7.2), provided a more general model is not used.

- System lines shall lie within the cross-section of the members in their entire length. Within the external members, normally the top and bottom chords, the system lines shall coincide with the centres of gravity of the members.

- Fictitious members may be used to design eccentric joints or supports (see figure 7.2). The direction of a fictitious member should as far as possible coincide with the direction of the force. Fictitious members shall be assumed to have the same stiffness as connecting members.

- In the calculations it is permitted to disregard the effect of non-linear behaviour (buckling) in compression members if this is taken into account in the design of the individual member.

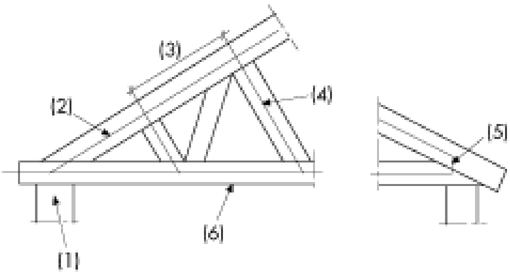


Figure 7.2 Example of a static truss model

(1) Support (2) System line

(3) Bay (4) Internal member

(5) Fictitious member (6) External membere

8.3.7.3 Design of members

If the truss is only loaded at nodes, e.g. by purlins, the members can usually be designed taking into account only normal forces and any eccentric moments. Members which are loaded between nodes must also be designed for moments.

The members can be single or composite. Double top and bottom chords are often combined with single diagonals. Glulam compression members are sometimes combined with tension members of steel (compare e.g. 8.2).

Tension members shall be designed with regard to cross-sectional reductions for bolt holes etc. The space needs of connectors etc at nodes are often critical for the sizes of members.

Compression members shall be designed according to chapter 5, taking into account the risk of buckling both in the plane of the truss and outwards. The buckling length is normally the length between two adjacent inflexion points. For members between two hinged nodes and for continuous members without lateral loading, this means that the buckling length equals the distance between adjacent nodes. To reduce the buckling length (if necessary) it is possible to brace the members in, or perpendicular to, the plane of the truss. The members forming the top chord often have natural bracing in the form of purlins or roof boarding.

8.4.7.4 Nodes

The joints at nodes are generally bolted with ring or toothed-plate connectors. Exceptionally, glued nodes are also used. When spans are large, the forces in the members are often so large that the transfer of forces at nodes is best achieved using steel plate connectors. Figure 7.3 shows an example of a node with slotted-in plates and steel dowels. This system has been successfully used in Norway, e.g. in the Olympic Halls in Hamar and Lillehammer, and at Gardemoen Airport outside Oslo.

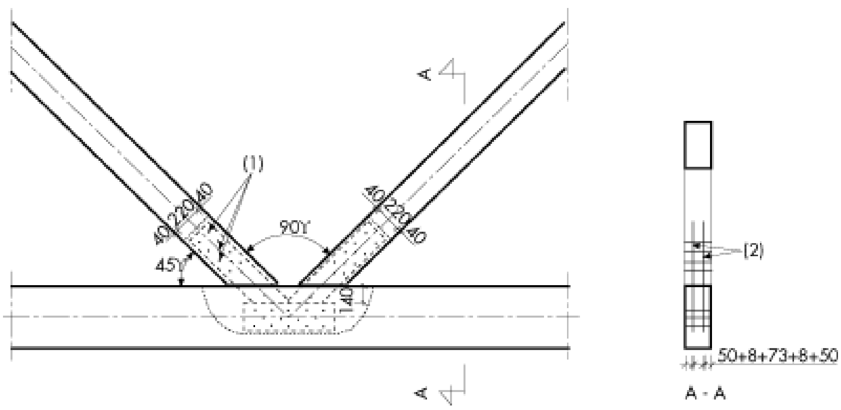


Figure 7.3 Example of design of node in a long-span truss

(1) 12 mm steel dowels

(2) Slotted-in steel plates

8.5.7.5 Deflection and camber

The pliability of the joints means that trusses are subject to larger deflections than solid beams. They should therefore always be cambered (both the top and bottom chords), approximately *l*/150 where *l*/*h*= 12 and *l*/200 where *l*/*h*= 10.

An assessment of the deflection in a parallel truss, without regard to deformations in the joints, can be obtained by calculating the deflection of a solid beam with a moment of resistance:

$$I = \Sigma A_i \cdot a_i^2$$

Formula 7.2

where

A_i= cross-sectional area of the external members

a_i= distance between the system line of the external

members and centre of gravity of the truss.

The real deflection is however considerably greater. An accurate calculation can be made using a computer.

9.1.8.1 Three-pin truss with straight roof beams

This type of structure consists of two straight glulam beams leaning against each other and with a hinged connection at the ridge. The lower ends of the two members are connected by a tension member or hinged to an immovable foundation. The roof slope should not be less than 14° with regard to the magnitude of the horizontal forces and to movement at the ridge and the supports.

The glulam beams are designed as struts subject to compression and bending in accordance with 5.2, and the risk of buckling must be taken into account. In general. lateral buckling can be considered as restrained by purlins, roof boarding or other secondary structures which in this case must be designed for bracing forces (see chapter 12). In buckling perpendicular to the plane of the roof, the buckling length is

$$l_c = 0{,}5l/\cos\alpha$$

At the ridge and supports, shear stresses and local compression must be checked, noting that the forces act diagonally to the grain (see 4.3).

The tension member can at spans up to 25 m be glulam but is usually steel. In heated spaces (over +5°C) high tension steel, e.g. Dywidag, can be used. Often, however, the opportunity of using the high strength are limited by the deformation caused by stretching of the tension member.

The connection with the glulam beams is designed to take into account the fact that the forces act at an angle to the grain.

The ridge joint is designed as a hinge free of moments and is designed for maximum horizontal compression. This has the same magnitude as the opposing force in the tension member. In loading combinations where the external load differs between the two halves of the roof there is also a shear force in the ridge. According to most loading regulations this is of interest only if the roof slope exceeds 15°. It is however always advisable to design the ridge connection so that some shear force can be transmitted, e.g. corresponding to half of a one-sided snow load which can occur during snow clearance.

9.2.8.1.1 Internal moments and forces. Support reactions.

In uniformly distributed, downward loading and with symbols as in figure 8.1, the reactions at supports and the internal moments and forces can be calculated using the following expressions:

Vertical support reaction:

$$R_1 = \frac{(3q_1 + q_2) \cdot l}{8}$$

Formula 8.1

$$R_2 = \frac{(q_1 + 3q_2) \cdot l}{8}$$

Formula 8.2

Force in the tension member:

$$H = \frac{(q_1 + q_2) \cdot l^2}{16f}$$

Formula 8.3

Maximum moment in roof beam:

$$M = \frac{q_1 l^2}{32}$$

Formula 8.4

Adherent longitudinal force:

$$N_1 = -\frac{(q_1 + q_2) \cdot l}{8 \sin \alpha}$$

Formula 8.5

Maximum longitudinal force in beam:

$$N_2 = N_1 - \frac{q_1 l}{4} \sin \alpha$$

Formula 8.6

Maximum shear force in beam:

$$V_1 = \frac{q_1 l}{4} \cos \alpha$$

Formula 8.7

Shear force in ridge (vertical):

$$V_2 = -\frac{(q_1 - q_2) \cdot l}{8}$$

Formula 8.8

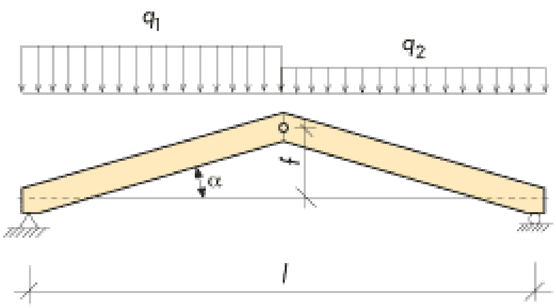


Figure 8.1 Three-pin truss with simple roof beams and tension member. Downward load.

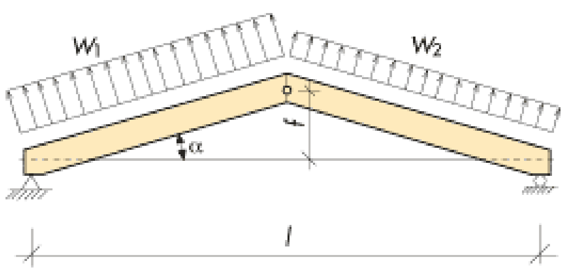


Figure 8.2 Three-pin truss with simple roof beams and tension member. Upward load.

Under uniformly distributed upward loads of different magnitude on the two roof halves as in figure 8.2, e.g. under wind suction, the reactions at supports and the internal moments and forces can be calculated using the following expressions:

Vertical (downward) support reaction:

$$R_1 = \frac{(3w_1 + w_2) \cdot l}{8} - \frac{(w_1 - w_2) \cdot l}{8} \tan^2 \alpha$$

Formula 8.9

$$R_2 = \frac{(w_1 + 3w_2) \cdot l}{8} + \frac{(w_1 - w_2) \cdot l}{8} \tan^2 \alpha$$

Formula 8.10

The horizontal support reaction is distributed equally between the two supports:

$$H_1 = H_2 = \frac{(w_1 - w_2) \cdot f}{2}$$

Formula 8.11

The (compression) force in the tension member is then:

$$H = \frac{w_1 + w_2}{2} \cdot \frac{l^2}{8f} \left(1 - \tan^2 \alpha\right)$$

Formula 8.12

Maximum moment in roof beam:

$$M = -\frac{w_1 l^2}{32 \cos^2 \alpha}$$

Formula 8.13

Longitudinal force in roof beam:

$$N = R_1 \sin \alpha + (H - H_1) \cos \alpha$$

Formula 8.14

Maximum shear force in roof beam:

$$V = R_1 \cos \alpha - (H - H_1) \sin \alpha$$

Formula 8.15

9.3.8.1.2 Deformation

Under uniformly distributed downward load as in figure 8.1, the vertical deflection of the ridge can be calculated using the expression:

$$w = \frac{(q_1 + q_2) \cdot l^2}{16 \tan^2 \alpha (E \cdot A)_{brace}} \left(\frac{1}{\cos^3 \alpha} + \frac{(E \cdot A)_{brace}}{(E \cdot A)_{tie}} \right)$$

Formula 8.16

If the supports are immovable, the second term in brackets will be 0.

9.4.8.1.3 Safety against lifting

Three-pin trusses with tension member shall be checked against lifting, e.g. due to wind suction. Since the tension member normally cannot resist compression forces, the structure only works for load combinations which produce tension in this member, i.e.

$\Sigma H \geq 0$

If the above condition is not fulfilled, the (inward) horizontal support reaction must be dealt with by other means, e.g. in wind bracing, roof diaphragm or similar

Alternatively the tie at least if it is of timber can be restrained laterally so that any compression forces can be taken up. The lateral restraint can for example be combined with a false ceiling if such exists.

The truss can also be designed with a rigid joint at the ridge, e.g. by a collar beam placed near the ridge. This makes the structure statically indeterminate, and the expressions given above no longer apply.

Example 8.1

By developing the expression $\Sigma H \geq 0$, a general expression can be derived for the required self weight of three-pin trusses with tension member and different wind loads.

The loading combination self-weight and wind suction as in figure 8.3 is examined, with the following symbols for the partial coefficients:

$\gamma_{f,g}$ for self weight

$\gamma_{f,w}$ for wind load

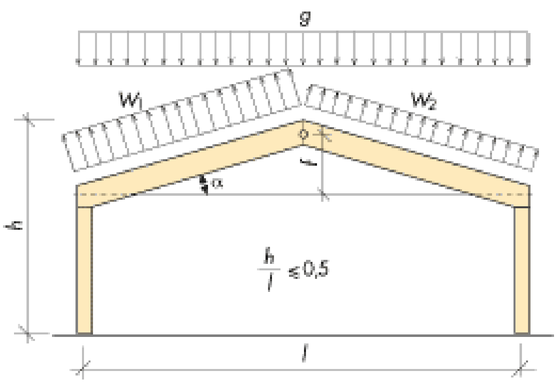


Figure 8.3 Example of calculation. Symbols.

The critical value of the force in the tension member then becomes, according to equations (8.3) and (8.12) and with:

g= self-weight of the structure

$$\Sigma H = \gamma_{f,g} \frac{g \cdot l^2}{8f} - \gamma_{f,w} \frac{w_1 + w_2}{2} \cdot \frac{l^2}{8f} \left(1 - \tan^2 \alpha\right)$$

According to the above, the condition $\Sigma H \geq 0$ should be fulfilled, therefore:

$$g \geq \frac{\gamma_{f,w}}{\gamma_{f,g}} \cdot \frac{w_1 + w_2}{2} \cdot \left(1 - \tan^2 \alpha\right)$$

Formula 8.17

Example 8.2

Determine, using the expression (8.17) derived in the previous example, the self-weight required in a roof structure using trusses with tension member under the following conditions:

- Roof slope 14°
- Characteristic value of wind pressure q_k= 0,6 kN/m2
- Value of partial coefficients 0,85 ($\gamma_{f,g}$) and 1,3 ($\gamma_{f,w}$)
- The pressure coefficient for internal wind load on the roof is and 0,2 (leeward, suction)
- The results of edge effects at the eaves and ridge can be ignored in this connection.

The wind loads on the two halves of the roof are thus:

$$w_1 = (\mu_{indoor} + \mu_{outdoor}) q_k = (0,2 + 0,4)0,6 = 0,36 \text{ kN} / \text{m}^2$$
$$w_2 = (\mu_{indoor} + \mu_{outdoor}) q_k = (0,2 + 0,2)0,6 = 0,24 \text{ kN} / \text{m}^2$$

$$\frac{w_1 + w_2}{2} = \frac{0,36 + 0,24}{2} = 0,30 \text{ kN} / \text{m}^2$$

The condition (8.17) then gives:

$$g \geq \frac{13}{0,85} \cdot 0,30 \cdot \left(1 - \tan^2 \alpha\right) = 0,43 \text{ kN} / \text{m}^2$$

By comparison, it can be mentioned that an insulated steel sheet roof on glulam purlins has a self-weight of 0,35 - 0,45 kN/m2.

9.5.8.2 Three-pin truss with timber struts and steel ties

Over large spans (> 30 m) the roof beams in a three-pin truss can suitably be designed with steel ties and timber struts (see figure 8.4). The moment in the roof beams is reduced by the introduction of one or more intermediate supports, while the longitudinal force increases to the same extent. This type of structure can, unlike the simple three-pin truss, be designed with a lower roof slope than 14°, provided that the angle αbetween the tie and the beam at the support is greater than 14° (see figure 8.4).

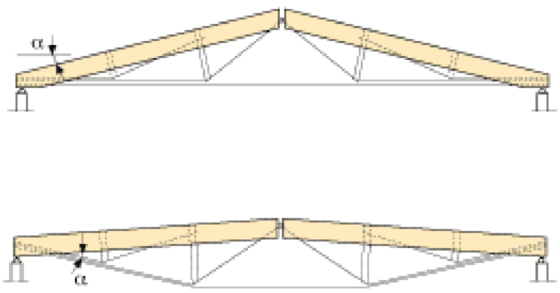


Figure 8.4 Three-pin truss with timber struts and steel ties.

The internal moments and forces in the truss members are calculated as in an ordinary truss (see chapter 7). Consideration must be given to changes of length in the tension members due to loading and temperature changes, and to any deformations at the nodes. Calculations are normally computer-based.

Beams are designed for simultaneous compression and bending. The risk of buckling either vertically or laterally must be taken into account (see 5.2).

The struts are designed as in 5.1. Special attention must be paid to the risk of lateral buckling. For this reason the connection to the roof beams should be capable of resisting moments. The bottom node can possibly be laterally restrained.

The ties should be designed for simple and functional nodes, e.g. by choosing a suitable number of rods and at each strut bending up one or more rods to the next node on the top chord. High-tensile steel can be used in certain cases.

The deflection of the ridge can be estimated by using formula 8.16. The cross sectional area of the tie connecting the supports is used here.

9.6.8.3 Design diagrams

Three-pin trusses can be designed using the diagram in figure 8.5.

The starter values in the diagram $(q/b)/f_{md}$can be obtained by calculating the load per running metre of the beam (kN/m) and dividing it by the beam width (m) and by the design value of the bending strength (N/mm2) under this loading. This is determined with regard to climate class, duration of load and, in appropriate cases, safety class in accordance with the regulations applying to the particular case. See e.g. Annex 2.

The diagram is based on the assumption that the relationship between the critical values of bending strength and compression strength parallel to the grain is 1,1 and that the relationship between the critical values of the Modulus of Elasticity in bending and the compression strength parallel to the grain is 370.

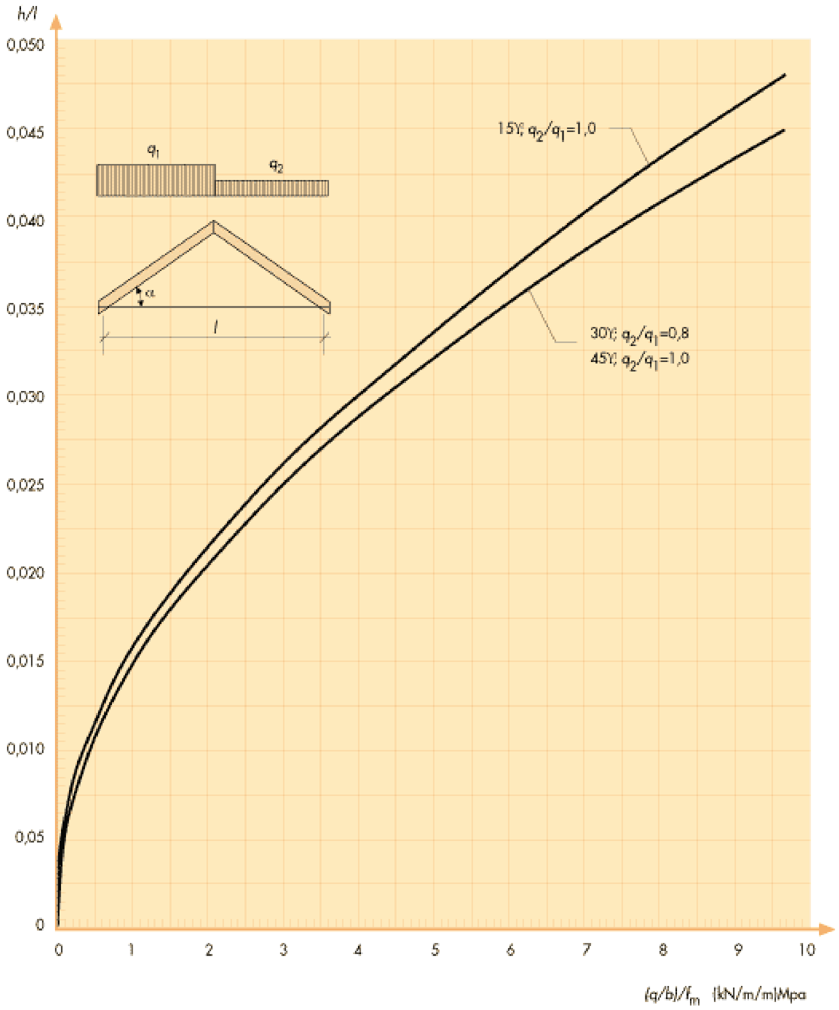


Figure 8.5 Diagram for design of three-pin truss regarding carrying capacity. The roof beams are assumed to be restrained from lateral buckling. Condition: f_{md}/ f_{cd}= 1.1 and E_{md} / f_{cd}= 370

10.1.9.1 Design of three-pin portal frame

The three-pin portal frame is the incomparably commonest type. It is stable against horizontal forces in its own plane and statically determinate, which means that the moment distribution is not affected by uneven subsidence of the foundations or by unforeseen deformations in joints and connections. Further, the three-pin frame is hinged into the foundations, which simplifies their basic construction. In poor soil conditions the horizontal reactions at the supports can be taken up by tension members between the foundations (within or under the slab). The load on the substrate is then principally vertical.

In normal cases, with roof slopes around 15°, the loading consists of self- weight and snow plus possible concentrated loads from hoist blocks and similar which are critical. In connection with steep roof slopes, e.g. in churches or certain types of storage buildings, loading combinations together with wind can be critical.

Calculation of the deformations is best carried out with the aid of a computer and one of the calculation programmes available on the market.

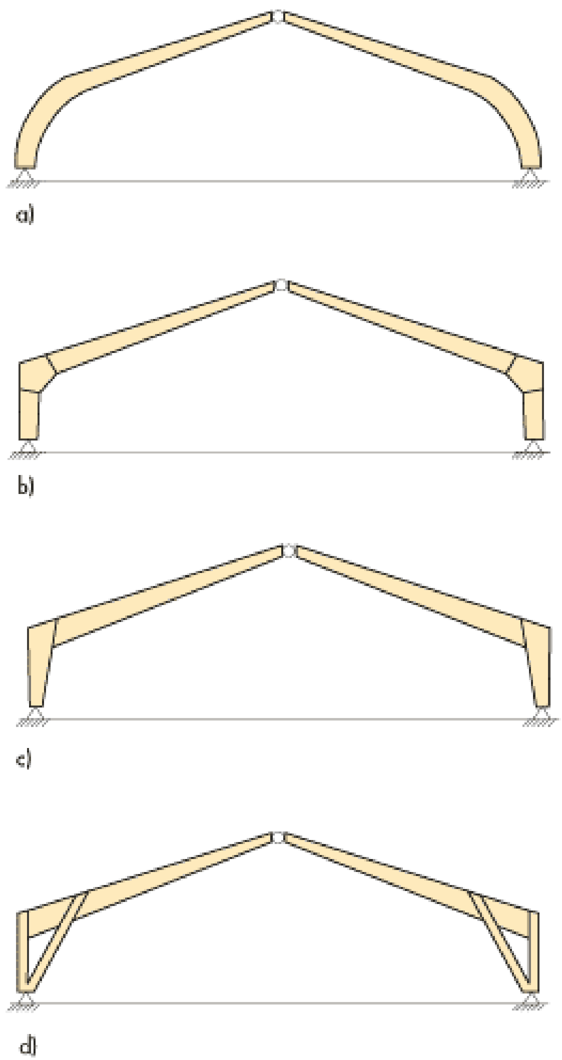


Figure 9.1 Examples of designs of three-pin portal frames

a) Frame with curved haunches

b) Frame with finger jointed haunches

c) Frame with bolted haunches

d) Built-up frame (knee braced frame)

10.2.9.2 Design of haunches

11.1.10.1 Design

Two- and three-pin arches can be designed using the same calculation methods as in 9.1.2. In this case, points 7 and 8 are replaced by the following:

7.*The arch is designed with regard to compression and bending as in 5.2 and to the following instructions.*

Three-pin parabolic arches with a rise ratio f/l= 0,14 can be designed with the aid of figure 10.6 (Design diagram). The same figure can also be used for assessment of sizes of three-pin circular arches.

11.2.10.2 Design diagram

The starter values in the diagram $(q/b)/f_{md}$ are obtained by calculating the load per running horizontal metre of arch (kN/m) and dividing this value with the width of the arch (metres) and with the design value of the bending strength (N/mm2) in the relevant loading. This is determined with regard to climate class, duration of load and, in relevant cases, safety class in accordance with the regulations applying in the particular case. See e.g. Annex 2.

The diagram is based on the assumption that the relationship between critical values of bending strength and compression strength parallel with the grain is 1,1 and that the relationship between the critical value of the modulus of elasticity in bending and the compression strength parallel to the grain is 370.

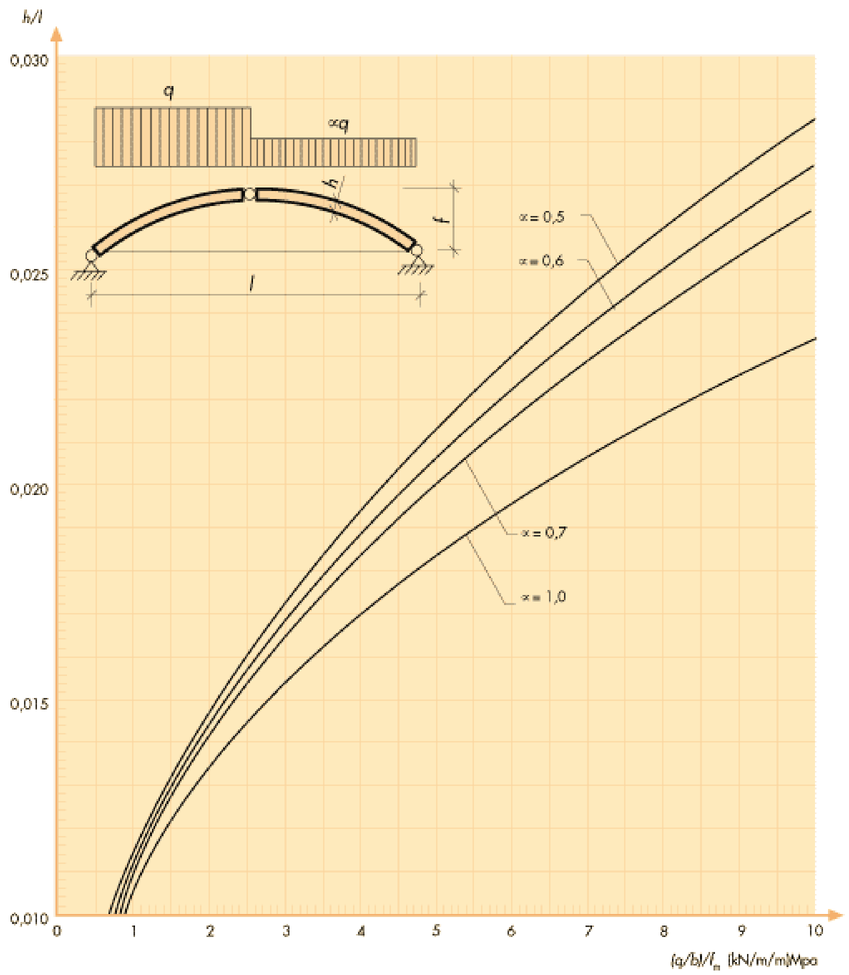


Figure 10.6 Diagram for design of 3-pin parabolic arch with reference to carrying capacity. The arch is assumed to be restrained from lateral buckling. Rise ratio of the arch f/l = 0,14.

12.1.11.1 Overlapping purlins

Overlapping purlins have the advantage that their carrying capacity is doubled over supports, where the moments are greatest. In the same way as with haunches the deflection of the purlin is also reduced. The overlap is made sufficiently long to reduce the moment to half; the field moment then becomes critical. Moments, support reactions and maximum deflection (*w*) can be calculated for a continuous beam with constant moment of inertia. Although the variation in stiffness affects the distribution of moments positively, this is counteracted by unavoidable deformations in the joints. If there are more than two similar bays, with overlaps as in figure 11.1, the following applies:

- End bays:

$$M_d = 0,080ql^2$$

Formula 11.1

$$w = 0,69ql^4 / 100EI$$

Formula 11.2

- Intermediate bay

$$M_d = 0,046ql^2$$

Formula 11.3

$$w = 0,32ql^4 / 100EI$$

Formula 11.4

Joints are designed for $F= 0.42ql$

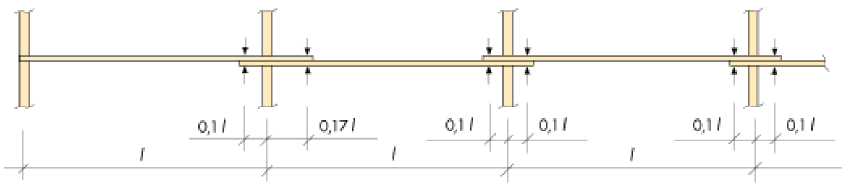


Figure 11.1 Continuous purlin system with overlaps.

Unjointed purlins in two or more bays are designed with regard to the fact that the roof beams form a deformable support. The moment of support in Table 6 has therefore been reduced by 10%. The support reactions at the intermediate supports can be reduced for the same reason. Purlins resting on three supports and jointed in the same line therefore rest on roof beams designed for 1,10 x *ql*(instead of 1,25 x *ql*).

12.2.11.2 The Gerber System

The Gerber system is designed so that field (span?) moments and support moments are equal. To reduce the risk of progressive collapse if one bay should collapse, the system should be designed so that every second bay is free from hinges. For two and three bays, the maximum deflection (*w*) can be calculated with the aid of table 6.2. For more than three bays, if the joints are placed as in alternative 1 in figure 11.2:

- End bay

$$M_{dm} = 0{,}096ql^2$$

Formula 11.5

$$V_d = V_{joint} = 0{,}44ql$$

Formula 11.6

$$w = 0{,}72ql^4 / 100EI$$

Formula 11.7

- Intermediate bay

$$M_d = 0{,}063ql^2$$

Formula 11.8

$$V_d = 0{,}56ql$$

Formula 11.9

$$V_{joint} = 0{,}35ql$$

Formula 11.10

$$w = 0{,}52ql^4 / 100EI$$

Formula 11.11

With joints placed as in alternative 2 in figure 11.2 (below), the same figures as above apply to the intermediate bays. The following applies to the end bays:

- End bay

$$M_d = 0{,}086ql^2$$

Formula 11.12

$$V_d = 0{,}59ql$$

Formula 11.13

$$w = 0{,}77ql^4 / 100EI$$

Formula 11.14

It is often practical to choose the same depth for purlins in the end and intermediate bays and if required instead widen the purlins in the end bays.

Where the roof slope exceeds 1:10, the inclination of the purlins must be taken into account. The vertical load, i.e. as a rule snow and self-weight, are split into a component perpendicular to the roof (q×cos a) and a component parallel with the roof (q×sin a) as in figure 11.3.

As a rule, the roof covering is stiff enough to take up the component in the plane of the roof by the diaphragm effect. This is not usually checked. If the roof slope is steep, or the diaphragm effect cannot be counted on, the purlins must however be checked for simultaneous bending in stiff and weak directions, e.g. as in 5.2, formulae 5.7 and 5.8. In steep roofs the other purlins can be hung up from the ridge purlin. In doubtful cases it can be worthwhile to reduce the span in the weak direction by hanging up the purlins from the ridge as in figure 11.4. The trusses should then be checked for the extra load which is placed on them by the suspension rods.

If the purlins are used to stiffen the primary beams, or to transfer tension or compression forces to wind bracing, the carrying capacity shall be checked for simultaneous compression and bending as in 5.2. Joints and connections to roof beams are also checked for these forces. See further 12.3.

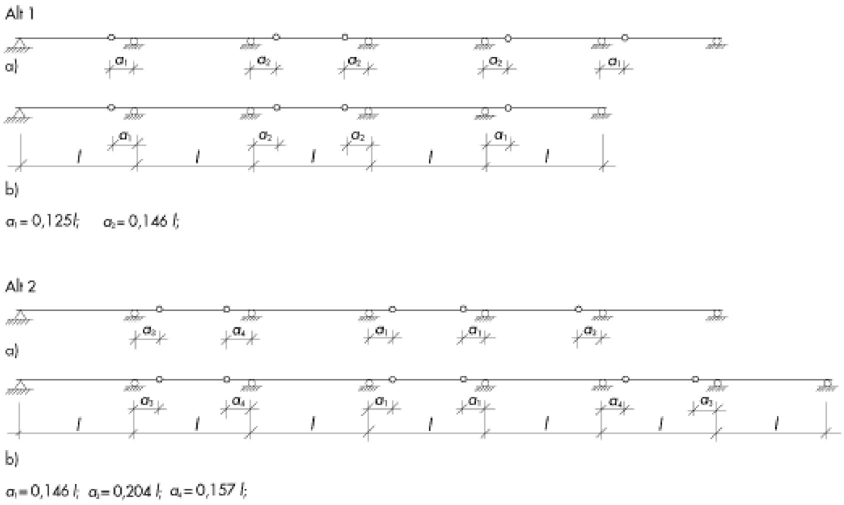


Figure 11.2 Various types of Gerber system.

Alternative 1: Joint in end bay

a) even number of bays b) odd number of bays.

Alternative 2: End bays without joints

a) even number of bays b) odd number of bays.

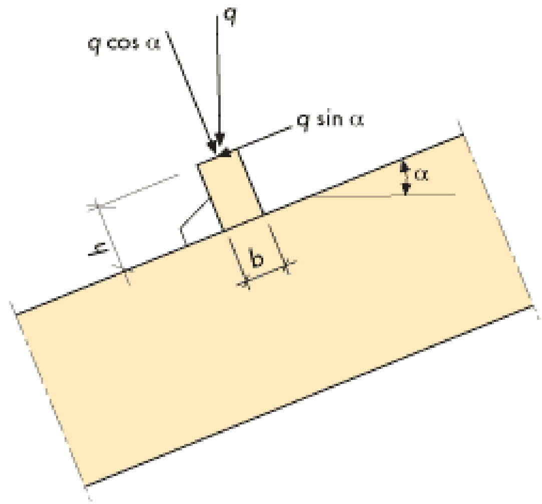


Figure 11.3 Vertical load on inclined purlin divided into components.

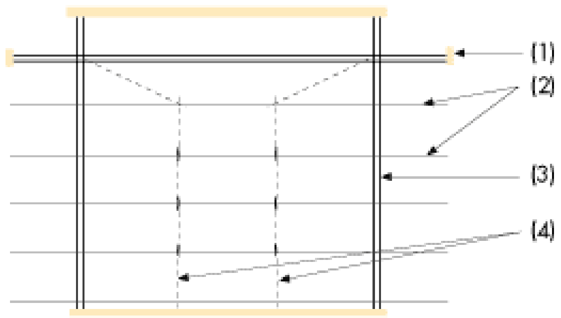


Figure 11.4 In steep roofs the other purlins can be hung up from the ridge purlin. (1) ridge purlin; (2) Other purlins; (3)Roof beam; (4) Suspension rod.

13.1.12.1 Diaphragm action

The diaphragm action in flat corrugated sheet metal roofs can be used to stabilise glulam timber structures under the following conditions:

- The sheets forming the roof can be jointed so that shear forces can be transferred.

- The roof diaphragm is stiffened along its edges by beams designed and fixed to take up forces. Edge beams at right angles to the corrugations can be of timber, while edge beams parallel with the corrugations are most easily made in steel.

- Concentrated loads (for example from posts (wind columns) are transferred via spreader struts, designed and fixed to take up the forces which act. A spreader strut at right angles to the corrugations can be of glulam, while that parallel with the corrugations shall be of steel.

The roof diaphragm can be designed according to the same principles as for steel structures.

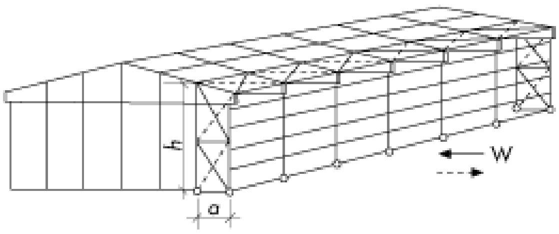


Figure 12.1 Bracing of glulam structure using wind trusses and wall bracing.

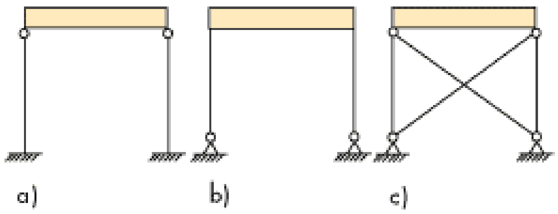


Figure 12.2 Examples of structures which are stable in their own plane.

13.2.12.2 Wind trusses

13.3.12.3 Lateral bracing of roof beams

Roof diaphragms or wind trusses are often used as lateral bracing of roof beams. The diaphragm or the truss shall then, in addition to other horizontal loads, e.g. wind (q_w), be designed for a horizontal, uniformly distributed load:

$$q_{bracing} = k_l \cdot \frac{n \cdot M}{50 l \cdot h}\left(1 - k_{crit}\right)$$

Formula 12.1

where

M= maximum moment in beam

h= depth of beam

l= span of roof diaphragm or wind truss

n= number of laterally braced beams

k_{crit}= factor (see 4.2) taking into account tipping risk in an unbraced beam

$$k_l = \min\begin{cases} 1 \\ \sqrt{0{,}15 / l} \end{cases}$$

The factor k_l takes into account the fact that greater care in workmanship can be expected in large structures.

Fixing of purlins or roofing sheets in the roof beam is designed for a force of:

$$F_1 = q_{bracing} \cdot a / n$$

Formula 12.2

where

a= distance between purlins or, in sheets direct on the roof beams, distance between screws.

The bracing structure shall be stiff enough to limit the deflection due to the design load $q_{bracing}$ to *l*/700 and due to the total load, including e.g. wind load, to *l*/500.

A purlin which is to brace several beams, and its fixing in the laterally bracing structure, is designed for the force:

$$F_2 = n \cdot F_1$$

Formel 12.3

Symbols as in figure 12.4 below.

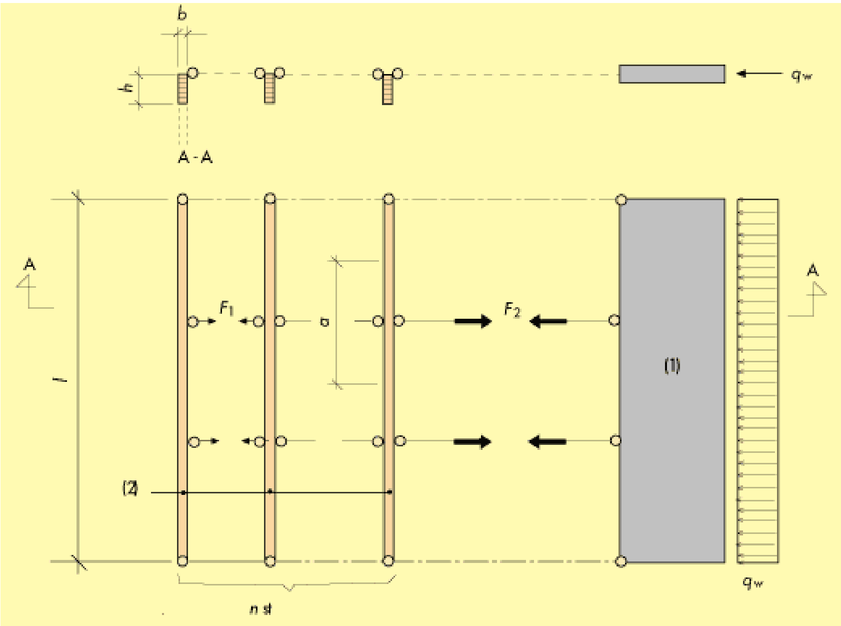


Figure 12.4. Lateral bracing of roof beams. 1) Laterally bracing structure. 2) Laterally bracing roof beams.

13.4.12.4 Progressive collapse

The structure in hall-type buildings with large spans (= 15 m) containing spaces where large numbers of people are present at the same time , e.g. sports halls, exhibitions and department stores, shall be designed in such a way that the risk of progressive collapse as a result of an accidental load is acceptably small.

An accidental load can for example be a car which runs into a column, or a gas explosion within the building and it is as a rule not reasonable to design individual structural components to withstand such loads. Instead, the structural system should be designed so that a primary part of the building, such as a main beam or column, may collapse without the building as a whole collapsing. In loading combinations with accidental loads, other loads are as a rule assumed to act with their normal (frequent) values and not with their extreme values (characteristic values).

Normally it is sufficient to estimate the horizontal stability of the remainder of the building if one main beam fails. In buildings containing spaces for large numbers of people it must also be shown that the damaged bay can be spanned by an alternative load-bearing system, e.g. by utilising the line- and membrane effect of purlins and roofing.

14.1.13.1 Bolted joints

Annex 4 shows the design carrying capacity of various screw diameters and glulam dimensions in

laterally loaded joints with side pieces of steel. The values are calculated both in accordance with EC 5 (ENV 1995:1-1) with NAD (Swedish).

14.2.13.2 Pinned column base

A pinned column base transfers horizontal and vertical forces. In principle, moments are not transferred. It is however an advantage if fixtures and fixings have sufficient moment capacity to be able to stabilise the column during erection. The connection should be designed in such a way that changes in the inclination of the column are not prevented, since any restraining forces here could give rise to splitting.

The fixtures are usually formed with external steel side plates nailed or screwed to the column. Alternatively a steel shoe can be used to avoid direct contact between the foundation and the column. If for aesthetic or fire safety reasons a hidden connection is desired a glued-in screw can be a suitable alternative.

The connection to the foundation can be made in different ways: The fixture can be cast into the concrete either direct or in pockets. They can also be welded to cast-in fixing plates. A third alternative is to secure the fixture with expander screws or kemankare.

Ends of columns resting directly on concrete, brick/blockwork or other hygroscopic material should be fitted with some kind of damp membrane, e.g. 3,2 mm oil tempered hardboard nailed and glued to the column base, or a rubber membrane. For columns out of doors or in spaces where water is regularly present, e.g. swimming pools, the connection with foundations shall be designed so that the end of the column is protected from water and can dry out quickly if it should become wet.

Fixtures are delivered separately except for glued-in screws which are always glued in the factory. Holes in columns for screwed joints should preferably be drilled during erection. It is thus possible to avoid fitting problems, specially with cast-in fixtures.

Design principles for four alternatives follow: stirrups of nailplate or flat steel,fishplate s of steel angle, column shoes and glued-in bolts. The instructions apply mainly to the design of timber and steel components. As regards the design of connecting concrete construction the conditions are too varied to enable general rules to be given in this handbook. The reader is instead referred to current concrete regulations.

14.3.13.3 Rigidly fixed column base

A rigidly fixed column base transfers moments in addition to horizontal and vertical forces. Rigidly fixed columns can be used to stabilise the building structure against horizontal forces. e.g. wind loads and braking loads from hoist blocks and traversers.

The moisture characteristics of the timber and its low strength at right angles to the grain in comparison with its bending strength mean that columns, specially rigidly fixed columns of glulam require special care in detail design in order to achieve a satisfactory fixing. The fixtures can be designed as external steel fishplates nailed or screwed/bolted to the column. If a hidden fixing is desired for aesthetic or fire reasons, glued-in bolts are a suitable alternative. However, glued-in bolts are only suitable for relatively small end moments. For large end moments steel fishplates with nailed or screwed joints are used.

The connection to the foundation can be designed in different ways. Columns with fixtures can be placed on bolts anchored in the foundation. Fixtures can also be welded to cast-in fixing plates. Alternatively the fixtures, e.g. glued-in bolts, can be placed in pockets in the foundation and the intervening space filled with concrete.

Column bases directly on the concrete, brick or other capillary material should be given some form of damp proofing, e.g. rubber membrane or 3,2 mm oil tempered board nailed and glued to the butt end of the column. Out of doors or in places where water occurs, e.g. swimming pools, the connection with the foundation shall be designed to protect the column base from water and allow it to dry out quickly if it should get wet occasionally.

The fixtures are assumed to be fitted to the column before erection. This is normally done in the factory, but for transport or other reasons they can be delivered separately to the site for fitting before erection. Glued-in bolts are always inserted at the factory.

Design principles for two solutions fishplates of nailplate or flat steel, and glued-in bolts are given below.

14.4.13.4 Pinned connection of beam to column

Pinned connections between beams and columns transfer horizontal and vertical forces. Moments are transferred only to a limited extent and are disregarded in the design. The fixing should be designed in such a way that the beam is not restrained against rotation. If the rotation is restrained, extra stresses will arise which can lead to unforeseen damage to glulam columns and beams.

The fixture is usually designed with external steel fishplates which are nailed, screwed or bolted into the column and the beam. If a hidden fixture is desired for aesthetic or fire reasons, glued-in bolts can be a suitable alternative. Glued-in bolts are however only suitable for indoor use and only for relatively small horizontal forces. External fishplates provide some lateral bracing of the beam against buckling, while glued-in bolts demand bracing of the top of the beam. Recessed beams are often used on end wall columns, to transfer their horizontal loads to the beam.

The fixtures are either delivered separately for fitting to the column on the site, or fitted to the column or beam. Glued-in bolts are always fitted in the factory.

The text below shows design principles for three alternatives   Fishplates of nailplate or flat steel, glued-in bolts and the beam recessed into the top of the column.

14.5.13.5 Pinned ridge joint

Pinned ridge joints transfer horizontal and vertical forces. Moments are transferred only to a limited extent and are not taken into account in the design. The fixture should not restrict changes of angle in the beams. If this movement cannot take place extra stresses will arise which can lead to unforeseen damage to the structure.

The fixture normally consists of external fishplates of nailplate. These can be combined with a routed-in steel connector if forces are large. Designs using external flat steel and fishplates and shear connectors can also be used, as can fishplates of timber or plywood. The text below shows the design principles for the first two types.

14.6.13.6 Pinned beam joint

Pinned beam joints transfer horizontal and vertical forces. Moments are only transferred to a limited extent and this is ignored in the design. The fixture should be designed in such a way that it cannot restrict rotation of the beams. If such rotation cannot take place extra stresses will arise, which can cause unforeseen damage to the structure.

For moderate forces the fixture can be designed with external steel fishplates nailed or screwed/bolted to the beams. For larger forces. Gerber fixtures are a suitable alternative.

In the following text the design principles for two types are explained: fishplates of nailplate and Gerber fixtures.

14.7.13.7 Connections with secondary beams

Secondary beams placed on top of primary beams transfer vertical forces and small horizontal forces in the direction of the primary beam. A secondary beam fixed to the side of a primary beam also transfers horizontal forces in the direction of the secondary beam. If necessary the fixture can be designed so that moments are also transferred.

For connections between secondary and primary beams there are a large number of different factory-made fixtures of steel which can be used to advantage. The manufacturers' product catalogues present information on the carrying capacity and fixing of various fixtures.

14.8.13.8 Fixing of ties

The tie fixture transfers only horizontal tension forces to the beam. The tie usually consists of steel or glulam.

The fixture is normally designed so that the tension force acts as near as possible to the intersection of the system lines of the beam and column. It is also possible to introduce some eccentricity on purpose, to be used in the design of a roof beam or arch.

In the text below the principles of the design of steel, alternatively glulam, ties are shown.

14.9.13.9 Supports for two- and three-pinned arches

Supports springing points for two- or three-pinned arches are designed as more or less moment-free hinges. For small spans up to about 25 m it is usual for reasons of cost to settle for an imperfect hinge. Although this transfers moments they are of such limited size that they do not need to be taken into account when designing the arch, though they will be of importance when designing the fixture itself and its anchorage.

For larger spans the support forces are so large that a careful design of the hinge is necessary if the static function of the arch shall be as intended.

15.1.14.1 Glulam and fire

If a glulam structure is exposed to the normal action of fire its surface will ignite. Burning will then continue inwards at a more or less constant speed. Penetration is however slow since the carbonised layer which is formed not only insulates but also reduces the flow of air to the burning zone. Any splits, and screws and other metal fasteners, will however cause the fire to spread faster. The glue joints, on the other hand, are more fire-resistant than the timber itself and are almost always undamaged in the unaffected part of the structure.

Even under a long exposure to fire the temperature in the unburnt parts of a massive glulam structure will stay under 100°C. Temperature movement in fire will therefore be small   notably less than in a steel frame where longitudinal expansion will often cause secondary damage, e.g. to supports or masonry. A glulam structure does not deform to anything like the same extent as with steel. The total damage is therefore usually less in buildings with a glulam frame than in those with a steel frame.

For those parts of a building which have to be demolished after a fire, the ability to clear up with a motor saw and simple hand tools saves time and money in a critical situation.

15.2.14.2 Fire insurance

The choice of structural material is often critically affected by the size of the fire insurance rates in various alternatives. When calculating the rate in individual cases the insurance companies take into account the type of activity and the building method. In addition factors such as distance to the nearest fire brigade, availability of water, risk of fire spread etc. are assessed.

15.3.14.3 Fire requirements in regulations

The fire safety requirements on building technology are in most regulations, including the Nordic ones, given as requirements on a certain class of buildings and on the parts of the building, materials and surfaces of which it is composed. In the early stages of a fire it is primarily the surfaces of roofs, walls and floors which are important for fire safety, particularly the surfaces in escape routes. The fire characteristics of structural and fire separating elements are on the other hand important in a fully developed fire. They are then critical to the stability of the building and the spread of fire to other parts of the building or to nearby buildings.

What requirements apply to a certain part of the building depend on the risk for serious personal injury if the element should fail due to the fire. Above all the size of the building, the number of floors and the use all play a part. If sprinklers are installed the authorities can in some cases make exceptions to the rules. This applies specially to requirements on surfaces.

Load bearing structures shall be designed so that there is adequate safety in fire and under the load that can be expected to occur in fire. These basic principles exist in most countries, while concepts and details in the rules may differ. The requirements can be shown to be met either by choice of fire classified structures or by calculating the carrying capacity of the structure in a fire. Normally the carrying capacity is calculated at a lower loading than when calculating without regard to fire, e.g. with frequent values of variable loads.

The fire class for load bearing or compartmentization is given, in accordance with a common EU system, with three notations for functional requirements   R for carrying capacity, E for integrity (against smoke and fire gases), I for thermal insulation. Each is given a figure in minutes showing the time it can withstand a standard fire without loss of function, e.g. R30, E15 or I90. The notations can be combined in various ways and the fire requirement for a part of the building with requirements on both carrying and separating functions can be RE or REI. Note that the former distinctions between combustible and non-combustible materials no longer exist. What is critical for the classification is the time it takes before a part of the building ceases to function due to fire   not what materials it contains.

Surfaces are classified according to ability to delay or prevent ignition and smoke generation in fire. Within EU there are common classes for surfaces and materials which for the time being can be used parallel with the national classes. The Euroclasses are A, B, C, D, E and F where A has the best and F the poorest fire characteristics. Class D corresponds to untreated raw boarding..

Glulam beams and columns usually form part or parts of the building with load bearing and/or separating functions. Glulam components are often visible and form parts of ceilings or walls. As far as load bearing is concerned, Norwegian and Swedish authorities allow beams and columns in all types of buildings   even multi-storey   in glulam. However, requirements on surfaces can pose certain restrictions. Finland is more restrictive and allows four storeys in buildings with a glulam frame.

Requirements calling for surfaces of a higher class than D (untreated timber) can be met by treating glulam with an approved paint system. Both transparent and opaque systems exist. On the other hand the requirement on a substrate of non-combustible material or fire protective covering can naturally not be met by those parts of the ceiling which consist of visible glulam.

In practice several different materials, with different fire characteristics, often occur near each other within the same ceiling or wall surface. A common assessment problem arises for example when for aesthetic or other reasons a wholly or partly visible glulam frame is desired. At present there exists no documented knowledge on how an only partly combustible surface performs in regard to flame spread and flashover. It is however obvious that small exposed glulam surfaces a long way from one another do not materially affect the fire performance of an otherwise non-combustible surface. In Sweden the practice has arisen of allowing small parts of the building whose surface is of no importance to the spread of fire, to be of a lower class than s strict application of the rules would allow, though not lower than class D. The same applies to small rooms whose surfaces do not affect escape from the building. The assessment of authorities is usually based on the fact that an untreated non-combustible ceiling covering (e.g. wood wool slabs) supported by untreated glulam beams are not affected if the total exposed area (sum of the undersides and sides of the beams) does not exceed 20% of the floor area. If the beams have a class B surface finish this can usually be increased to 50%.

15.4.14.4 Designing timber structures for fire

15.5.14.5 Fire resistance of connections and joints

While glulam components, as has been shown, have extremely good fire characteristics, steel connections and fixtures are weak points which often need to be protected if the structure as a whole is to fulfil the requirements for a certain fire class.

In EC 5 part 1-2 there are rules for calculating the carrying capacity in fire of both protected and unprotected nail and screw joints.

Refined methods for calculating the structural principle and the carrying capacity of complicated structural connections in fire do not exist at present. However. A large number of fire tests have been documented and these can serve as a basis for assessments. The examples given below are mainly based on Holz Brandschutz Handbuch (Kordina & Meyer-Ottens, Deutsche Gesellschaft für Holzforschung e.V., München 1983), where those interested can find further information.

16.1.15.1 Constructional types

There are many examples, particularly in continental Europe and other countries, of glulam poles with architecturally advanced forms, where the opportunities of glulam for free design have been used to the full. As a rule this applies to lines going through fairly densely-populated areas where aesthetic considerations are of great importance. In the comparatively sparsely populated Nordic countries rational production and economy are also important. The types of construction traditionally used for roundwood poles are therefore also the most usual in glulam (see figure 15.1):

- simple poles for one or two phases
- H-poles with or without cross bracing
- A-poles

Glulam for line construction is made in the same way and under the same rules as for buildings. They can be made in practically unrestricted lengths and with very large cross-sections. Solid poles are however comparatively heavy and it is therefore often preferable to choose H-, T- or hollow sections. Figure 15.2 below shows some examples.

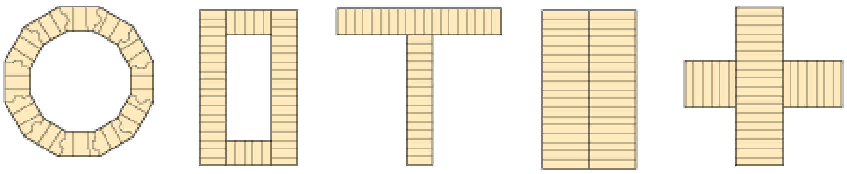


Figure 15.2 Examples of cross sectional forms.

16.2.15.2 Design

While the design of telephone and lighting poles is more or less stereotyped and based on practical experience, poles for high voltage lines are designed according to normal principles of safety for load-bearing structures. Attention shall be paid to loading and to the properties of materials so that the risk of structural failure is acceptably low from the point of view of society.

The loads which occur are (a) vertical support reactions from lines and other structural parts, and (b) horizontal support reactions perpendicular to and parallel with the line.

The vertical support reactions consist of permanent loads from the self-weight of the structure and live loads consisting of snow and ice on the lines. The live load should be placed in the least favourable position.

Horizontal reactions perpendicular to the line are caused by wind on poles and lines, and by line forces where the line changes direction. During the winter, when the lines can be weighed down by snow and ice, the horizontal loads can be considerable.

Horizontal support reactions parallel with the line are caused by the fact that the distance between poles varies and by non-symmetrical loads, e.g. when one span is loaded with ice while the adjacent one is ice-free. They can also be caused by breakage of one or more lines.

What loads various types of lines shall be designed for is given in national regulations

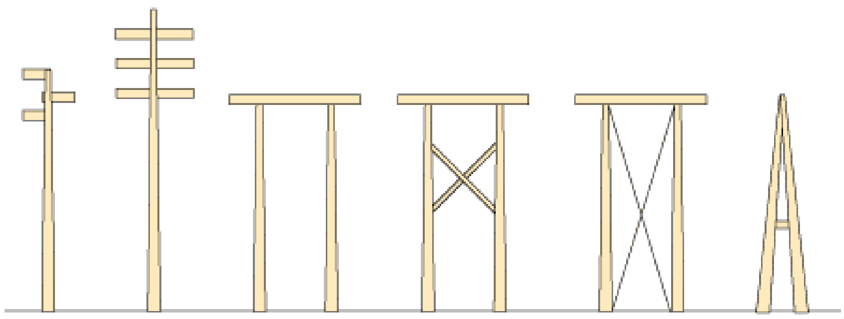


Figure 15.1 Examples of common types of glulam poles.

16.3.15.3 Foundations

One of the advantages of using timber poles is that foundation work is simplified. Special foundations are in general not required.

On solid ground the glulam pole is rigidly fixed by digging down to a suitable depth depending on pole length, load and soil type, usually 2 3 metres.

16.4.15.4 Timber protection

Line poles of timber are subjected to extremely tough climatic stresses. Where the pole passes through topsoil, effective chemical protection is a necessity. Normally the whole pole is pressure treated. Note however that the use of preservative is regulated both in standards and in national environmental laws. See also 1.5.5.

Glulam poles can be treated before or after gluing. In the first case manufacture is based on treated laminates. Only laminates impregnated with water-soluble chemicals can be used, as laminates treated with oil-based fluids, e.g. creosote oil, cannot be glued. For perfect gluing surfaces must nevertheless be planed before gluing. This removes much of the preservative, and can also expose untreated heartwood. In addition, pole sides are often planed to improve the dimensional accuracy of the product, with the result that even more preservative is removed and more heartwood is exposed.

Treatment after gluing gives better protection against rot, but the method is limited by the availability of treatment plants with sufficiently large pressure cylinders. Currently it is possible to treat 30 m long products in Sweden. Norway and Finland. The method is used primarily for creosote treatment, since treatment with water-based preservatives leads to major splitting problems when the poles dry out after treatment.

The best method, but also the most expensive one, is double treatment , where the glulam is made of salt-treated laminates and pressure treated after gluing with creosote oil.

17.1.16.1 General

In the Nordic countries today it is principally bridges for pedestrians, cyclists and moped users that are built in timber. In recent years, however, the interest in timber road bridges has shown a marked increase and a considerable number of timber bridges have been built in Norway, Finland and Sweden.

17.2.16.2 Bridge types

From the structural point of view, a distinction is made between the sub-structure of a bridge and its superstructure.

The superstructure is the mainly horizontal load-bearing construction that spans an impediment such as a waterway. The superstructure includes the road deck, the main beams carrying the road deck and finally the primary structure, e.g. girders or arches which take the loads (self- weight, traffic load and wind) from the main beams and transfer them to the sub-structure.

The sub-structure transfers the loads further to the foundations in the form of abutments and various types of intermediate support. Nowadays these are generally concrete, but stone and brick were common earlier, as were timber poles, the lastnamed however with the limitations mentioned earlier.

Girder bridges, arches and suspension bridges are the three principal types of superstructure. Girder bridges are usually taken to include slab bridges, trusses and other types of structure consisting of assemblies of bars such as hanging and strut bridges (see figures 16.1, 16.2 and 16.3). Combinations of various types are also used.

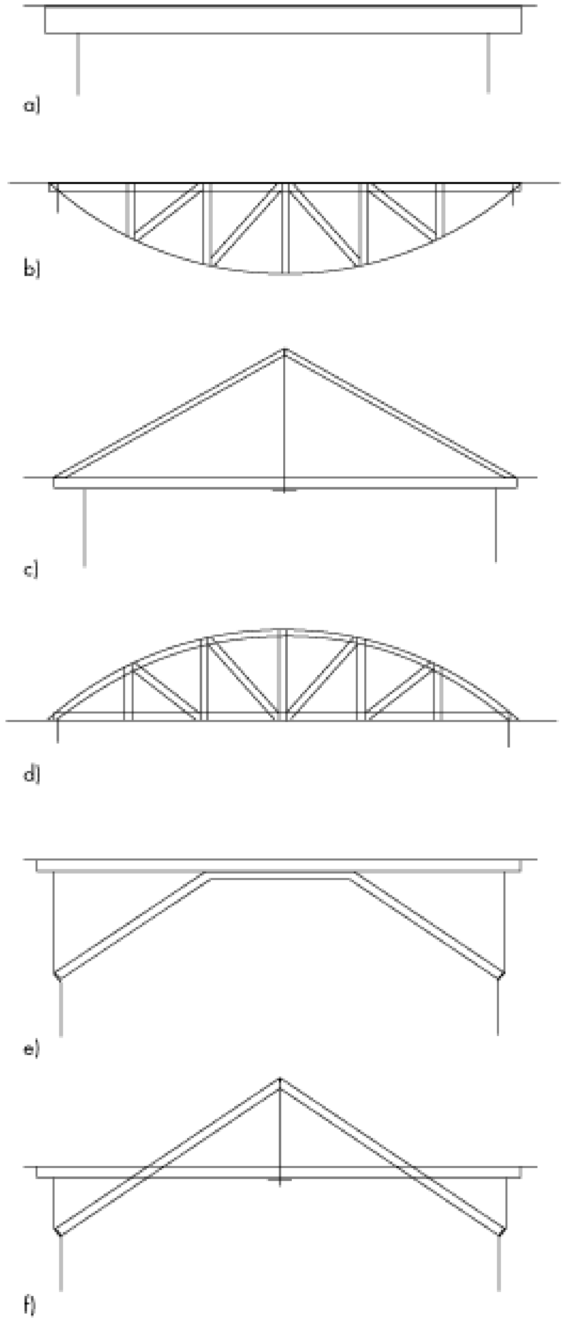


Figure 16.1 Girder Bridge types a) Simple girder. b) Bottom tensioned girder. c) King post. d) Truss. e) Strut frame. f) Combined king post-strut frame.

Which type of structure is most suitable in each case depends on the specific conditions, e.g. what free span and what unobstructed height is demanded, the height available for the structure and the type of traffic for which the bridge is intended. Appearance is often of great importance, since bridges are as a rule a dominating feature in the landscape. Other factors which influence the choice are the soil conditions and any requirements that a certain building material should be used.

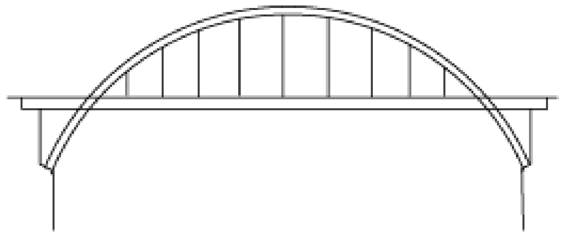


Figure 16.2 Arch bridge.

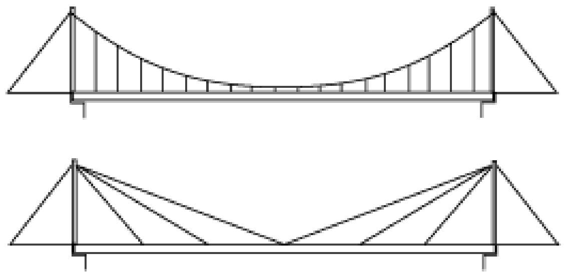


Figure 16.3 Suspension bridge and cable stayed bridge.

17.3.16.3 Design of glulam bridges

18.1. Cross section properties

Annex 1 Cross-sectional properties of rectangular sections

h mm	No. of lami-nations	b = 42 mm			b = 56 mm			b = 66 mm			b = 78 mm			b = 90 mm		
		A mm²	W_x mm³	I_x mm⁴	A mm²	W_x mm³	I_x mm⁴	A mm²	W_x mm³	I_x mm⁴	A mm²	W_x mm³	I_x mm⁴	A mm²	W_x mm³	I_x mm⁴
180	4	$76\cdot10^2$	$227\cdot10^3$	$204\cdot10^5$	$101\cdot10^2$	$302\cdot10^3$	$272\cdot10^5$	$119\cdot10^2$	$356\cdot10^3$	$321\cdot10^5$	$140\cdot10^2$	$421\cdot10^3$	$379\cdot10^5$	$162\cdot10^2$	$486\cdot10^3$	$437\cdot10^5$
225	5	$95\cdot10^2$	$354\cdot10^3$	$399\cdot10^5$	$126\cdot10^2$	$473\cdot10^3$	$532\cdot10^5$	$149\cdot10^2$	$557\cdot10^3$	$626\cdot10^5$	$176\cdot10^2$	$658\cdot10^3$	$740\cdot10^5$	$203\cdot10^2$	$759\cdot10^3$	$854\cdot10^5$
270	6	$113\cdot10^2$	$510\cdot10^3$	$689\cdot10^5$	$151\cdot10^2$	$680\cdot10^3$	$919\cdot10^5$	$178\cdot10^2$	$802\cdot10^3$	$108\cdot10^6$	$211\cdot10^2$	$948\cdot10^3$	$128\cdot10^6$	$243\cdot10^2$	$109\cdot10^4$	$148\cdot10^6$
315	7	$132\cdot10^2$	$695\cdot10^3$	$109\cdot10^6$	$176\cdot10^2$	$926\cdot10^3$	$146\cdot10^6$	$208\cdot10^2$	$109\cdot10^4$	$172\cdot10^6$	$246\cdot10^2$	$129\cdot10^4$	$203\cdot10^6$	$284\cdot10^2$	$149\cdot10^4$	$234\cdot10^6$
360	8	$151\cdot10^2$	$907\cdot10^3$	$163\cdot10^6$	$202\cdot10^2$	$121\cdot10^4$	$218\cdot10^6$	$238\cdot10^2$	$143\cdot10^4$	$257\cdot10^6$	$281\cdot10^2$	$168\cdot10^4$	$303\cdot10^6$	$324\cdot10^2$	$194\cdot10^4$	$350\cdot10^6$
405	9	$170\cdot10^2$	$115\cdot10^4$	$233\cdot10^6$	$227\cdot10^2$	$153\cdot10^4$	$310\cdot10^6$	$267\cdot10^2$	$180\cdot10^4$	$365\cdot10^6$	$316\cdot10^2$	$213\cdot10^4$	$432\cdot10^6$	$365\cdot10^2$	$246\cdot10^4$	$498\cdot10^6$
450	10	$189\cdot10^2$	$142\cdot10^4$	$319\cdot10^6$	$252\cdot10^2$	$189\cdot10^4$	$425\cdot10^6$	$297\cdot10^2$	$223\cdot10^4$	$501\cdot10^6$	$351\cdot10^2$	$263\cdot10^4$	$592\cdot10^6$	$405\cdot10^2$	$304\cdot10^4$	$683\cdot10^6$
495	11	$208\cdot10^2$	$172\cdot10^4$	$425\cdot10^6$	$277\cdot10^2$	$229\cdot10^4$	$566\cdot10^6$	$327\cdot10^2$	$270\cdot10^4$	$667\cdot10^6$	$386\cdot10^2$	$319\cdot10^4$	$788\cdot10^6$	$446\cdot10^2$	$368\cdot10^4$	$910\cdot10^6$
540	12	$227\cdot10^2$	$204\cdot10^4$	$551\cdot10^6$	$302\cdot10^2$	$272\cdot10^4$	$735\cdot10^6$	$356\cdot10^2$	$321\cdot10^4$	$866\cdot10^6$	$421\cdot10^2$	$379\cdot10^4$	$102\cdot10^7$	$486\cdot10^2$	$437\cdot10^4$	$118\cdot10^7$
585	13	$246\cdot10^2$	$240\cdot10^4$	$701\cdot10^6$	$328\cdot10^2$	$319\cdot10^4$	$934\cdot10^6$	$386\cdot10^2$	$376\cdot10^4$	$110\cdot10^7$	$456\cdot10^2$	$445\cdot10^4$	$130\cdot10^7$	$527\cdot10^2$	$513\cdot10^4$	$150\cdot10^7$
630	14	$265\cdot10^2$	$278\cdot10^4$	$875\cdot10^6$	$353\cdot10^2$	$370\cdot10^4$	$117\cdot10^7$	$416\cdot10^2$	$437\cdot10^4$	$138\cdot10^7$	$491\cdot10^2$	$516\cdot10^4$	$163\cdot10^7$	$567\cdot10^2$	$595\cdot10^4$	$188\cdot10^7$
675	15	$284\cdot10^2$	$319\cdot10^4$	$108\cdot10^7$	$378\cdot10^2$	$425\cdot10^4$	$144\cdot10^7$	$446\cdot10^2$	$501\cdot10^4$	$169\cdot10^7$	$527\cdot10^2$	$592\cdot10^4$	$200\cdot10^7$	$608\cdot10^2$	$683\cdot10^4$	$231\cdot10^7$
720	16				$403\cdot10^2$	$484\cdot10^4$	$174\cdot10^7$	$475\cdot10^2$	$570\cdot10^4$	$205\cdot10^7$	$562\cdot10^2$	$674\cdot10^4$	$243\cdot10^7$	$648\cdot10^2$	$778\cdot10^4$	$280\cdot10^7$
765	17				$428\cdot10^2$	$546\cdot10^4$	$209\cdot10^7$	$505\cdot10^2$	$644\cdot10^4$	$246\cdot10^7$	$597\cdot10^2$	$761\cdot10^4$	$291\cdot10^7$	$689\cdot10^2$	$878\cdot10^4$	$336\cdot10^7$
810	18				$454\cdot10^2$	$612\cdot10^4$	$248\cdot10^7$	$535\cdot10^2$	$722\cdot10^4$	$292\cdot10^7$	$532\cdot10^2$	$853\cdot10^4$	$345\cdot10^7$	$729\cdot10^2$	$984\cdot10^4$	$399\cdot10^7$
855	19							$564\cdot10^2$	$804\cdot10^4$	$344\cdot10^7$	$667\cdot10^2$	$950\cdot10^4$	$406\cdot10^7$	$770\cdot10^2$	$110\cdot10^5$	$469\cdot10^7$
900	20							$594\cdot10^2$	$891\cdot10^4$	$401\cdot10^7$	$702\cdot10^2$	$105\cdot10^5$	$474\cdot10^7$	$810\cdot10^2$	$122\cdot10^5$	$547\cdot10^7$
945	21							$624\cdot10^2$	$982\cdot10^4$	$464\cdot10^7$	$737\cdot10^2$	$116\cdot10^5$	$549\cdot10^7$	$851\cdot10^2$	$134\cdot10^5$	$633\cdot10^7$
990	22							$653\cdot10^2$	$108\cdot10^5$	$534\cdot10^7$	$772\cdot10^2$	$127\cdot10^5$	$631\cdot10^7$	$891\cdot10^2$	$147\cdot10^5$	$728\cdot10^7$
1035	23										$807\cdot10^2$	$139\cdot10^5$	$721\cdot10^7$	$932\cdot10^2$	$161\cdot10^5$	$832\cdot10^7$
1080	24										$842\cdot10^2$	$152\cdot10^5$	$819\cdot10^7$	$972\cdot10^2$	$175\cdot10^5$	$945\cdot10^7$
1125	25													$101\cdot10^3$	$190\cdot10^5$	$107\cdot10^8$
1170	26													$105\cdot10^3$	$205\cdot10^5$	$120\cdot10^8$
1215	27													$109\cdot10^3$	$221\cdot10^5$	$135\cdot10^8$
1260	28															
1305	29															
1350	30															
1395	31															
1440	32															
1485	33															
1530	34															
1575	35															
1620	36															

h mm	No. of lami-nations	b = 115 mm			b = 140 mm			b = 165 mm			b = 190 mm			b = 215 mm		
		A mm²	W_x mm³	I_x mm⁴	A mm²	W_x mm³	I_x mm⁴	A mm²	W_x mm³	I_x mm⁴	A mm²	W_x mm³	I_x mm⁴	A mm²	W_x mm³	I_x mm⁴
180	4	$207\cdot10^2$	$621\cdot10^3$	$559\cdot10^5$	$252\cdot10^2$	$756\cdot10^3$	$680\cdot10^5$	$297\cdot10^2$	$891\cdot10^3$	$802\cdot10^5$	$342\cdot10^2$	$103\cdot10^4$	$923\cdot10^5$	$387\cdot10^2$	$116\cdot10^4$	$104\cdot10^6$
225	5	$259\cdot10^2$	$970\cdot10^3$	$109\cdot10^6$	$315\cdot10^2$	$118\cdot10^4$	$133\cdot10^6$	$371\cdot10^2$	$139\cdot10^4$	$157\cdot10^6$	$428\cdot10^2$	$160\cdot10^4$	$180\cdot10^6$	$484\cdot10^2$	$181\cdot10^4$	$204\cdot10^6$
270	6	$311\cdot10^2$	$140\cdot10^4$	$189\cdot10^6$	$378\cdot10^2$	$170\cdot10^4$	$230\cdot10^6$	$446\cdot10^2$	$200\cdot10^4$	$271\cdot10^6$	$513\cdot10^2$	$231\cdot10^4$	$312\cdot10^6$	$581\cdot10^2$	$261\cdot10^4$	$353\cdot10^6$
315	7	$362\cdot10^2$	$190\cdot10^4$	$300\cdot10^6$	$441\cdot10^2$	$232\cdot10^4$	$365\cdot10^6$	$520\cdot10^2$	$273\cdot10^4$	$430\cdot10^6$	$599\cdot10^2$	$314\cdot10^4$	$495\cdot10^6$	$677\cdot10^2$	$356\cdot10^4$	$560\cdot10^6$
360	8	$414\cdot10^2$	$248\cdot10^4$	$447\cdot10^6$	$504\cdot10^2$	$302\cdot10^4$	$544\cdot10^6$	$594\cdot10^2$	$356\cdot10^4$	$642\cdot10^6$	$684\cdot10^2$	$410\cdot10^4$	$739\cdot10^6$	$774\cdot10^2$	$464\cdot10^4$	$836\cdot10^6$
405	9	$466\cdot10^2$	$314\cdot10^4$	$637\cdot10^6$	$567\cdot10^2$	$383\cdot10^4$	$775\cdot10^6$	$668\cdot10^2$	$451\cdot10^4$	$913\cdot10^6$	$770\cdot10^2$	$519\cdot10^4$	$105\cdot10^7$	$871\cdot10^2$	$588\cdot10^4$	$119\cdot10^7$
450	10	$518\cdot10^2$	$388\cdot10^4$	$873\cdot10^6$	$630\cdot10^2$	$473\cdot10^4$	$106\cdot10^7$	$743\cdot10^2$	$557\cdot10^4$	$125\cdot10^7$	$855\cdot10^2$	$641\cdot10^4$	$144\cdot10^7$	$968\cdot10^2$	$726\cdot10^4$	$163\cdot10^7$
495	11	$569\cdot10^2$	$470\cdot10^4$	$116\cdot10^7$	$693\cdot10^2$	$572\cdot10^4$	$142\cdot10^7$	$817\cdot10^2$	$674\cdot10^4$	$167\cdot10^7$	$941\cdot10^2$	$776\cdot10^4$	$192\cdot10^7$	$106\cdot10^3$	$878\cdot10^4$	$217\cdot10^7$
540	12	$621\cdot10^2$	$559\cdot10^4$	$151\cdot10^7$	$756\cdot10^2$	$680\cdot10^4$	$184\cdot10^7$	$891\cdot10^2$	$802\cdot10^4$	$217\cdot10^7$	$103\cdot10^3$	$923\cdot10^4$	$249\cdot10^7$	$116\cdot10^3$	$104\cdot10^5$	$282\cdot10^7$
585	13	$673\cdot10^2$	$656\cdot10^4$	$192\cdot10^7$	$819\cdot10^2$	$799\cdot10^4$	$234\cdot10^7$	$965\cdot10^2$	$941\cdot10^4$	$275\cdot10^7$	$111\cdot10^3$	$108\cdot10^5$	$317\cdot10^7$	$126\cdot10^3$	$123\cdot10^5$	$359\cdot10^7$
630	14	$725\cdot10^2$	$761\cdot10^4$	$240\cdot10^7$	$882\cdot10^2$	$926\cdot10^4$	$292\cdot10^7$	$104\cdot10^3$	$109\cdot10^5$	$344\cdot10^7$	$120\cdot10^3$	$126\cdot10^5$	$396\cdot10^7$	$135\cdot10^3$	$142\cdot10^5$	$448\cdot10^7$
675	15	$776\cdot10^2$	$873\cdot10^4$	$295\cdot10^7$	$945\cdot10^2$	$106\cdot10^5$	$359\cdot10^7$	$111\cdot10^3$	$125\cdot10^5$	$423\cdot10^7$	$128\cdot10^3$	$144\cdot10^5$	$487\cdot10^7$	$145\cdot10^3$	$163\cdot10^5$	$551\cdot10^7$
720	16	$828\cdot10^2$	$994\cdot10^4$	$358\cdot10^7$	$101\cdot10^3$	$121\cdot10^5$	$435\cdot10^7$	$119\cdot10^3$	$143\cdot10^5$	$513\cdot10^7$	$137\cdot10^3$	$164\cdot10^5$	$591\cdot10^7$	$155\cdot10^3$	$186\cdot10^5$	$669\cdot10^5$
765	17	$880\cdot10^2$	$112\cdot10^5$	$429\cdot10^7$	$107\cdot10^3$	$137\cdot10^5$	$522\cdot10^7$	$126\cdot10^3$	$161\cdot10^5$	$616\cdot10^7$	$145\cdot10^3$	$185\cdot10^5$	$709\cdot10^7$	$164\cdot10^3$	$210\cdot10^5$	$802\cdot10^7$
810	18	$932\cdot10^2$	$126\cdot10^5$	$509\cdot10^7$	$113\cdot10^3$	$153\cdot10^5$	$620\cdot10^7$	$134\cdot10^3$	$180\cdot10^5$	$731\cdot10^7$	$154\cdot10^3$	$208\cdot10^5$	$841\cdot10^7$	$174\cdot10^3$	$235\cdot10^5$	$952\cdot10^7$
855	19	$983\cdot10^2$	$140\cdot10^5$	$599\cdot10^7$	$120\cdot10^3$	$171\cdot10^5$	$729\cdot10^7$	$141\cdot10^3$	$201\cdot10^5$	$859\cdot10^7$	$162\cdot10^3$	$231\cdot10^5$	$990\cdot10^7$	$184\cdot10^3$	$262\cdot10^5$	$112\cdot10^8$
900	20	$104\cdot10^3$	$155\cdot10^5$	$699\cdot10^7$	$126\cdot10^3$	$189\cdot10^5$	$851\cdot10^7$	$149\cdot10^3$	$233\cdot10^5$	$100\cdot10^8$	$171\cdot10^3$	$257\cdot10^5$	$115\cdot10^8$	$194\cdot10^3$	$290\cdot10^5$	$131\cdot10^8$
945	21	$109\cdot10^3$	$171\cdot10^5$	$809\cdot10^7$	$132\cdot10^3$	$208\cdot10^5$	$985\cdot10^7$	$156\cdot10^3$	$246\cdot10^5$	$116\cdot10^8$	$180\cdot10^3$	$283\cdot10^5$	$134\cdot10^8$	$203\cdot10^3$	$320\cdot10^5$	$151\cdot10^8$
990	22	$114\cdot10^3$	$188\cdot10^5$	$930\cdot10^7$	$139\cdot10^3$	$229\cdot10^5$	$113\cdot10^8$	$163\cdot10^3$	$270\cdot10^5$	$133\cdot10^8$	$188\cdot10^3$	$310\cdot10^5$	$154\cdot10^8$	$213\cdot10^3$	$351\cdot10^5$	$174\cdot10^8$
1035	23	$119\cdot10^3$	$205\cdot10^5$	$106\cdot10^8$	$145\cdot10^3$	$250\cdot10^5$	$129\cdot10^8$	$171\cdot10^3$	$295\cdot10^5$	$152\cdot10^8$	$197\cdot10^3$	$339\cdot10^5$	$176\cdot10^8$	$223\cdot10^3$	$384\cdot10^5$	$199\cdot10^8$
1080	24	$124\cdot10^3$	$224\cdot10^5$	$121\cdot10^8$	$151\cdot10^3$	$272\cdot10^5$	$147\cdot10^8$	$178\cdot10^3$	$321\cdot10^5$	$173\cdot10^8$	$205\cdot10^3$	$369\cdot10^5$	$199\cdot10^8$	$232\cdot10^3$	$418\cdot10^5$	$226\cdot10^8$
1125	25	$129\cdot10^3$	$243\cdot10^5$	$136\cdot10^8$	$158\cdot10^3$	$295\cdot10^5$	$166\cdot10^8$	$186\cdot10^3$	$348\cdot10^5$	$196\cdot10^8$	$214\cdot10^3$	$401\cdot10^5$	$225\cdot10^8$	$342\cdot10^3$	$454\cdot10^5$	$255\cdot10^8$
1170	26	$135\cdot10^3$	$262\cdot10^5$	$153\cdot10^8$	$164\cdot10^3$	$319\cdot10^5$	$187\cdot10^8$	$193\cdot10^3$	$376\cdot10^5$	$220\cdot10^8$	$222\cdot10^3$	$433\cdot10^5$	$254\cdot10^8$	$252\cdot10^3$	$491\cdot10^5$	$287\cdot10^8$
1215	27	$140\cdot10^3$	$283\cdot10^5$	$172\cdot10^8$	$170\cdot10^3$	$344\cdot10^5$	$209\cdot10^8$	$200\cdot10^3$	$406\cdot10^5$	$247\cdot10^8$	$231\cdot10^3$	$467\cdot10^5$	$284\cdot10^8$	$261\cdot10^3$	$529\cdot10^5$	$321\cdot10^8$
1260	28	$145\cdot10^3$	$304\cdot10^5$	$192\cdot10^8$	$176\cdot10^3$	$370\cdot10^5$	$233\cdot10^8$	$208\cdot10^3$	$437\cdot10^5$	$275\cdot10^8$	$239\cdot10^3$	$503\cdot10^5$	$317\cdot10^8$	$271\cdot10^3$	$569\cdot10^5$	$358\cdot10^8$
1305	29	$150\cdot10^3$	$326\cdot10^5$	$213\cdot10^8$	$183\cdot10^3$	$397\cdot10^5$	$259\cdot10^8$	$215\cdot10^3$	$468\cdot10^5$	$306\cdot10^8$	$248\cdot10^3$	$539\cdot10^5$	$352\cdot10^8$	$281\cdot10^3$	$610\cdot10^5$	$398\cdot10^8$
1350	30	$155\cdot10^3$	$349\cdot10^5$	$236\cdot10^8$	$189\cdot10^3$	$425\cdot10^5$	$287\cdot10^8$	$223\cdot10^3$	$501\cdot10^5$	$338\cdot10^8$	$257\cdot10^3$	$577\cdot10^5$	$390\cdot10^8$	$290\cdot10^3$	$653\cdot10^5$	$441\cdot10^8$
1395	31				$195\cdot10^3$	$454\cdot10^5$	$317\cdot10^8$	$230\cdot10^8$	$535\cdot10^5$	$373\cdot10^8$	$265\cdot10^3$	$616\cdot10^5$	$430\cdot10^8$	$300\cdot10^3$	$697\cdot10^5$	$486\cdot10^8$
1440	32				$202\cdot10^3$	$484\cdot10^5$	$348\cdot10^8$	$238\cdot10^8$	$570\cdot10^5$	$411\cdot10^8$	$274\cdot10^3$	$657\cdot10^5$	$473\cdot10^8$	$310\cdot10^3$	$743\cdot10^5$	$535\cdot10^8$
1485	33				$208\cdot10^3$	$515\cdot10^5$	$382\cdot10^8$	$245\cdot10^8$	$606\cdot10^5$	$450\cdot10^8$	$282\cdot10^3$	$698\cdot10^5$	$519\cdot10^8$	$319\cdot10^3$	$790\cdot10^5$	$587\cdot10^8$
1530	34				$214\cdot10^3$	$546\cdot10^5$	$418\cdot10^8$	$252\cdot10^8$	$644\cdot10^5$	$492\cdot10^8$	$291\cdot10^3$	$741\cdot10^5$	$567\cdot10^8$	$329\cdot10^3$	$839\cdot10^5$	$642\cdot10^8$
1575	35				$221\cdot10^3$	$579\cdot10^5$	$456\cdot10^8$	$260\cdot10^8$	$682\cdot10^5$	$537\cdot10^8$	$299\cdot10^3$	$786\cdot10^5$	$619\cdot10^8$	$339\cdot10^3$	$889\cdot10^5$	$700\cdot10^8$
1620	36				$227\cdot10^3$	$612\cdot10^5$	$496\cdot10^8$	$267\cdot10^8$	$722\cdot10^5$	$585\cdot10^8$	$308\cdot10^3$	$831\cdot10^5$	$673\cdot10^8$	$348\cdot10^3$	$940\cdot10^5$	$762\cdot10^8$

18.2.Design values in ultimate limit states

Annex 2c
Design in accordance with Eurocode 5 (ENV 1995 - 1 - 1:1993) med Swedish NAD.
Design values of material properties (N/mm2) for glulam L 40 (GL 32) in ultimate limit states. Service class 1 or 2[1].
Partial factor for material •$_m$ = 1,2.

Load duration class with shortest duration in the design load combination		Long term[2] (L)			Medium term (M)			Short term (S)		
Examples of loading			Storage			Imposed floor load snow[3]			Snow[3] Wind[3]	
Modification factor (k_{mod})			0,7			0,8			0,9	
		h • 300 mm		h • 600 mm	h • 300 mm		h • 600 mm	h • 300 mm		h • 600 mm
Strength values										
Bending edgewise [4][5]	f_{mx}	22,1		19,2	25,3		19,9	28,5		24,7
flatwise	f_{my}		17,4			19,9			22,4	
Tension parallel to the grain [4]	f_t	15,4		13,4	17,6		15,3	19,8		17,2
Perpendicular to the grain	$f_{t,90}$		0,23			0,27			0,30	
Compression parallel to the grain	f_c		21,0			24,0			27,0	
Perpendicular to the grain	$f_{c,90}$		4,7			5,3			6,0	
Shear parallel to the grain [6]	f_v		2,3			2,7			3,0	
Stiffness values										
Modulus of Elasticity	E_R		6 100			6 900			7 800	
Shear modulus	G_R		400			450			500	

1) In service class 3 the values shall be reduced with 20%.
2) For permanent loads the long term values shall be reduced with 15%
3) Since climatic loads varies upon countries, their load assignment should be defined in national annexes.
4) In design of members with a depth (h) between 300 and 600 mm, the values for h • 600 mm multiplied by the factor $(600/h)^{0,2}$ may apply
5) The values apply to straight elements. For curved elements the values shall be reduced according to chapter 4.7.
6) The values apply to elements with rectangular cross section.
Half the value applies to rolling shear.

18.3.Design values in serviceability limit states

Annex 2d
Design in accordance with Eurocode 5 (ENV 1995 - 1 - 1:1993) with Swedish NAD.
Design values of material properties[1] (Nmm^2) for glulam L 40 (GL 32) in serviceability limit states.

Load duration class Load type		Permanent P	Long term L	Medium term M	Short term S
Examples of loading		Self-weight	Storage	Imposed floor load Snow[2]	Snow[2] Wind[2]
Service class 1					
Modulus of Elasticity	E	8 100	8 650	10 400	13 000
parallel to the grain	E_{90}	300	300	350	450
perpendicular to the grain	G	550	550	700	850
Shear modulus					
Service class 2					
Modulus of Elasticity	E	7 200	8 650	10 400	13 000
parallel to the grain	E_{90}	250	300	350	450
perpendicular to the grain	G	450	550	700	850
Shear modulus					
Service class 3					
Modulus of Elasticity	E	4 350	5 200	7 450	10 000
parallel to the grain	E_{90}	150	200	250	350
perpendicular to the grain	G	300	350	500	650
Shear modulus					

1) The values are calculated according to the formula $E = E_k / (1+ k_{def})$ and with k_{def} according to table 4.1 i Eurocode 5
2) Since climatic loads varies upon countries, their load assignment should be defined in national annexes.

18.4. Design load bearing capacity for glulam

Annex 3b

Design in accordance with Eurocode 5 (ENV 1995-1-1:1993) with Swedish NAD.
Design loadbearing capacity with respect to edgewise bending when lateral buckling is prevented.
Strength class L 40 (GL 32). Service class 1 or 2[1)]. Medium duration of load (M)[2)].
Partial factor for material $\bullet_m$ = 1,2. Size effect according to EC5 is taken into account.

	b = 42 mm		*b* = 56 mm		*b* = 66 mm		*b* = 78 mm		*b* = 90 mm	
h mm	*T* kN	*M* kNm	*T* kN	*M* kNm	*T* kN	*M* kNm	*T* kN	*M* kNm	*T* kN	*M* kNm
180	13,44	5,74	17,92	7,65	21,12	9,02	24,96	10,66	28,80	12,30
225	16,80	8,97	22,40	11,95	26,40	14,09	31,20	16,65	36,00	19,21
270	20,16	12,91	26,88	17,21	31,68	20,29	37,44	23,98	43,20	27,67
315	23,52	17,38	31,36	23,18	36,96	27,32	43,68	32,28	50,40	37,25
360	26,88	22,11	35,84	29,47	42,24	34,74	49,92	41,05	57,60	47,37
405	30,24	27,33	40,32	36,43	47,52	42,94	56,16	50,75	64,80	58,55
450	33,60	33,03	44,80	44,04	52,80	51,91	62,40	61,34	72,00	70,78
495	36,96	39,21	49,28	52,29	58,08	61,62	68,64	72,83	79,20	84,03
540	40,32	45,86	53,76	61,15	63,36	72,07	74,88	85,17	86,40	98,28
585	40,68	52,97	58,24	70,63	68,64	83,24	81,12	98,37	93,60	113,51
630	47,04	61,12	62,72	81,50	73,92	96,05	87,36	113,51	100,80	130,98
675	50,40	70,17	67,20	93,56	79,20	110,26	93,60	130,31	108,00	150,36
720			71,68	106,44	84,48	125,45	99,84	148,26	115,20	171,07
765			76,16	120,17	89,76	141,62	106,08	167,37	122,40	193,12
810			80,64	134,72	95,04	158,78	112,32	187,64	129,60	216,51
855			85,12	150,10	100,32	176,91	118,56	209,07	136,80	241,24
900					105,60	196,02	124,80	231,66	144,00	267,30
945					110,88	216,11	131,04	255,41	151,20	294,70
990							137,28	280,31	158,40	323,43
1035							143,52	306,37	165,60	353,50
1080							149,76	333,59	172,80	384,91
1125							156,00	361,97	180,00	417,66
1170									187,20	451,74
1215									194,40	487,15
1260										
1305										
1350										
1395										
1440										
1485										
1530										
1575										
1620										

	b = 115 mm		*b* = 140 mm		*b* = 165 mm		*b* = 190 mm		*b* = 215 mm	
h mm	*T* kN	*M* kNm	*T* kN	*M* kNm	*T* kN	*M* kNm	*T* kN	*M* kNm	*T* kN	*M* kNm
180	36,80	15,71	44,80	19,13	52,80	22,54	60,80	25,96	68,80	29,37
225	46,00	24,55	56,00	29,89	66,00	35,22	76,00	40,56	86,00	45,90
270	55,20	35,35	67,20	43,04	79,20	50,72	91,20	58,41	103,20	66,09
315	64,40	47,59	78,40	57,94	92,40	68,29	106,40	78,63	120,40	88,98
360	73,60	60,53	89,60	73,68	105,60	86,84	121,60	100,00	137,60	113,16
405	82,80	74,82	100,80	91,09	118,80	107,35	136,80	123,62	154,80	139,88
450	92,00	90,44	112,00	110,11	132,00	129,77	152,00	149,43	172,00	169,09
495	101,20	107,37	123,20	130,71	145,20	154,05	167,20	177,40	189,20	200,74
540	110,40	125,58	134,40	152,88	158,40	180,17	182,40	207,47	206,40	234,77
585	119,60	145,04	145,60	176,57	171,60	208,10	197,60	239,63	223,60	271,16
630	128,80	167,36	156,80	203,74	184,80	240,12	212,80	276,51	240,80	312,89
675	138,00	192,12	168,00	233,89	198,00	275,65	228,00	317,42	258,00	359,18
720	147,20	218,59	179,20	266,11	212,20	313,63	243,20	361,15	275,20	408,67
765	156,40	246,77	190,40	300,42	224,40	354,06	258,40	407,71	292,40	461,35
810	165,60	276,66	201,60	336,80	237,60	396,94	273,60	457,08	309,60	517,23
855	174,80	308,25	212,80	375,26	250,80	442,27	288,80	509,28	326,80	576,29
900	184,00	341,55	224,00	415,80	264,00	490,05	304,00	564,30	344,00	638,55
945	193,20	376,56	235,20	458,42	277,20	540,28	319,20	622,14	361,20	704,00
990	202,40	413,28	246,40	503,12	290,40	592,96	334,40	682,80	378,40	772,65
1035	211,60	451,70	257,60	549,90	303,60	648,09	349,60	746,29	395,60	844,48
1080	220,80	491,83	268,80	598,75	316,80	705,67	364,80	812,59	412,81	919,51
1125	230,00	533,67	280,00	649,69	330,00	765,70	380,00	881,72	430,01	997,73
1170	239,20	577,22	291,20	702,70	343,20	828,18	395,20	953,67	447,21	1079,15
1215	248,40	622,47	302,40	757,80	356,40	893,12	410,41	1028,44	464,41	1163,76
1260	257,60	669,44	313,60	814,97	369,60	960,50	425,61	1106,03	481,61	1251,56
1305	266,80	718,11	324,80	874,22	382,80	1030,33	440,81	1186,44	498,81	1342,55
1350	276,00	768,49	336,00	935,55	396,00	1102,61	456,01	1269,68	516,01	1436,74
1395		820,57	347,20	998,96	409,21	1177,35	471,21	1355,73	533,21	1534,12
1440		874,37	358,40	1064,45	422,41	1254,53	486,41	1444,61	550,41	1634,69
1485		929,87	369,60	1132,02	435,61	1334.16	501,61	1536,31	567,61	1738,45
1530		987,08	380,80	1201,66	448,81	1416,24	516,81	1630,83	584,81	1845,41
1575		1046,00	392,00	1273,39	462,01	1500,78	532,01	1728,17	602,01	1955,56
1620		1106,62	403,21	1347,19	475,21	1587,76	547,21	1828,33	619,21	2068,90

1) In service class 3 the listed values shall be reduced with 20%.
2) For long term loads (L) the listed values shall be reduced with 15%.
For short term loads (S) the listed values shall be increased with 10%.

18.5.Design load bearing capacity for bolted joints steel to timber

Annex 4b

Design in accordance with Eurocode 5 (ENV 1995-1-1:1993) with Swedish NAD.
Design load bearing capacity for bolted joints steel to timber, loaded in shear[1] (kN/shear plane).
Glulam L 40 (GL 32). Bolts in strength class 4.6. Thickness of steel plate less than half the bolt diameter.
Service class 1 or 2[2]. Medium duration of load (M)[2].
Partial factor for material γ_m = 1,3.

Bolt diameter (mm)	Glulam width (mm)	Force parallel to the grain: Single shear	Force parallel to the grain: Double shear, side member of steel	Force parallel to the grain: Double shear, centre member of steel	Force perpendicular to the grain: Single shear	Force perpendicular to the grain: Double shear, side member of steel	Force perpendicular to the grain: Double shear, centre member of steel
12	42	3,67	4,59	5,68	2,40	3,00	4,19
12	56	4,74	4,74	6,47	3,20	3,83	4,77
12	66	4,74	4,74	6,47	3,77	3,83	5,23
12	78	4,74	4,74	6,47	3,83	3,83	5,23
12	90	4,74	4,74	6,47	3,83	3,83	5,23
12	115	4,74	4,74	6,47	3,83	3,83	5,23
12	140	4,74	4,74	6,47	3,83	3,83	5,23
12	165	4,74	4,74	6,47	3,83	3,83	5,23
12	190	4,74	4,74	6,47	3,83	3,83	5,23
12	215	4,74	4,74	6,47	3,83	3,83	5,23
16	42	4,67	5,84	8,73	2,94	3,67	6,61
16	56	6,23	7,79	9,75	3,92	4,90	7,04
16	66	7,34	8,24	10,65	4,62	5,77	7,50
16	78	8,24	8,24	11,23	5,46	6,53	8,14
16	90	8,24	8,24	11,23	6,30	6,53	8,86
16	115	8,24	8,24	11,23	6,53	6,53	8,91
16	140	8,24	8,24	11,23	6,53	6,53	8,91
16	165	8,24	8,24	11,23	6,53	6,53	8,91
16	190	8,24	8,24	11,23	6,53	6,53	8,91
16	215	8,24	8,24	11,23	6,53	6,53	8,91
20	42	5,56	6,95	12,69	3,37	4,21	9,27
20	56	7,42	9,27	13,48	4,49	5,62	9,93
20	66	8,74	10,92	14,32	5,30	6,62	10,24
20	78	10,33	12,56	15,52	6,26	7,82	10,78
20	90	11,92	12,56	16,86	7,22	9,03	11,45
20	115	12,56	12,56	17,13	9,23	9,78	13,08
20	140	12,56	12,56	17,13	9,78	9,78	13,33
20	165	12,56	12,56	17,13	9,78	9,78	13,33
20	190	12,56	12,56	17,13	9,78	9,78	13,33
20	215	12,56	12,56	17,13	9,78	9,78	13,33
24	42	6,34	7,92	17,43	3,71	4,63	10,20
24	56	8,45	10,57	18,01	4,94	6,18	13,48
24	66	9,96	12,45	18,68	5,83	7,28	13,59
24	78	9,57	14,72	19,76	6,89	8,61	13,94
24	90	11,77	16,98	21,06	7,94	9,93	14,46
24	115	13,59	17,63	24,04	10,15	12,69	15,93
24	140	17,36	17,36	24,04	12,36	13,48	17,72
24	165	17,36	17,36	24,04	13,48	13,48	18,38
24	190	17,36	17,36	24,04	13,48	13,48	18,38
24	215	17,36	17,36	24,04	13,48	13,48	18,38

1) Listed values apply to single bolts. For multiple bolts in a row parallel to the force the load bearing capacity should be calculated on the base of a reduced number of bolts: $n_{ef} = 2(n / 2)^{0,8}$ where n is the true number of bolts in the row.

2) In other sevice classes and with other types of loading, multiply the listed values by a factor given in the table below

Duration of load	Permanent	Long	Medium	Short	Instanceous
Service class 1, 2	0,75	0,875	1	1,125	1,375
Service class 3	0,625	0,688	0,813	0,875	1,125

3.10.2.10 Shells

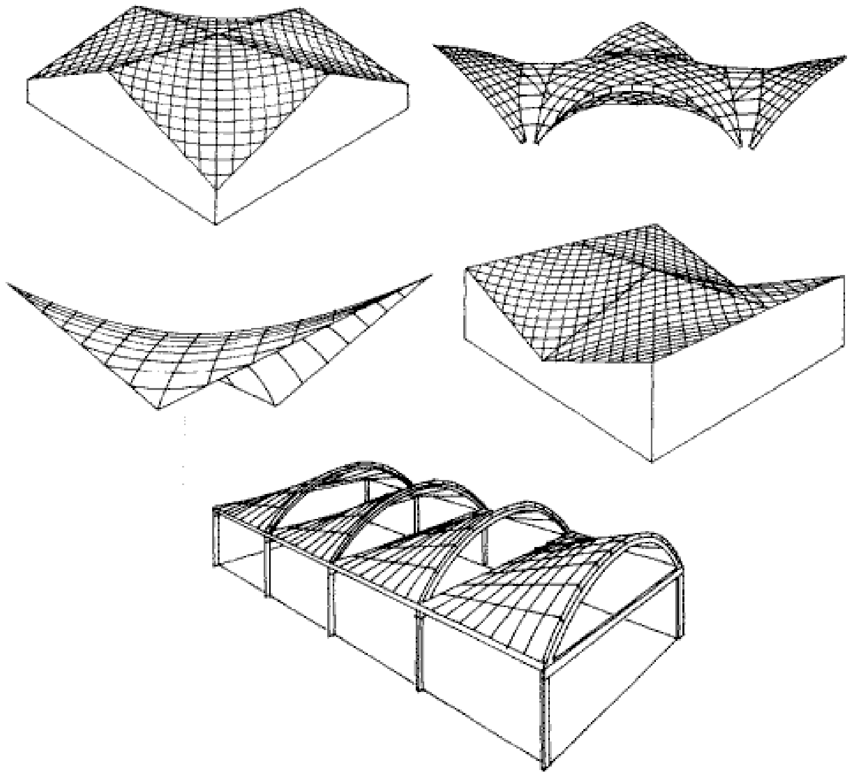


Figure 2.14 Shells.

Shells provide a wide choice of advanced forms and large areas free from columns. By combining several shells of the same type, a variety of roof forms can be achieved. Amongst other shell forms, the dome has already been mentioned in connection with arches conoid and HP shells (hyperbolic parabaloid) are other common types. A valuable characteristic of the two lastnamed types is that they can be generated by straight lines and can thus easily be built up of one or more intersecting layers of timber boarding or corrugated sheet material.

3.11.2.11 Composite systems

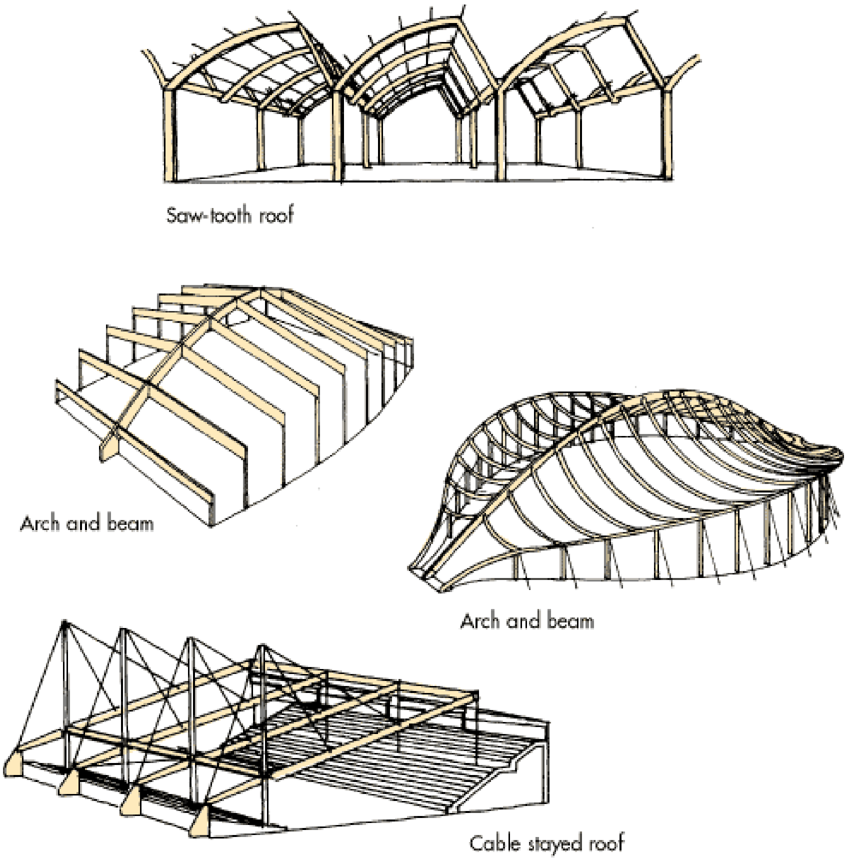


Figure 2.15 Composite systems.

Combinations of various structural systems often provide elegant solutions.

The desire for plenty of daylight in a structure can be satisfied with a sawtooth structure consisting of three-pin trusses placed on continuous beams.

Difficult subsoil conditions can be mastered by concentrating the load reaction forces to a few points, which are reinforced. In the composite arch-beam systems in the two middle figures the main part of the roof load is taken down in the springing points of the arches.

3.12.2.12 Fixtures and connection details

Glulam structures are often visible and constitute an important part of the architecture of the building. This is no less true of the fixtures and connection details. The design of these should therefore receive special attention on the part of the architect.

In the old timber building technology the joints were normally designed to transfer compression forces only and could only to a very limited extent transfer tension. Today, connections and intersections are made with nails, screws and various types of steel fixtures which can equally well transfer tensional and compressive forces.

Steel fixtures transfer forces in a more concentrated and well-defined manner than the old types of timber joints. A hinge, i.e. a joining of two structural components without the ability to take up moments, can really be designed as a true hinge.

Joints shall normally be placed at the intersection of the system lines (the hinge point) so as to avoid unintended moments in the joint.

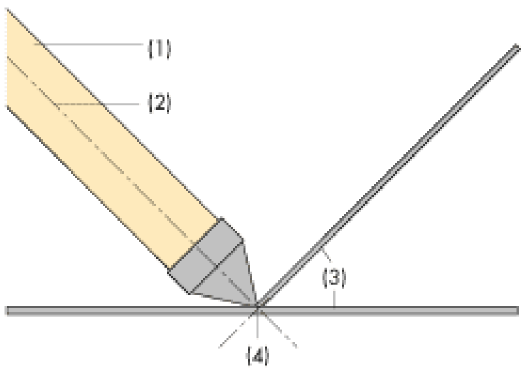


Figure 2.16. System lines should if possible meet at one point, to avoid moments in the node. (1) strut. 2) System line. (3) tie. (4) Theoretical node.

The design of steel fixtures is often dependent on structural limitations, e.g. the contact pressure between steel and glulam.

It is possible to design hidden or external fixtures which act as hinges or which transfer moments. Examples of connections are:

- Foundation details
- Bearing details such as the joint column-beam or beam-beam
- Nodes, i.e. the joints and connections between glulam components or ties which meet at one point

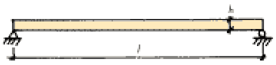
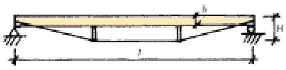
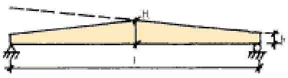
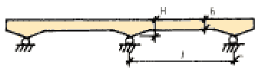
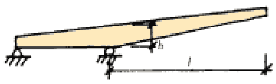
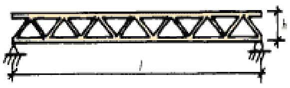
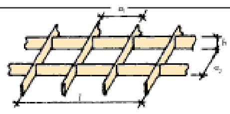
There are standard fixtures on the market such as nailplates, angles, hangers and steel strip. Normally, however, forces and glulam sizes are so large that the steel details are better suited to being purpose-made in the workshop.

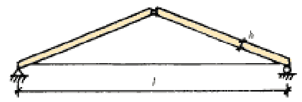
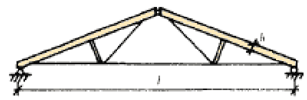
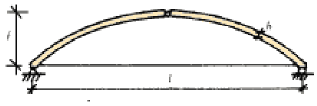
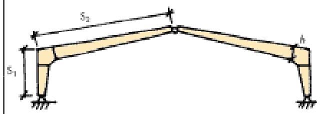
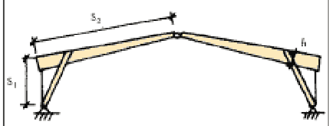
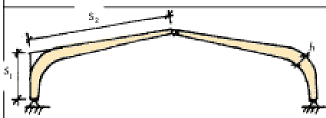
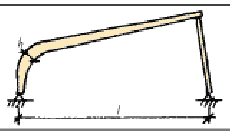
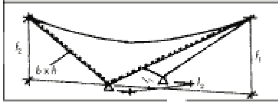
Possible requirements regarding fire protection of steel details must be taken into account. A hidden, built-in fixture is usually better in this respect than an external one (see further chapter Designing for Fire Resistance).

3.13.2.13 Summary table

The summary in table 2.1 covers only the most common types of construction. To simplify design, recommended spans and approximate constructional heights are given for various types of structure. They correspond to average values under normal circumstances. Light loading or tight spacing of units mean somewhat lower depths than those in the table. The reverse is also true. The choice of structural system is often influenced by various limitations in production and transport (see 1.4).

Table 2.1 Structural systems using glulam. Recommended roof slopes and spans. Approximate sectional depths with normal loading. Members at 4-10 centres.

SYSTEM SKETCH	NAME	SUITABLE SLOPE	SUITABLE SPAN	DEPTH
	Straight beam on two supports	• 3•	< 30	$h \sim \frac{l}{17}$
	Straight braced beam on two supports	3-30•	< 50	$h \sim \frac{l}{40}$ $H \sim \frac{l}{12}$
	Symmetrical double pitched beam	3-10•	10-30	$h \sim \frac{l}{30}$ $H \sim \frac{l}{16}$
	Symmetrical double pitched beam with curved underside	3-15•	10-20	$h \sim \frac{l}{30}$ $H \sim \frac{l}{16}$
	Straight continous beam on several supports	• 3•	< 25	$h \sim \frac{l}{20}$
	Haunched continous beam on several supports	• 3•	< 25	$h \sim \frac{l}{24}$ $H \sim \frac{l}{16}$
	Cantilevered beam on two supports	< 10•	< 15	$h \sim \frac{l}{10}$
	Straight trussed beam on two supports	• 3•	30-85	$h \sim \frac{l}{10}$
	Grid	• 3•	12-25	$h \sim \frac{l}{20}$ ($a = 2{,}4 - 7{,}2$ m)

SYSTEM SKETCH	NAME	SUITABLE SLOPE	SUITABLE SPAN	DEPTH
	Three-pin frame with or without a tie	• 14•	15 - 50	$h \sim \frac{l}{30}$
	Three-pin frame with tie and braced struts	• 14•	20 - 100	$h \sim \frac{l}{40}$
	Three-pin (two-pin) arch with or without a tie	$\frac{f}{l}$ • 0,14	20 - 100	$h \sim \frac{l}{50}$
	Three-pin portal frame with finger-jointed haunches	• 14•	15 - 25	$h \sim \frac{s_1+s_2}{13}$
	Knee braced portal frame	• 14•	10 - 35	$h \sim \frac{s_1+s_2}{15}$
	Three-pin portal frame with curved haunches	• 14•	15 - 50	$h \sim \frac{s_1+s_2}{15}$
	Propped half portal frame	• 20•	10 - 25	$h \sim \frac{l}{25}$
	Hyperbolic paraboloid shell (HP shell)	$\frac{f_1+f_2}{l_1+l_2}$ • 0,2	$l_1 \sim l_2$ 15 - 60	$h \sim b \sim \frac{l}{70}$ (kantbalkar)

2.3.1.1.3.1 Manufacturing process

Glulam manufacture is carried out in much the same way regardless of manufacturer or country. Figure 1.2 shows, schematically, a sketch of the manufacture.

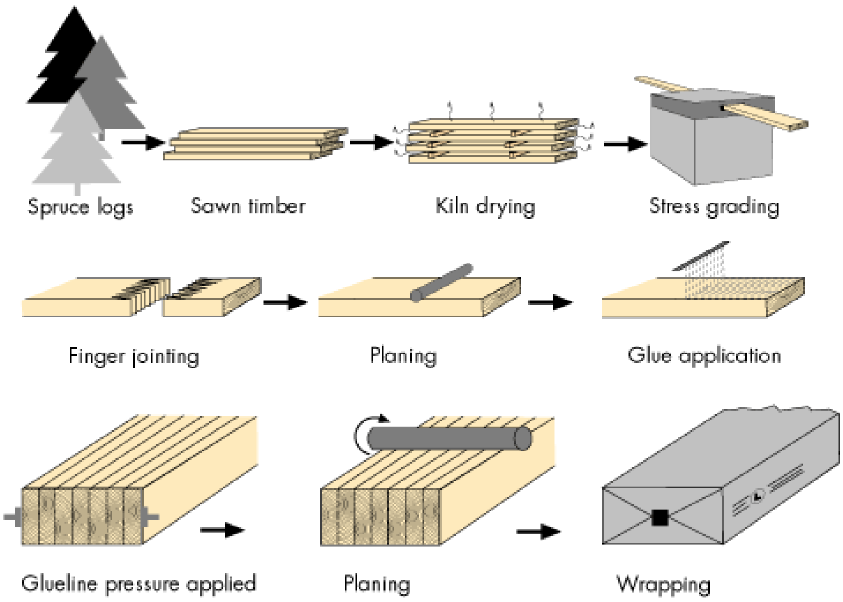


Figure 1.2 Glulam manufacture. Schematic.

The raw material is strength graded timber, in the Nordic countries usually spruce, but for construction expected to be exposed in the long term to damp conditions pressure treated pine is also used. Normally dried, strength graded timber is supplied direct from the sawmill. The moisture content in the laminates shall be 8 15% when they are glued together. The difference in moisture content between adjacent laminates may not exceed about 5%. The strength of the glueline will then be optimal and the moisture content in the finished construction will be balanced, avoiding troublesome splitting. Some fissures will always occur in the timber, but this has generally no ill effects on the load bearing capacity of the construction.

The cross-section of the glulam can be built up of laminates with approximately the same strength,   homogeneous glulam . To utilise the strength of the timber to best advantage, however, it is customary to use timber of higher quality in the outer laminates of the cross-section, where stresses normally are highest,   combined glulam (see figure 1.3). In the factory it is therefore necessary to have space to store at least two strength classes of laminate timber at the same time.

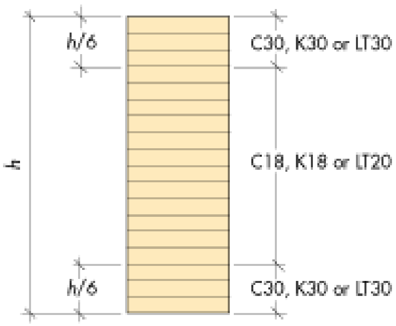


Figure 1.3 Lay-up of combined glulam

Finger jointing joins the timber into laminates. The laminates are cut to the required length and placed on top of each other. For combined glulam, attention must be paid to the placing of the inner and outer laminates. To reduce internal stresses the laminates are turned so that the core sides face the same way throughout the cross-section. The outermost laminates are however always turned with the core side outwards.

The glue in the finger joints is allowed to harden for some hours before the flat sides of the laminates are planed and immediately glued.

The laminate packages are then lifted over to gluing benches and the necessary pressure applied, see figure 1.4. This operation must be carried out before the glue hardens, after an hour or so, the exact time depending on glue type and room temperature. The laminates may be bent when the pressure is applied, producing cambered or curved forms. The glue then hardens in controlled moisture and temperature conditions, possibly with the application of heat. Straight beams can alternatively be produced in a continuous high frequency press.

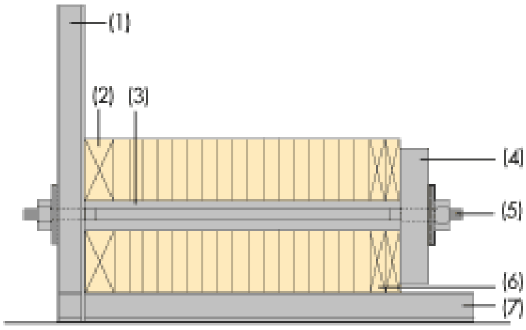


Figure 1.4. Schematic section through gluing bench. (1) Vertical stop. (2) Pressure distributing base, possible camber pattern. (3) Distance piece. (4) Compression block. (5) Tension screw. (6) Pressure distributing boards. (7) Horizontal stop.

When the glue joints have hardened, the pressure is released and the glulam components are lifted from the benches to a planning machine where the sides are planed to the required degree of finish.

Then follows the final working of the component, e g fine sawing of arrises (see figure 1.5), hole drilling and pre-drilling for connectors. Exceptionally, components receive a surface finish in the factory. Finally the components are checked visually and marked before being wrapped and loaded for transport to the building site or to storage of finished goods.

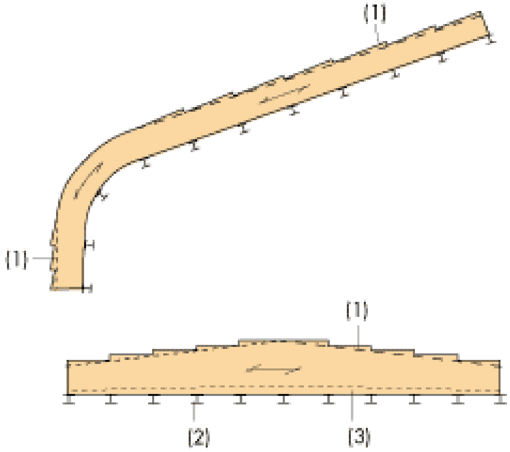


Figure 1.5. Structural component with varying sectional height. (1) Trimming. (2) Vertical stop. (3) Pressure distributing base, possible camber jig.

Manufacture is supervised by the firm's controller (who records conditions of decisive importance to the quality of the products, such as the moisture content of the laminate timber, temperature and moisture content in the gluing hall, time of gluing and lifting).

2.3.2.1.3.2 L-timber, certified glulam

In the Nordic countries L-marked glulam is the accepted term for glulam with at least four laminates which has been manufactured, controlled and marked in accordance with certain rules. The rules are virtually identical in all the Nordic countries and co-operation is through a common organization   the Nordic Glulam Control Board. Glulam manufactured in accordance with these rules complies with the requirements in EN 386 Glued laminated timber   Performance requirements and minimum production requirements.

Certified glulam is marked in the same way in the Nordic countries with the L-mark (see figure 1.6a). See below under Manufacturing control and marking. Certain countries can in addition demand other types of marking, e.g. in Sweden the trident mark is obligatory on glulam, while in Germany a Gütezeichen is required.

Several of the Nordic glulam producers are certified in accordance with ISO 9002.

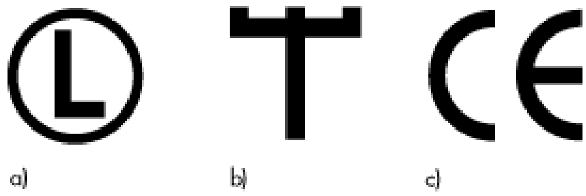


Figure 1.6. The L-mark (a) is used by glulam producers to show that the manufacture of the product has been controlled and approved. Glulam used in Sweden shall in addition be marked with the Boverket trident mark (b). In the future, this marking may be replaced - or complemented by - the CE mark (c).

2.3.3.1.3.3 Glued Structural Timber

Glulam with fewer than four laminates, but which in other respects has been manufactured and controlled in accordance with the same rules as for L-marked glulam is designated Glued Structural Timber (in Sweden called Limmat konstruktionsvirke). It has the same characteristics as L-marked glulam with the exception that its strength is somewhat lower.

2.3.4.1.3.4 Strength and stiffness

Glulam has in general the same strength characteristics as ordinary structural timber.

- Strength varies with the angle between the load and the direction of the grain (anisotropism)
- Strength diminishes with increased moisture content
- Strength diminishes with increased duration of loading
- Wide variation in material characteristics, both within a single component and between components (heterogeneous)

In comparison with corresponding components of structural timber, components of Glulam have however higher strength and a smaller spread of strength characteristics. This lamination effect is usually explained as follows:

Critical for the strength of structural timber is the strength of the weakest cross-section usually at a knot, finger joint or similar. The difference between boards is therefore considerable. In a glulam beam, however, laminates with differing strengths are mixed and the risk that several laminates with major flaws should occur in the same beam is minimal.

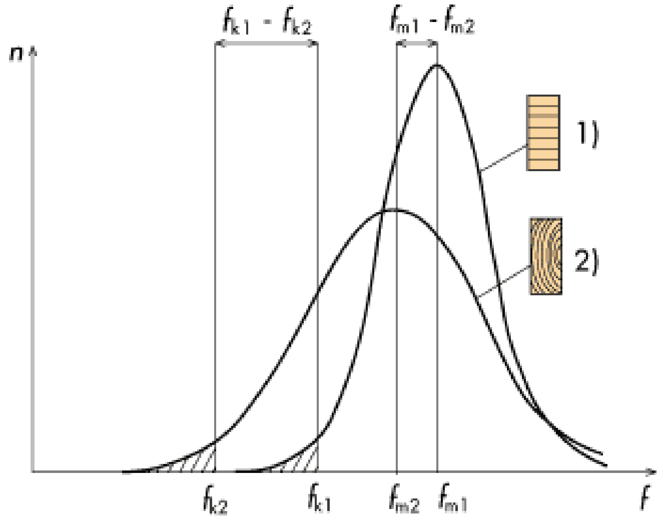


Figure 1.7. Structural components of glulam have higher average strength and a smaller spread of strength characteristics than corresponding components of structural timber.

Typical for glulam beams which are tested to breakage under laboratory conditions, i.e. short-term loading and a moisture content of about 12%, are very brittle failures almost always caused by a knot or a finger joint on the tension side of the beam. Crushing of the timber on the compression side can sometimes precede the final failure, though without changing its brittle character. A brittle failure means amongst other things that a redistribution of loading does not have time to occur, and the load bearing capacity is used up when the stress at a certain point exceeds a critical value. Since the probability that a beam contains a defect capable of causing failure increases with an increase in the volume of the beam, the strength of large beams tends to be lower than that of small beams. This "volume effect" (size effect) is quite well documented under short-

term testing in the laboratory, while it has so far only been incompletely studied under long-term loading.

When designing glulam structures and timber structures in general the starting point is a characteristic strength or stiffness value arrived at by testing a large number of test pieces to failure in laboratory conditions.

The results of this type of testing can be shown with the aid of a frequency diagram for the ultimate strength (see figure 1.8). It is then possible with an acceptable level of accuracy to adapt a statistical distribution   usually normal distribution   to the resulting frequency diagram, or at least to its central part.

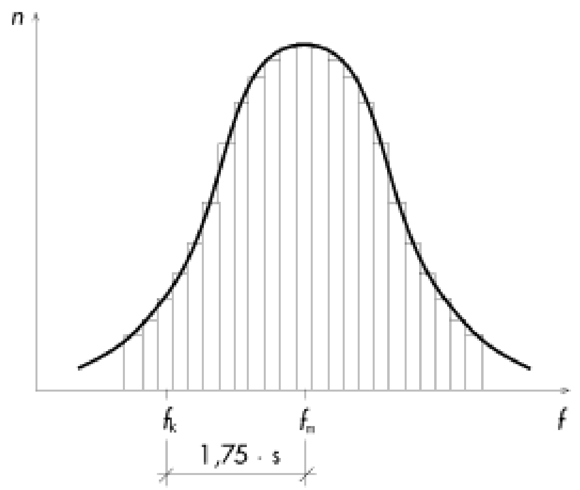


Figure 1.8. Example of a frequency diagram with superimposed normal distribution curve.

If it is assumed that the breaking strength is normally distributed, it is possible to calculate the characteristic value f_k using the formula:

$f_k = f_m - c \cdot s$

where

f_m is the mean, s is the standard deviation and c a coefficient whose value depends on how the characteristic value is defined. The standard deviation is a statistical measure of the spread between the measured values. The characteristic strength of the material thus depends not only on the average value but also to a large extent on the spread. Normally, dimensioning for strength is based on the value of the lower 5% fractile, i.e. the value below which, statistically, 5 of the actual values out of 100 will fall. If the number of tests is large, c = 1,75 in this case.

Characteristic stiffness (module of elasticity, module of shear) is calculated in the same way, but with the mean instead of the 5% fractile as the starting point.

2.3.5.1.3.5 Strength classes

Glulam which has been manufactured in accordance with the rules given in national or European rules is placed in strength classes. The appropriate class is determined by the strength of the timber used and its position in the cross-section. The strength and stiffness values of the various strength classes are then given in the rule in question.

Glulam manufactured in the Nordic countries is normally of strength class GL32 in accordance with Eurocode 5 (EC 5).

It approximates to the earlier class L40. Glulam manufactured by other methods than according to the regulations contained in the rules, e.g. with a different layup of the cross-section, can be used structurally if it is an approved product.

2.3.6.1.3.6 Glue types

The glue used in glulam manufacture has documented high strength and durability under long-term loading. Only glues on which long-term practical experience exists are used. The formal requirements are given in EN 301 which classifies two types of glue, I and II. Glue type I may be used for glulam construction in any climate class, while glue type II is limited to climate classes 1-2. A list of approved glues is kept by the Nordic Glulam Control Board, an organization for co-operation between the control organizations in the Nordic countries.

Earlier, mostly synthetic two-component glue of PRF type (phenol-resorcinol-formaldehyde) was used in glulam manufacture. All PRF glue used for glulam manufacture belongs to type I, which is approved for use in any climate class, i.e. both in- and outdoors. PRF glue gives dark red-brown joints.

MUF glue (melamine-urea-formaldehyde) is now being used to an increasing extent. MUF glue belongs, like PRF glue, to type I. Melamine-glued joints are light when new but darken later.

For finger jointing of the laminates either the dark phenol-resorcinol-formaldehyde glue or the lighter, melamine glue is used. Finger joints can thus appear as dark patches or thin lines on the faces of the component.

Marking on the component shall show which type of glue has been used (I or II in accordance with EN 301).

Continuous development is taking place in the field of glue and new types of glue are being introduced. Thus, a single-component polyurethane glue appears in the Nordic Glulam Control Board list where it is classed as II (climate class 1-2). Approval is coupled to the condition that the manufacturer shall have used the glue in the factory with good results. Experience from practical use shall be documented and presented to Nordic Glulam Control Board.

2.3.7.1.3.7 Manufacturing control and marking

Glulam manufacture demands great care, e.g. during the cutting of the finger joints, preparation and application of the glue, application of pressure, measurement of pressing time etc. To guarantee an even and high product quality, the manufacturer must have a well-documented system of quality control, with a continuous internal control which ensures that samples are regularly taken to check the strength of glue joints and durability.

The quality system shall be approved by a special certification organization and the internal control shall be monitored by an external, independent inspection body which makes unannounced inspection visits to the factory.

In the Nordic countries, glulam is marked with the L-mark , see figure 1.6. In addition, each glulam component shall be marked with:

- Manufacturer's name or other identification

- Strength class

- Glue type (I or II in accordance with EN 301)

- Production week and year or similar identification

- Manufacturing standard (EN 386)

Glulam exported to other European countries may also need to be marked in accordance with the importing country's rules. Thus in Germany it must be marked with a Gütezeichen .

Glulam components are generally marked with a stamp on the ends, but signs nailed or glued on also exist. Examples of marking are shown in figure 1.9. The marking or the delivery note must also state the name of the certifying organization giving the right to mark, and the number of the certificate.

Figure 1.9 Examples of marking.

2.4.1.1.4.1 Production standard - Stock Sizes

Straight glulam components of rectangular cross-section are normally made of 45 mm thick laminates in widths corresponding to the sawmills' standard range. After planing of the sides, the finished width is a few millimetres less than the width of the laminates. The exact size depends on whether the sides are planed and sanded or only planed, in which case occasional patches of unplaned laminate are accepted. Table 1 shows widths in accordance with the industry standard which has been established in Sweden (swedish standard SS 23 27 21).

Sizes marked in **bold figures**are stocked and should be first choice, especially if only a few beams are involved. The sizes apply to components with planed surfaces (see 1.5.3) at a moisture content of 12%.

Annex 1 gives cross-sectional properties for rectangular sections in accordance with SS 23 27 21. Curved components can, depending on the radius, demand thinner laminates than 45 mm (see 4.3.6).

Glued	42×90	56×90	66×90	78×90	**90×90**	115×90	140×90	165×90	190×90	215×90
Struct. Timber	42×135	56×135	66×135	78×135	90×135	**115×135**	140×135	165×135	190×135	215×135
L-marked	**42×180**	56×180	66×180	78×180	90×180	115×180	140×180	165×180	190×180	215×180
glulam	**42×225**	**56×225**	66×225	78×225	**90×225**	115×225	140×225	165×225	190×225	215×225
	42×270	**56×270**	66×270	78×270	**90×270**	115×270	140×270	165×270	190×270	215×270
	42×315	56×315	**66×315**	78×315	**90×315**	**115×315**	140×315	165×315	190×315	215×315
	42×360	56×360	66×360	78×360	90×360	115×360	140×360	165×360	190×360	215×360
	42×405	56×405	66×405	78×405	**90×405**	**115×405**	140×405	165×405	190×405	215×405
	42×450	56×450	66×450	78×450	90×450	115×450	140×450	165×450	190×450	215×450
	42×495	56×495	66×495	78×495	90×495	**115×495**	140×495	165×495	190×495	215×495
	42×540	56×540	66×540	78×540	90×540	115×540	140×540	165×540	190×540	215×540
	42×585	56×585	66×585	78×585	90×585	115×585	140×585	165×585	190×585	215×585
	42×630	56×630	66×630	78×630	90×630	**115×630**	140×630	165×630	190×630	215×630
	42×675	56×675	66×675	78×675	90×675	115×675	140×675	165×675	190×675	215×675
		56×720	66×720	78×720	90×720	115×720	140×720	165×720	190×720	215×720
		56×765	66×765	78×765	90×765	115×765	140×765	165×765	190×765	215×765
		56×810	66×810	78×810	90×810	115×810	140×810	165×810	190×810	215×810
			66×855	78×855	90×855	115×855	140×855	165×855	190×855	215×855
			66×900	78×900	90×900	115×900	140×900	165×900	190×900	215×900
			66×945	78×945	90×945	115×945	140×945	165×945	190×945	215×945
				78×990	90×990	115×990	140×990	165×990	190×990	215×990
				78×1035	90×1035	115×1035	140×1035	165×1035	190×1035	215×1035
				78×1080	90×1080	115×1080	140×1080	165×1080	190×1080	215×1080
					90×1125	115×1125	140×1125	165×1125	190×1125	215×1125
					90×1170	115×1170	140×1170	165×1170	190×1170	215×1170
					90×1215	115×1215	140×1215	165×1215	190×1215	215×1215
						115×1260	140×1260	165×1260	190×1260	215×1260
						115×1305	140×1305	165×1305	190×1305	215×1305
						115×1350	140×1350	165×1350	190×1350	215×1350
							140×1395	165×1395	190×1395	215×1395
							140×1440	165×1440	190×1440	215×1440
							140×1485	165×1485	190×1485	215×1485
							140×1530	165×1530	190×1530	215×1530
							140×1575	165×1575	190×1575	215×1575
							140×1620	165×1620	190×1620	215×1620

Table 1:1 Glulam cross-sections for straight components in accordance with SS 23 27 21. Nominal sizes corresponding to planed surfaces. Stock sizes marked in bold figures. Glulam with fewer than 4 laminates is designated Glued structural timber.

2.4.2.1.4.2 Maximum cross-sectional sizes

Maximum width (lateral size measured parallel with the plane of the glue joint) is restricted by the simple fact that it is difficult to obtain timber wider than 225 mm. After planing this corresponds to a nominal width of 215 mm. Components up to 500 mm wide have been produced by edge-gluing laminates or by gluing components together.

Maximum depth (cross-sectional size measured at right angles to the plane of the glue joint) is limited to about 2 m by the availability of planing equipment. Larger depths can be achieved by various expedients, such as gluing the ridge part of a double pitched beam on at a later stage, after planing. Up to 3 m deep glulam beams have been made in this way.

2.4.3.1.4.3 Maximum length

Glulam factories exist in the Nordic countries which can produce up to 60 m long components. In practice, however, length is restricted by transport considerations (see 1.4.4 Erection and handling and 1.6 Transport and erection).

2.4.4.1.4.4 Erection and handling

A high utilisation of material often means deep and narrow beams which can be difficult to handle on the site. For practical reasons the width should not be less than 1/10 of the depth.

2.4.5.1.4.5 Camber

Simply supported glulam beams can, if the span is greater than 6 8 m, need to be cambered to reduce any problems caused by deflections. A moderate camber of up to 200 mm can easily be arranged during manufacture. Recommendations on the size of the camber are given in 2.3.2.

2.4.6.1.4.6 Permitted Deviations

Glulam components are manufactured with the same accuracy as rolled steel sections or concrete components. Permitted deviations are given in a European Standard, EN 390.

2.5.1.1.5.1 Timber

Glulam is manufactured from strength graded timber, which while it does mean a reduction in knot sizes certainly does not mean that the timber is free from knots. Timber of high strength can also contain quite large knots   in the middle laminates the permitted knot diameter can be as large as the thickness of the laminate.

2.5.2.1.5.2 Glue joints

For the manufacture of structural glulam, glues with high strength and good durability are used (see 1.3.6). Normally used are phenol-resorcinol glue, which gives dark joints, or melamine glue which gives light joints. Melamine joints can however darken with time. Finger joints in the laminates therefore appear as dark patches or thin zig-zag lines on the sides of the components, specially if phenol-resorcinol glue has been used.

Glulam components narrower than approx. 90 mm are normally resawn from wider components. The sawcut can pass through open or glue-filled fissures, which can mean splintering and, especially with phenol-resorcinol glue, clearly visible patches of glue on the sawn surface. Resawn components should therefore be avoided if appearance requirements are high.

2.5.3.1.5.3 Surfaces

When glulam components are lifted from the gluing tables they (and particularly the sides) are uneven and marred by excess glue which has been pressed out of the joints. For practical and aesthetic reasons all four sides are therefore planed before delivery. The customer's requirements on appearance then decide how much is to be planed off. The architect and the structural engineer should agree how much working is suitable in each case. The structural engineer can then design the structure with the real sizes of the members as a starting point.

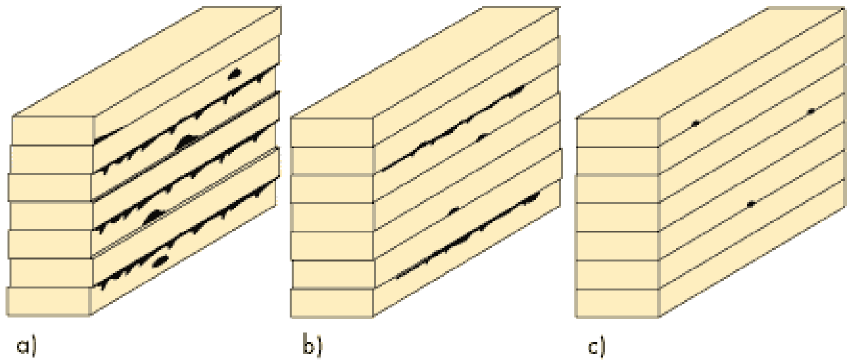


Figure 1.11 Appearance grades, examples. a) equalized surfaces b) industrial appearance grade c) architectural appearance grade

2.5.4.1.5.4 Surface treatment

The purpose of surface treatment is to give the timber surface a certain appearance and to protect the material from sudden humidification or drying-out and thus combat splitting. Film-type finishes such as paint or lacquer also makes the timber easier to keep clean and gives some protection against mechanical damage. Surface finishes can also be used to combat flame spread and smoke in fires.

Glulam can be surface-treated with the same products and methods as ordinary timber, e.g. be stained, painted or lacquered.

The technical, economic and aesthetic conditions determine the choice in each particular case. Note that the dark glue colours have the disadvantage that they fill harmless drying cracks which can appear after the final treatment will be highlighted against a dark surface rather than a light one. Glulam components are supplied untreated unless otherwise agreed.

If the demands on appearance are low, glulam normally needs no further treatment indoors. The structure must then be protected by other means during the building period so that harmful humidification and dirtying are avoided. Out of doors under roofs it is often sufficient to prime with a colourless or pigmented stain, perhaps combined with some kind of chemical protection against discolouring fungal attacks. If the visual requirements are high, one or more coats may be required.

Out of doors, glulam is subject to large variations in moisture content and should be treated to reduce the risk of dangerous splitting. Protection against damp can also be combined here with chemical timber protection. Lasting protection against rot cannot however be provided by surface treatment (see below on timber protection). Outdoor structures of glulam should if possible be protected by being under a roof or by ventilated cladding.

Without external treatment the material ages under the influence of wind and weather and in time acquires a velvet grey surface, in sunny positions tar-brown, which are typical of old wooden buildings. There is at present no surface treatment which in the long term and with a reasonable amount of maintenance preserves the white appearance of fresh timber externally. If the natural ageing of timber is unacceptable in an external structure, a pigmented treatment should be chosen.

All external surface treatment must be maintained. The scope and intervals between maintenance depend on the placing of the structure and the type of surface treatment chosen. Thus stains demand a shorter interval than opaque paints, but are on the other hand easier to maintain. In order that the surface should not become darker each time it is treated, maintenance of stained surface can be carried out using colourless or diluted stain.

2.5.5.1.5.5 Timber preservation

Timber is an organic material and under certain circumstances can be attacked by fungi or destructive insects. In each particular case this can seem to be a disadvantage but seen from an ecological point of view it must be counted as one of the major advantages of the material. During the lifetime of the building the structure must, however, be protected against such attacks. This is done primarily by detail design which ensures that the conditions which produce rot do not arise.

This "design for timber protection" is based on keeping the timber dry (moisture content less than 20%). If this is not possible the construction must be designed so that the timber can dry out after wetting. Dry timber does not rot.

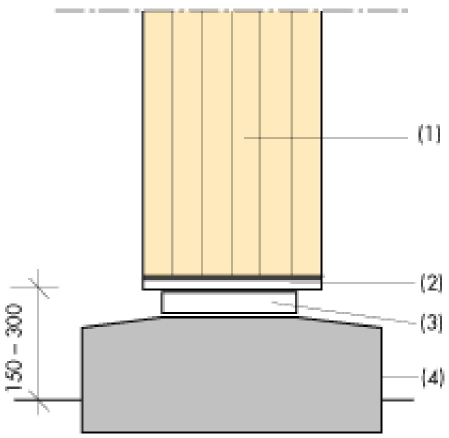


Figure 1.12. Design for timber protection. Example. 1) Glulam column. 2) Moisture barrier, i.e. oil treated hard board. 3) Bearing plate of steel. 4) Concrete foundation.

Another effective method of protecting timber against rot is to pressure-treat the timber with suitable protective fluid. Pressure treatment can however never replace correct detail design, and should only be seen as a complement, since damp in the structure gives rise to other problems besides rot.

The choice of treatment fluid and the demands on penetration and absorption are a compromise between the desire of the building owner for effective protection against rot and the demands of society for a non-poisonous and healthy environment. The use of chemical protection is regulated both in law and in various standards.

Glulam can be manufactured from laminates which have been pressure treated before planing and gluing. After gluing, the sides can either be left unworked or planed in the normal way at the factory. In both cases the protective effect can be improved by applying a suitable product.

Glulam can also be pressure treated after gluing but the component size is then limited by the size of the pressure treatment equipment. On account of the risk of splitting, pressure treatment should be carried out using oil-based products or creosote. The latter should, for reasons of workers' safety, be used with great care. Creosote-treated timber is not allowed in building construction.

2.5.6.1.5.6 Protection during transport, storage, erection

As a protection against rain/snow and dirt during transport, storage and erection glulam components intended to be visible are delivered wrapped, either individually or in packages, in plastic film or paper. Glulam intended to be built in is normally not wrapped.

The wrapping is not a protection against moisture. Indeed, under adverse circumstances moisture can condense on the inside of the wrapping . The water must then be drained by cutting open the wrapping on its underside.

When storing glulam components on the building site, pay attention to the following:

- Never place glulam components directly on the ground

- Place the components on clean (!) transverse bearers at least 250 mm deep and which ensure good ventilation

- The ground shall be dry and flat so that the components do not warp or become unevenly loaded

- Place clean (!) transverse battens between the components and place the battens vertically above one another

- When storing out of doors, protect the glulam components e.g. with tarpaulins placed on clean (!) battens so that there is satisfactory ventilation under the tarpaulin. Do not let the tarpaulins go down to the ground.

- Avoid long-term storage on the site, especially out of doors

Glulam components can, by agreement, be delivered with edge protection in order to reduce the risk of damage during transport and erection. When lifting by crane, use wide straps and protect the edges of the glulam with metal angles or similar, to avoid lifting marks. Working gloves, straps and other lifting equipment shall be free from loose dirt. Do not walk on surfaces which will be visible after erection!

2.6.1.1.6.1 Transport

Transport is normally by road, where traffic regulations vary from one country to another. This may affect the sizes given below.

Component lengths of up to 9 - 10 m can be transported by ordinary lorry. Components longer than 12 m are transported on trailers with extendable platforms. Components up to 30 - 35 m long can be transported in this way. For small quantities of long beams, the cost of this type of transport is high and it is therefore advantageous if the beams can be divided up into shorter sections transportable on a normal lorry.

Long vehicles require special permission from the authorities. Components up to 30 m are usually no problem. Special transport is usually required if the width exceeds 2,5 m or the total height 4,5 m, which is often the case with frame or arch structures. Where rail or water transport are options, different limits apply. Transport problems can often be solved by dividing the structure into a number of transport sections which are joined on the site.

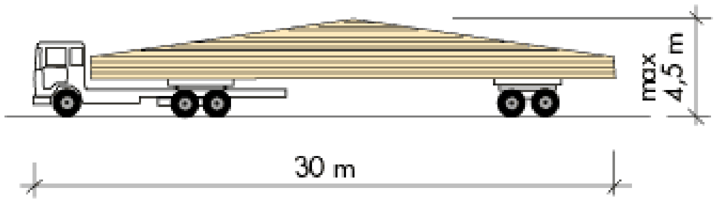


Figure 1.13. If the length of the vehicle exceeds 30 m police escort is usually required.

2.6.2.1.6.2 Erection

Erection of glulam structures almost always requires access to some form of lifting equipment, usually a mobile crane.

The best alternative is to lift the glulam component direct from the lorry to its place in the building. This is however seldom possible and as a rule it is necessary to take into account storage for some time on the site. The instructions in 1.5.6 should then be followed.

On delivery, the number of glulam components and fixtures should be checked against the order.

It is important before the components are unloaded to have planned erection so that time-consuming re-loading can be avoided. Clear and systematic marking of individual glulam components and fixtures is also crucial for effective erection.

Until the stabilisation system of the building is complete, temporary measures must be taken to safeguard the structure against wind and other loads during the building period. Frame and arch structures can best be safeguarded with steel cables, tensioned with turnbuckles. Cables are also used to position the structure correctly until the roof construction or similar is erected.

Plastic wrappings should be cut open at the underside to avoid moisture inside the film. The wrapping can alternatively be removed completely, but the risk that visible elements become dirty during erection must always be borne in mind. Roof structures consisting of corrugated sheets laid direct on the beams are specially vulnerable, as water leakage from joints in the sheeting runs down and dirties the sides of the beams before insulation and roofing felt are in place.

Three-pin frames and arches consist of two parts which are connected to concrete foundations or columns and are joined by steel fixtures at the ridge. Large structures are erected simply and safely with the aid of a mobile crane and a movable erection tower under the ridge. Erection is carried out in such a way that a frame- or half-arch is lifted into place by the mobile crane. The foot or springing point is fixed in a fixture or to the top of a column and the top end is placed on the tower, where it is connected to the other half which has been placed in the same way. As soon as bracing is complete the tower is moved to the next module and the process repeated.

2.7.1.1.7.1 Specification

Example of a specification:

4 no. double pitched beams 165 x 680 x 680/1370/680 x 22000 mm

L40-I   Planed surfaces   Camber 120 mm.

When asking for a quotation or ordering, give also the following:

- Number of identical units (note that a three-pin frame or three-pin arch each consist of two components).
- Possible reference to drawing showing holes, notches etc.
- Special requirements regarding wrapping (e.g. individual wrapping, edge protection etc).
- Delivery timetable.
- Unloading details (crane or loading machine).
- Name and address of the project.

2.7.2.1.7.2 Dimensioning

A precise and clear dimensioning will facilitate the manufacture and reduces the risk for mistakes.

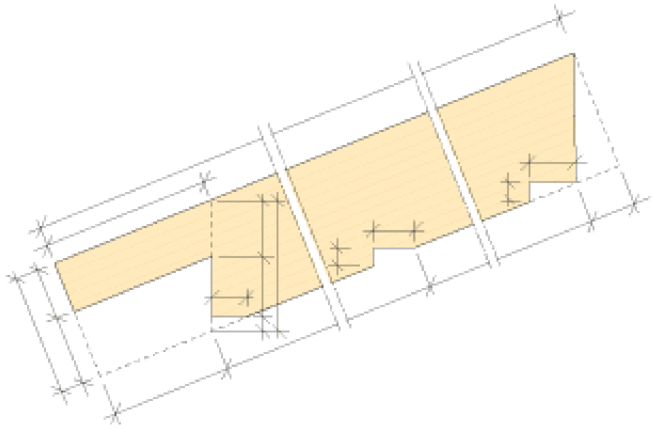


Figure 1.14. Example of dimensioning.

4.2.1.3.2.1 Load Effects

The term Load Effects covers for example deflections and internal forces or moments caused by loads. The design load effect $S_d$is determined on the basis of the design values of the loads in question, placed in the least favourable positions.

The design value of a load is

$$F_d = \gamma_f F_k$$

Formula 3.2a   or

$$F_d = \gamma_f \Psi F_k$$

Formula 3.2 b

where

$F_k$is characterististic and $\Psi F_k$is a normal value of the load, while $\gamma_f$is the partial coefficient for load, sometimes called the load factor.

As a rule the structure is not designed for a single load but for a combination of loads, e.g. self-weight and snow load. In the ultimate limit state the loads can be assumed to have characteristic or frequent values, while in the serviceability limit state it is often only necessary to check for combinations of loads with frequent values.

4.2.2.3.2.2 Safety classes

The risk that failure in the structure will involve serious personal injuries differs for different types of buildings, depending on their use, and for building elements depending on their function. Thus the risk of personal injury is greater in failure of a roof beam than in failure of a stud in a

partition, and greater if the beam supports the roof of a sports hall than if it is in a building for storing timber.

In certain countries, for example in the Nordic countries, these differences are taken into account by assigning load-bearing structures to different safety classes depending on the consequences of a structural failure. The safety class then determines either the extent of control or the value of the partial coefficient γ_n in the ultimate limit state. In the serviceability limit state however no distinction is made between the safety classes.

Assignment to safety classes is not practised at present in the Eurocodes.

4.2.3.3.2.3 Types of load

Stiffness and loading capacity in a timber structure are to a large extent dependent on the duration of the loads acting on the structure. When designing, a difference is therefore made between loads with different time-spans, e.g. between permanent loads such as self-weight and loads whose intensity varies during the life-span of the building. The latter are normally divided into long-term, medium-term and short-term loads. Sometimes there are also momentary instantaneous loads such as an impact.

The building regulations give different strength and stiffness values for loads of different duration.

Loading capacity is calculated on the basis of the material values for the load in a combined loading which has the shortest duration. Deflection is calculated as the sum of the contributions to deflection from each of the loads, each one being calculated according to the duration of that particular load.

The duration class or type of load to which a load is assigned depends to a certain extent on geographic/climatic and cultural conditions. Snow load, for example, is regarded as long-term (normal value) or medium-term (characteristic value) in Sweden, Norway and Finland while Denmark and large parts of the rest of Europe treat it as short-term. The regulations usually provide guidance in this matter.

4.2.4.3.2.4 Service classes

The moisture content of timber has, like the duration of loads, a major influence on the strength and stiffness of the material. Dry timber is both stronger and stiffer than moist timber. The building regulations deal with this by defining a number of service classes, each representing a defined

moisture content interval within the range which is typical for building structures. Strength and stiffness values are then given for each service class.

It is the task of the structural engineer to decide to which class a certain constructional element shall be assigned, on the basis of the conditions in that particular case. The building regulations provide guidance with examples of common parts of the structure. In EC5 and in the national regulations of the Nordic countries the following service classes apply:

*Service Class 1*is characterised by an environment where the relative moisture content only exceeds 65% for a few weeks of the year and never reaches 80%.

This corresponds to a moisture content in the material which only for short periods exceeds 12%. This includes for example:

- floors and trusses in unheated but ventilated attics above permanently heated rooms.
- columns and studs in external walls to permanently heated buildings, provided they are protected by a ventilated and drained cladding
- ground floors above crawling spaces ventilated with indoor air.

Glulam frames covering indoor swimming pools and insulated manèges are generally included in this class.

*Service Class 2*is characterised by an environment where the relative moisture content only exceeds 80% for a few weeks per year.

This corresponds to a moisture content in the material which only for short periods exceeds 16%. This includes for example:

- ground floors above crawling spaces ventilated with outdoor air
- glulam structures in rooms or buildings which are not permanently heated, e.g. weekend cottages, unheated storage space, uninsulated manèges and farm buildings
- glulam structures covering poorly ventilated swimming pools.

*Service Class 3*is characterised by an environment with a higher moisture content than that in Service Class 2. This includes for example:

- glulam construction in rooms or buildings where moisture is generated by processes or storage
- structures which are not protected from damp or are in direct contact with the ground.

Glulam structures in buildings with humidifying equipment are assessed in each case with regard to the extent of the humidification.

In some countries there is also a special class for dry indoor climate, Service Class 0.

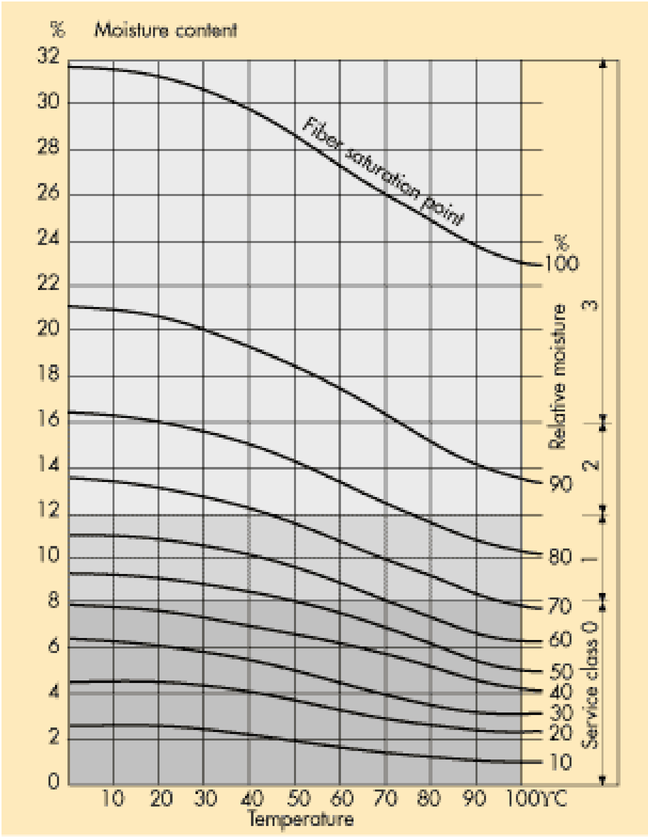


Figure 3.1 Equilibrium moisture content in timber as a function of the temperature at different levels of vapour pressure.

4.2.5.3.2.5 Design in the ultimate limit state

The design load carrying capacity in the ultimate limit state is decided on the basis of the design strength value. This is calculated by dividing the characteristic value f_k, adjusted with regard to type of load and service class, by the partial coefficient γ_m for uncertainty in the material and according to certain regulations - by the partial coefficient γ_n for the consequences of failure.

$$f_d = \frac{k_{\mathrm{mod}} \cdot f_k}{\gamma_m \cdot (\gamma_n)}$$

Formula 3.3

The modification factor k_{mod} is then decided on the basis of the duration of the shortest-term type of load in the design load combination.

According to certain regulations the bending and tension strength shall in addition be corrected with regard to size effects. When curved structural components are designed the bending strength should be further reduced with regard to initial bending stresses in the laminates.

Characteristic basic strength values and modofication factors for various cases are given in current regulations.

The value of γ_m depends, amongst other things on the degree of control during design and manufacture. For L-marked glulam which is designed and erected in a professional manner, a lower value of γ_m may be applied than for structural timber in general.

Annex 2 shows the design material values for glulam in the ultimate limit state.

Annex 3 shows the design load carrying capacity with regard to

shear forces and bending in the depth of the beam for rectangular glulam components.

4.2.6.3.2.6 Design in the serviceability limit state

In the serviceability limit state the structure shall be sufficiently stiff to eliminate unpleasant oscillation or deformation which might impair the function of the building element , e.g. roof drainage. Many regulations contain special rules for checking stiffness in timber floors and roofs.

The stiffness of a glulam component is affected by several other factors, in addition to its geometry, such as the duration of the load and the moisture content and temperature of the material. Above all, variations in load, moisture content and temperature are of great importance.

When deformations are calculated the above can be taken into account by correcting the stiffness values in the regulations for the load duration and the service class. Since one is normally more interested in a correct assessment of the deformation than on a value which with a certain probability is on the safe side, the characteristic stiffness values given in the regulations for deformation calculations correspond to a 50% fractile, i.e. the mean value.

When the loading consists of several loads of different durations, the deformation is calculated as the sum of the various loads' contributions to deformation, each contribution being calculated with the material values corresponding to the duration of that load.

Critical material values for the serviceability limit state are obtained by first correcting the characteristic value of e.g. the Module of Elasticity with regard to loading type and service class. The result is then divided by the partial coefficient $\gamma_m$for uncertainty in the material and   in those cases where the regulation stipulates it   also by the partial coefficient $\gamma_n$for consequences of a failure.

As a rule (γ_n=) γ_m= 1,0 is used when designing in the serviceability limit state, i.e.

$$E_d = k_{mod} \cdot E_k$$

Formula 3.4

where

k_{mod}= modification factor with regard to duration of load and service class in accordance with current regulations

E_k= characteristic basic value of the Module of Elasticity in deformation calculations.

The modification factor $k_{mod}$is given in EC 5 by the formula

$$k_{mod} = \frac{1}{1+k_{def}} \text{ eller } k_{mod} = \frac{1}{1+\Psi_2 k_{def}}$$

Formula 3.5

where

the creep factor is given different values in different service classes and $\psi_2$takes into account the effect of load duration.

Annex 2 shows design values of moduli of elasticity and shear for glulam for serviceability state design.

4.3.1.3.3.1 Camber

The disadvantages of deflections can to some extent be countered by designing the structure with a certain amount of camber. The size of the camber can suitably be equal to the deflection caused by the calculated self-weight plus variable load with a normal value. This can be assumed to give horizontality in use. Camber is relevant above all for freely supported beams with a span exceeding 6 to 8 m. Continuous beams with several supports do not as a rule need camber.

Camber should always be specified when the pitch of a roof is shallow. It should be designed so that there will be a fall to the gullies even under full snow load. Standing water from melted snow, snow and ice can otherwise produce successively increasing deflection and risk of failure in the roof beams. In addition the risk of leakage and water damage is great. A minimum fall of 3° should always be aimed for.

4.3.2.3.3.2 Deflections

Table 3.1 gives normally acceptable deflections in relation to the span for some types of components and in various uses. The values are the same as the corresponding recommendations in EC 5. The information can in suitable parts also be used for arches, frames and other structures.

For components without camber, the deflection under the total load should be further limited, e.g. to 2/3 of the value in the table.

Table 3.1 Normally acceptable deflection in relation to the free span. Serviceability limit state.

Use	**Variable load** (normal value)	**Total load** (incl. Self-weight)
Roof beams		
Industrial buildings	1/200	1/150
Schools, shops etc.	1/250	1/200
Floor beams[1)]		
Normally	1/500	1/300
Storage and other premises Not open to the public	1/200	1/150
Trusses		
Detailed calculations	1/250	1/200
Approximate calculations	1/500	1/400
Cantilevers	1/250	1/200
Purlins		
Without separate false ceiling	1/250	1/200
With separate false ceiling	1/150	1/100

1) Stiffness of the timber floor structure shall also be checked for vibration.

Connection to non-loadbearing internal walls shall be designed in such a way that the floor structure can deflect freely without transferring loads to the wall. If this is not done the wall may be damaged and/or may overload floors below. Secondary structures can generally be designed with reduced requirements on limitation of deflection than the main structure.

However, this must not lead to impairment of the function, e.g. through rupture of the roofing material.

5.2.1.4.2.1 Straigth beams

The critical moment for a simply supported straight beam of constant section which is restrained against buckling at the supports and loaded with a constant moment is

$$M_{crit} = \frac{\pi}{l} \sqrt{EI_y \cdot GK_v}$$

Formula 4.5

where

l= the span

$$I_y = \frac{b^3 h}{12}$$

$$K_v = \frac{b^3 h}{3} \left(1 - 0,63 b / h\right)$$

Formula 4.6

E, G = design values for stiffness in accordance with current regulations

In other cases of support and loading, the critical moment can be written as

$$M_{crit} = \frac{\pi}{l_e} \sqrt{EI_y \cdot GK_v}$$

Formula 4.7

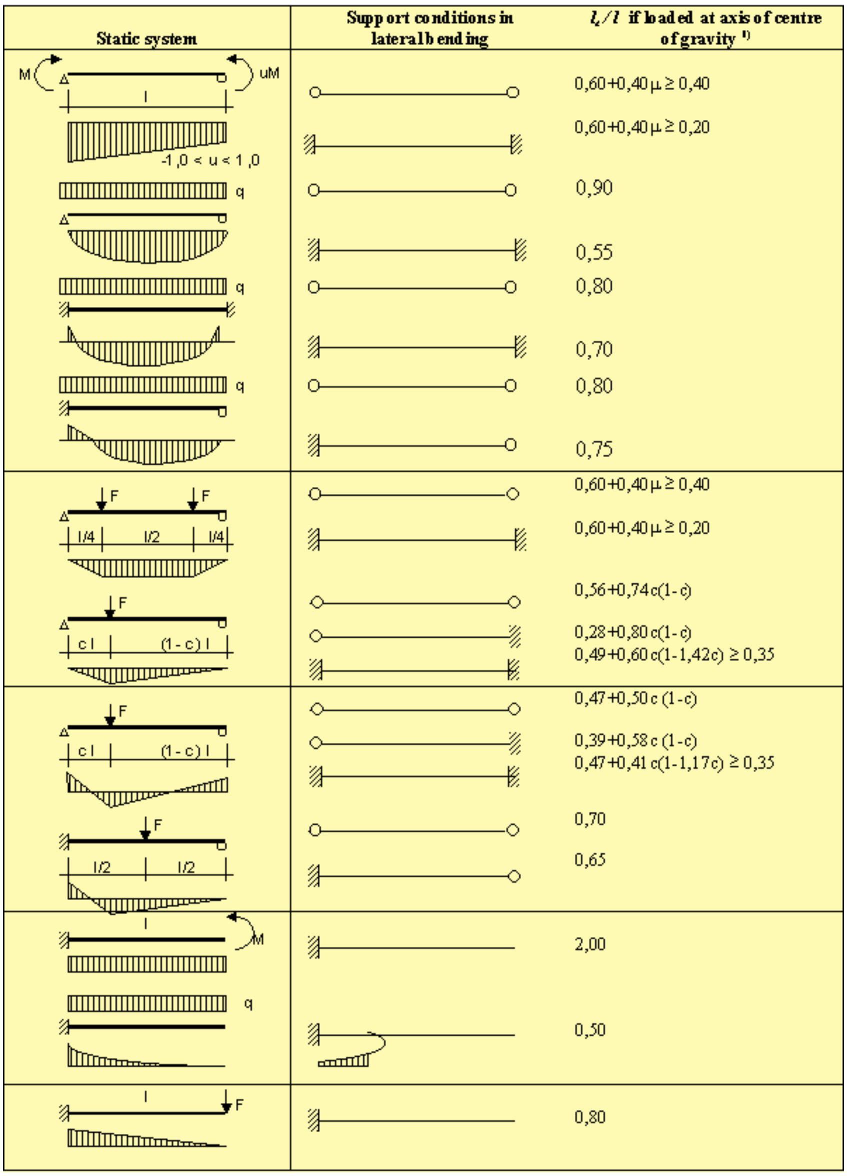
where

*l*e/sub> = is the effective length in accordance with table 4.1 or 4.2. If the critical moment is expressed thus, the slenderness ratio will be

$$\lambda_m = 0,07 \frac{\sqrt{l_e h}}{b}$$

Formula 4.8

Table 4.1 Effective beam length $l_e$in relation to real beam length *l*(from NS 3470)

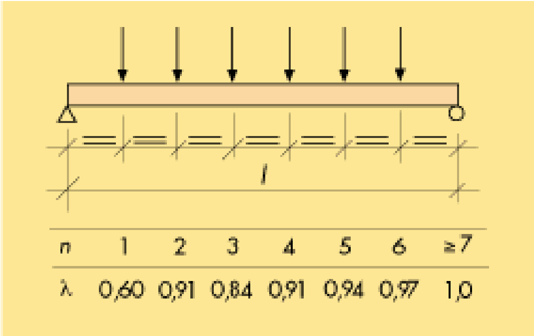


Static system	Support conditions in lateral bending	l_e/l if loaded at axis of centre of gravity [1]
		$0,60+0,40\mu \geq 0,40$
		$0,60+0,40\mu \geq 0,20$
		0,90
		0,55
		0,80
		0,70
		0,80
		0,75
		$0,60+0,40\mu \geq 0,40$
		$0,60+0,40\mu \geq 0,20$
		$0,56+0,74c(1-c)$
		$0,28+0,80c(1-c)$
		$0,49+0,60c(1-1,42c) \geq 0,35$
		$0,47+0,50c(1-c)$
		$0,39+0,58c(1-c)$
		$0,47+0,41c(1-1,17c) \geq 0,35$
		0,70
		0,65
		2,00
		0,50
		0,80

[1] If the loading is at the top of the beam, l_e shall be increased by twice the beam depth. If however it is at the bottom of the beam, l_e shall be reduced by half the beam depth.

For a beam whose compression edge is restrained from lateral movement, in its whole length e.g. by roof boarding or profiled sheet material, and which is restrained at the supports against rotation, it can be assumed that $k_{crit} = 1,0$. The fixings of the construction which restrain against lateral movement shall be designed for an evenly distributed lateral force in accordance with 12.3.

For a beam whose load consists of secondary beams which restrain lateral bending of the compression edge, the effective beam length l_e in the expression for λ_m above is determined in accordance with table 4.2. Secondary beams and their fixings shall be designed for an extra force in accordance with 12.3.



n	1	2	3	4	5	6	≥7
λ	0,60	0,91	0,84	0,91	0,94	0,97	1,0

Table 4.2. Effective beam length l_e in relation to total beam length. Simply supported beam, restrained from buckling at supports and loaded with n equal loads. The concentrated loads bear at the top edge of the beam and restrain lateral bending in the beam at the loading points.

Continuous beams can buckle at intermediate supports unless sufficient bracing of the bottom edge is provided. If the top edge is continuously restrained the slenderness ratio λ is calculated with an effective beam length l_e equal to twice the depth of the beam. If the top edge is only restrained at certain points, e.g. by secondary beams, l_e, must be calculated as the least of the distances from support to zero moment point or point of restraint. If special restraint is provided along the bottom edge of the beam, e.g.. in accordance with figure 4.1, l_e can be calculated as 0.3 times the distance from the support to the point of restraint. The beam is assumed in all cases to be restrained from rotation at the supports.

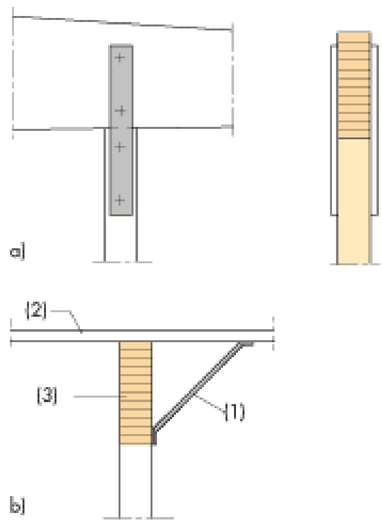


Figure 4.1 Lateral restraint at intermediate support. a) with U-section or heavy fishplate. b) with diagonal bracing. (1) from purlin (2) to bottom edge of beam.

5.2.2.4.2.2 Curved components

Curved components can be treated according to the principles given earlier for straight components.

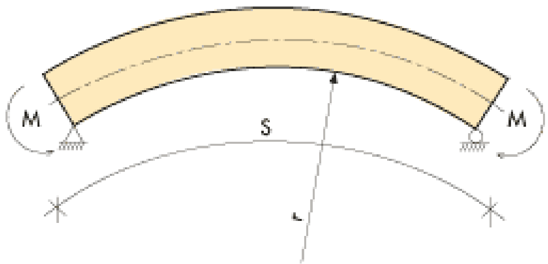


Figure 4.2 Circular arch component. Designations.

For a circular arch component as in figure 4.2, freely supported on two supports and loaded with a constant moment, the critical moment can be calculated according to the formula - provided the supports restrain rotation.

$$M_{crit} = \frac{\pi}{s}\sqrt{EI_y \cdot GK_v} \pm \frac{EI_y + GK_v}{2r}$$

Formula 4.9

where

$$I_y = \frac{b^3 h}{12}$$

and

$$K_v = \frac{b^3 h}{3}\left(1 - 0{,}63b/h\right)$$

The plus sign in equation (4.9) applies to the direction of the moment which causes tension on the convex side. Some curvature in the direction of the moment thus reduces the risk of buckling.

For a curved frame haunch as in figure 4.3 the critical moment and the corresponding slenderness ratio can be calculated using the above formulae with *s*equalling the arch length between two points where the structure is restrained against rotation and laterally braced. If the distance between the braced points is considerably shorter than *s*(see figure 4.3) there will be a tensioning effect which means that M_{crit}can be calculated with a somewhat reduced effective arch length s_e.

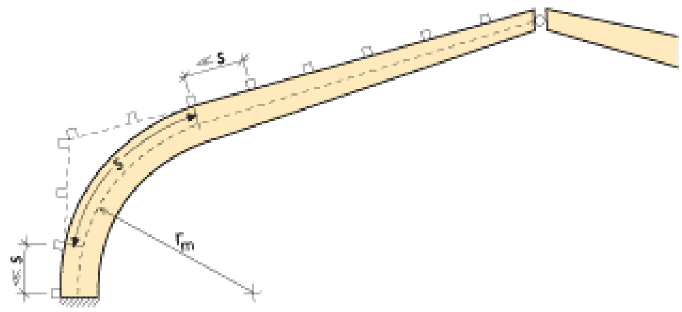


Figure 4.3 Buckling at a curved frame haunch. Symbols.

Distribution of moments --- Magnifying factor *k*--- Conditions

Haunches of frames are usually loaded with both compressive stresses and bending moments. The design criterion in EC 5 is in this case

$$\frac{\sigma_{c,0}}{k_c f_{c,0}} + 0,7 \cdot \frac{\sigma_{m,x}}{k_{crit} k_r f_{m,x}} \leq 1$$

Formula 4.10

where

$\sigma_{c,0}$= design compression stress

k_c= reduction factor with regard to buckling (see 5.1)

f_{c0}= design compression strength

$\sigma_{m,x}$= design bending stress

k_{crit}= reduction factor with regard to lateral buckling (see 4.2)

k_r= reduction factor with regard to curvature of laminates (see 4.7)

5.4.1.4.4.1 Unreinforced notches

The carrying capacity of a notch can be checked using the following method, which is based on fracture mechanics studies at the inner angle of the notch. For the sake of simplicity the criterion has been formulated as a check on the shear stresses in the residual cross section, despite the fact that it is the lateral tension forces which are critical for the carrying capacity:

$$\tau = \frac{1{,}5 \cdot V}{bh_1} \leq k_v f_v$$

Formula 4.14

For beams with the notch at the top. k_v= 1,0.

For beams with the notch at the bottom,

$$k_v = \min \begin{cases} 1{,}0 \\ 6{,}5\left(1 + \frac{1{,}1 i^{1{,}5}}{\sqrt{h}}\right)\frac{1}{\sqrt{h}}\left(\sqrt{\alpha - \alpha^2} + 0{,}8\frac{e}{h}\sqrt{\frac{1}{\alpha} - \alpha^2}\right)^{-1} \end{cases}$$

Formula 4.15

where

h= total beam depth in mm

$\alpha = h_1/h$

$i = a/(h-h_1)$

Other symbols, see figure 4.6.

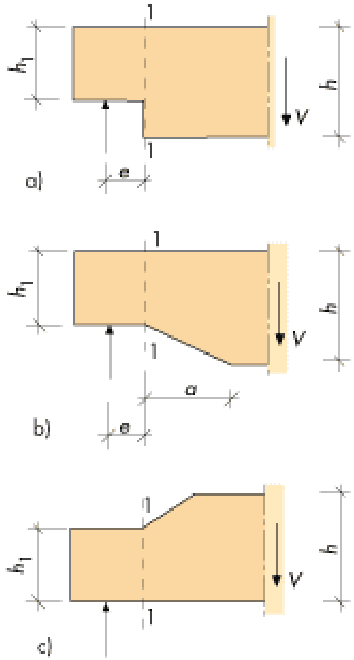


Figure 4.6. Notches in beam end. Symbols.

5.4.2.4.4.2 Reinforced notches

The carrying capacity of beams with reinforced rectangular notches can be checked using the method given above. If the reinforcement is designed in accordance with the following instructions, k_v= 1.0 will apply.

The reinforcement can consist of plywood or a glued-in screw and shall be designed for a tension force $F_s$which is applied at section 2-2 in figure 4.7.

$$F_S = R\left[3\left(\frac{h-h_1}{h}\right)^2 - 2\left(\frac{h-h_1}{h}\right)^3\right]$$

Formula 4.16

where

R= design support reaction.

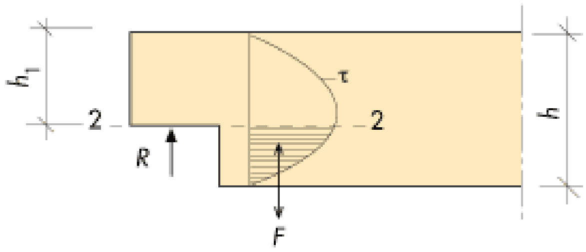


Figure 4.7 Reinforcement of notch in beam end. Symbols.

4.4.2.1 Reinforcement using plywood

Plywood of minimum thickness d= 10 mm is glued on each side of the beam as in figure 4.8. The outer layer is orientated perpendicular to the longitudinal direction of fibres in the beam. During gluing, pressure can be applied with the aid of annular ring-shanked nails or with screws. In either case the plywood shall be pre-bored. Penetration in the glulam shall be at least twice the thickness of the plywood. Fastenings shall be evenly distributed across the plywood sheet and correspond to 1 nail/screw per 6000 mm2. Distance between fastenings should be equal both parallel to and perpendicular to the grain and conform with current regulations. Gluing shall be carried out by the glulam manufacturer in accordance with current regulations for structural gluing.

The carrying capacity of the plywood and glue joint $F_R$is calculated using formula 4.17. The strength values in the regulation $f_t$and $f_v$are reduced to 25 % since the stress distribution in the glue joint is uneven.

$$F_R = \min\begin{cases} 2cd_{eff} \cdot 0,25 f_t \\ 2ch_1 \cdot 0,25 f_v \\ 2c\left(h-h_1\right) \cdot 0,25 f_v \end{cases}$$

Formula 4.17

where

$2d_{ef}$ = combined thickness of ply on both sides of the beam with the direction of the grain perpendicular to the longitudinal direction of the beam.

Other symbols, see figure 4.8.

Design conditions:

$$F_S \leq F_R$$

Formula 4.18

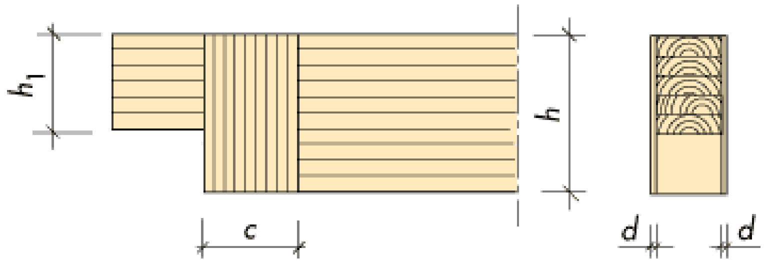


Figure 4.8 Notch in beam end. Plywood reinforcement .

4.4.2.2 Reinforcement using glued-in screws

Glued-in screws may only be used in indoor construction in service class 1 and should generally be avoided if major variations in the moisture content are a threat. The screw/screws are placed as near as possible to the edge of the notch with regard to regulations on minimum edge and centre distance. Also, the glued-in length above the notch shall be at least half the depth of the beam at the support (see figure 4.9). More than two rows of screws, counted along the length of the beam, are not recommended.

The carrying capacity of the screw F_R, regarding tension strength and anchorage, is calculated in accordance with current regulations or approvals.

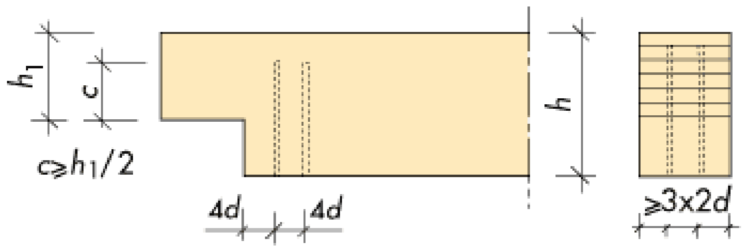


Figure 4.9 Notch in beam end. Reinforcement with glued-in screws.

4.4.2.3 Glass fibre reinforcement

Recent research has shown that reinforcement with glass fibre-reinforced epoxy resin is a usable method.

The glass fibre reinforcement works in the same way as the plywood but has the advantage that it is transparent and is reminiscent of a thick coat of lacquer. Amongst the drawbacks are that experience with the method is so far limited and that no generally accepted calculation method exists. The use of epoxy glue can also be problematic from the point of view of workers' health.

5.5.1.4.5.1 Unreinforced holes

The carrying capacity in moment and shear forces is checked for the remainder of the cross-section. Shear forces can be distributed to the parts above and below the hole in proportion to their stiffness. If the hole is rectangular, the extra moment which the shear force causes in the upper and lower flanges must be taken into account (see figure 4.11). If fewer than four laminates remain over or under the hole the critical bending strength must be reduced by 25%.

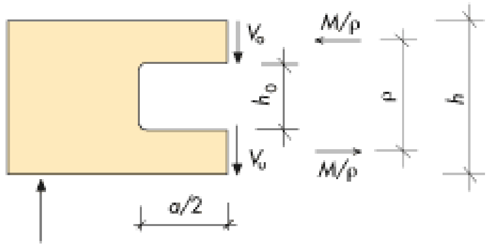


Figure 4.12 When designing rectangular holes it is necessary to take into account the effect of the moment caused by the shear force.

Critical shear strength is reduced with regard to hole size and beam width, where the diameter of round holes (and the diagonal of rectangular holes) represents the size of the hole.

$$D=\sqrt{h_0^2+a^2} \text{ for rectangular holes}$$

Formula 4.19

$$f_{vred}=k_{hole}\cdot k_{vol}\cdot f_v$$

Formula 4.20

where

$$k_{hole}=\begin{cases} 1-555(D/h)^3 & \text{for } D/h\leq 0{,}1 \\ \dfrac{1{,}62}{(1{,}8+D/h)^2} & \text{for } D/h>0{,}1 \end{cases}$$

Formula 4.21

$$k_{vol}=\left(\frac{90}{b}\right)^{0{,}2} \text{ for } 90\leq b\leq 215 \text{ mm}$$

Formula 4.22

The factor k_{vol} takes into account the size-dependence of the strength (compare 4.1).

Beams narrower than 90 mm are normally resawn from twice the width, which in this connection gives an unfavourable orientation of the annual rings on the resawn side. Resawn beams, i.e. beams narrower than 90 mm, are not covered in the studies on which the above design rules are based.

The stress conditions round a hole bear a clear resemblance to those at a notch in a beam end (see figure 4.12).

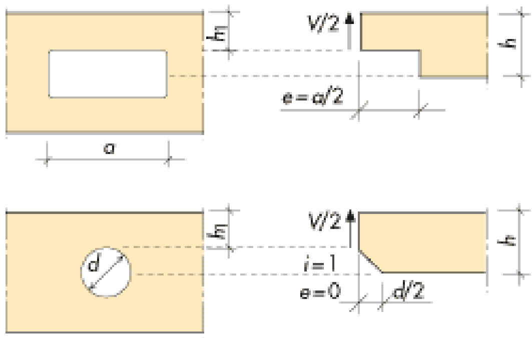


Figure 4.12 Analogy between holes and notches at the end of a beam. Symbols.

By using the above analogy the carrying capacity at a hole with dominating lateral loading can be calculated using formulae 4.14 and 4.15. Symbols as in figure 4.12.

5.5.2.4.5.2 Reinforced holes

4.5.2.1 Plywood reinforcement

Reinforcement with plywood can be carried out using the following method, taken from the German timber standard DIN 1052:

Beech or birch plywood with a minimum thickness *d*= 10 mm is glued to both sides of the beam. The size of the plywood sheets shall meet the following conditions (see also figure 4.12):

$$h_0 \leq 0,5h \qquad h_1 \geq \begin{cases} 0,4h_0 \\ 0,1h \end{cases}$$

Formula 4.23

$$a \leq h \qquad a_1 \geq \begin{cases} 0,25a \\ h_1 \end{cases}$$

Formula 4.24

The outer layer is oriented with the grain parallel with the length of the beam. During gluing, pressure can be applied with the aid of ring-shank nails or with screws. In either case the plywood shall be pre-bored. Penetration in the glulam shall be at least twice the thickness of the plywood. Fastenings shall be evenly distributed across the plywood sheet and correspond to 1 nail/screw per 6000 mm^2. Distance between fastenings should be equal both parallel to and perpendicular to the grain and conform with current regulations. Gluing shall be carried out by the glulam manufacturer in accordance with current regulations for structural gluing.

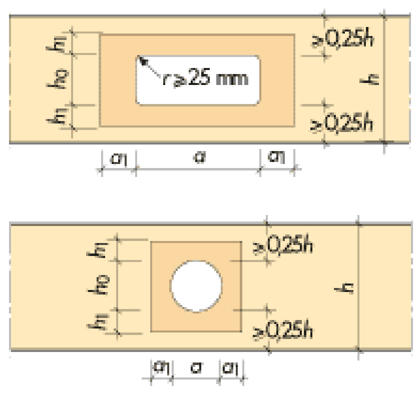


Figure 4.13. Plywood reinforcement of hole. Symbols.

The required thickness of plywood depends on the utilisation factor µand can be calculated using table 4.3.

$$\mu = \frac{1,5V/(bh)}{f_v}$$

Formula4.25

The utilisation factor depends on the overall cross-section of the beam

*b*x *h*without regard to the cross-sectional reduction caused by the hole

and with... *V*= the actual shear force at the section passing through the centre of the hole

f = the critical shear strength at the hole (without reduction for the hole)

Utilization factor μ	Plywood thickness on each side d
0	0,05b
1/3	0,18 b
2/3	0,25 b
1	0,33 b

Table 4.3 Required thickness of plywood reinforcement.

4.5.2.2 Reinforcement using glued-in screws

Reinforcement with glued-in screws is recommended only for structures in service class 1 and for hole sizes *a*0,6h and $h_0$0,35*h*. Since the screw restricts the free shrinkage of the beam and therefore gives rise to lateral tension in the critical area round the hole, use is limited to cases where the length of the screw does not exceed 500 mm. The screw or screws are placed as near the edge of the hole as possible with regard to the regulations on minimum edge and centre distance, and continue at least 40 mm beyond the hole (see figure 4.14). More than two rows of screws, counted along the length of the beam are not recommended.

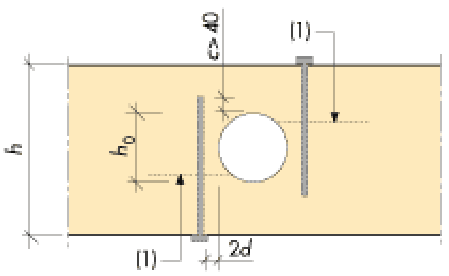


Figure 4.14 Reinforcement of hole using glued-in screw.

The screw/screws are designed for a force F_s:

$$F_s = 0,5V\left[3\left(\frac{h-h_0}{h}\right)^2 - 2\left(\frac{h-h_0}{h}\right)^3\right]$$

Formula 4.26

where

V= critical shear force at the centre of the hole

The carrying capacity of the screw F_R, with respect to tension strength and anchorage, is calculated using current regulations and approvals.

Design condition:

$$F_S \leq F_R$$

Formula 4.27

4.5.2.3 Glass fibre reinforcement

As with notches, reinforcement with glass fibre reinforced plastic is a usable method (see 4.4.2.3).

6.3.1.5.3.1 Continuously bolted columns

Continuously bolted columns as shown in figure 5.4a are assumed to be assembled using toothed connectors, Bulldog or similar. Design can be based on an effective slenderness ratio:

$$\lambda_e = \lambda \frac{2}{\sqrt{1+6\frac{\gamma}{1+\gamma}}}$$

Formula 5.9

where

λ= slenderness ratio for buckling in the plane of paper if there is full interaction between the shafts:

$$\gamma = \frac{1}{1+\frac{\pi^2}{2}\cdot\frac{E\cdot A\cdot s}{K\cdot l^2}}$$

where

γ= reduction factor depending on degree of interaction

E= modulus of elasticity in ultimate limit state (E_0)

K= slip modulus (N/mm) for bolt with bolts with tooth-plate connector

A= total cross-sectional area of the column

l = length of column

s= vertical distance between centres of bolts or groups of bolts

n= number of bolts per group

$E_0$and *K*are assigned their characterstic values according to the relevant code. Since both quantities are affected in almost the same way by variations in moisure content and by duration of load, the degree of interaction, expressed in the factor γ, is independent of them.

The codes often give only one value K_{ser}on the slip modulus, which is intended for calculation of deformations. In those cases the factor γmay be calculatet with a reduced value $K= {}^2/_3 K_{ser}$.

Individual bolts and connectors must be checked for shear forces.

$$F = \frac{1}{n}\cdot V\cdot\frac{s}{b}\cdot\frac{1}{1+\frac{1+\gamma}{6\gamma}}$$

Formula 5.10

where

$$V = \begin{cases} \frac{N}{120} \cdot \frac{1}{k_c} & \text{for} \quad \lambda \leq 30 \\ \frac{N}{60} \cdot \frac{1}{k_c} \cdot \frac{\lambda_e}{60} & 30 < \lambda_e < 60 \\ \frac{N}{60} \cdot \frac{1}{k_c} & \lambda_e \geq 60 \end{cases}$$

Formula 5.11

N= design load on the column

λ_e= effective slenderness ratio in accordance with formula (5.9)

k_c= reduction factor with regard to buckling risk

The buckling factor $k_c$is calculated in accordance with formula (5.5), using λ_{rel}calculated from the effective slenderness ratio of the column λ_e.

6.4.1.5.4.1 Horizontal forces due to columns out of plumb

The deviation from verticality of the rigidly fixed columns can be regarded as part of the initial curvature which is anticipated when designing with regard to buckling and need not be taken into account. The unintentional lack of verticality in the pinned columns, on the other hand, produces horizontal forces in the system which must be taken care of by the rigidly fixed columns. The critical horizontal force can be calculated using the relation

$$H = \alpha_d \Sigma N_i$$

Formula 5.12

where

ΣN_i= total design load on the pinned columns

$\alpha_d = 0{,}003 + 0{,}012 / \sqrt{n}$

n= number of interacting columns (including pinned)

The lateral force is distributed among the rigidly fixed columns in proportion to their stiffness. It is assumed to have the same duration as the critical vertical load and is combined with the other horizontal forces (wind etc) which must be taken up by the rigidly fixed columns.

6.4.2.5.4.2 Buckling

In comparison with a single rigidly fixed column, the buckling load of the interconnected columns in the above system is reduced since they must not only carry their own loads but also brace the pinned columns.

For a system with one or more similar, rigidly fixed columns which brace a number of pinned columns as in figure 5.5, the effect of the pinned columns on the buckling load of the system is taken into account by checking the carrying capacity of the rigidly fixed columns with regard to the slenderness ratio:

$$\lambda_e = \lambda \sqrt{1 + \frac{\pi^2}{12} \cdot \frac{\Sigma(N_i / l_i)}{\Sigma(N_0 / l_0)}}$$

Formula 5.13

where

λ= the slenderness ratio of the rigidly fixed columns in buckling in the plane of the system, without regard to the effect of the pinned columns

N_i, l_i= vertical load and actual height of the individual pinned column

N_o, l_o= vertical load and actual height of rigidly fixed column.

Example

Calculate the moment of fixture and the slenderness ratio of the columns in figure 5.6 with regard to buckling in the plane of the frame.

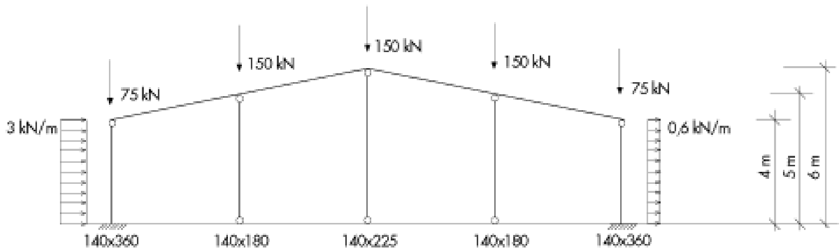


Figure 5.6. Conditions for the calculation example.

Horizontal forces due to the columns being out of plumb

In accordance with formula 5.12:

$$H = \left(0{,}003 + 0{,}012 / \sqrt{n}\right) \sum N_i =$$
$$= \left(0{,}003 + 0{,}012 / \sqrt{5}\right) \cdot 3 \cdot 150 = 3{,}8 \text{ kN}$$

The horizontal force causes moments in the rigidly fixed columns:

M= 0,5x3,8x4 = 7,6 kNm

Wind load moment

The tops of the columns are first assumed to be fixed.

The moment of fixture in the left-hand columns is then:

$$M_0 = \frac{q_1 h^2}{8} = \frac{3 \cdot 4^2}{8} = 6,0 \text{ kNm}$$

The horizontal support reaction at the top of the column:

$$R = \frac{3}{8} q_1 h + \frac{3}{8} q_2 h = \frac{3}{8}(3,0 + 0,6) \cdot 4 = 5,4 \text{ kN}$$

The tops of the columns are then freed and the support reaction distributed over the columns in relation to their stiffness, i.e. each column in the external wall is given 0,5 *R*= 2,7 kN.

The moment of fixture due to this horizontal force is:

$$M_R = 0,5 Rh = 0,5 \cdot 5,4 \cdot 4 = 10,8 \text{ kNm}$$

The total moment of fixture in the left-hand columns is:

$$M = M_0 + M_R = 6,0 + 10,8 = 16,8 \text{ kNm}$$

Buckling

Without taking into account the effect of the pinned columns, the slenderness ratio of the rigidly fixed columns is in accordance with formula 5.1:

$$\lambda = \beta \frac{h\sqrt{12}}{b} = 2,25 \frac{4000\sqrt{12}}{360} = 87$$

where

b= beam depth

h= column length

$$\lambda_e = \lambda\sqrt{1+\frac{\pi^2}{12}\cdot\frac{\sum(N_i/l_i)}{\sum(N_o/l_o)}} = 87\sqrt{1+\frac{\pi^2}{12}\cdot\frac{2\cdot 150/5+150/6}{2\cdot 75/4}} = 147$$

The slenderness ratio of the pinned columns is calculated in the normal way:

$$\lambda = \frac{5000\sqrt{12}}{180} = 96$$

$$\lambda = \frac{6000\sqrt{12}}{225} = 92$$

7.1.1.6.1.1 Internal forces and reactions

Internal forces and support reactions can most easily be calculated by computer. For manual calculations, tables 6.1 and 6.2 give beam coefficients for some common loading cases.

Using the coefficients *k* in the tables, the following apply:

Moment $M = k \times ql^2$

Reaction at support $R = k \times ql$

Deflection $w = k \times ql^4/100EI$ where $x = k \times l$

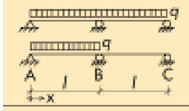
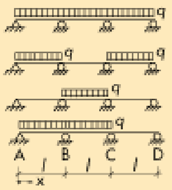
	Moment					Reaction at support				Deflection	
	M_{AB}	M_B	M_{BC}			R_A	R_B	R_C		w	x
	0,070	-0,125	0,070			0,375	1,250	0,375		0,540	0,42
	0,096	-0,063	-0,025			0,438	0,625	0,063		0,920	0,47
	M_{AB}	M_B	M_{BC}	M_C	M_{CD}	R_A	R_B	R_C	R_D	w	x
	0,080	-0,100	0,025	-0,100	0,080	0,400	0,100	1,100	0,400	0,690	0,45
	0,101	-0,050	-0,050	-0,050	0,101	-0,450	0,550	0,550	0,450	0,990	0,48
	-0,025	-0,050	0,075	-0,050	-0,025	-0,050	0,550	0,550	-0,050	0,677	1,50
	-0,072	-0,117	0,053	-0,033	-0,017	-0,383	1,200	0,450	-0,033	0,586	0,43

Table 6.1. Coefficients k for moment, reaction at support and deflection. Applies to unjointed beam of constant depth.

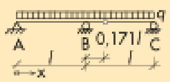
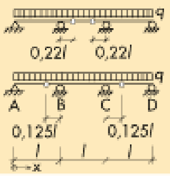
	Moment					Reaction at support				Deflection	
	M_{AB}	M_{Bc}	M_B			R_A	R_B	R_C		w	x
	0,086	0,086	-0,086			0,414	1,172	0,414		0,774	0,46
	M_{AB}	M_B	M_{BC}	M_C	M_{CD}	R_A	R_B	R_C	R_D	w	x
	0,086	-0,086	0,039	-0,086	0,086	0,414	1,085	1,085	0,414	0,774	0,46
	0,096	-0,063	0,063	-0,063	0,096	0,438	1,062	1,062	0,438	0,714	0,49
										0,521	1,50

Table 6.2 Coefficients *k* for moment, reaction at support and deflection. Applies to Gerber system with constant depth.

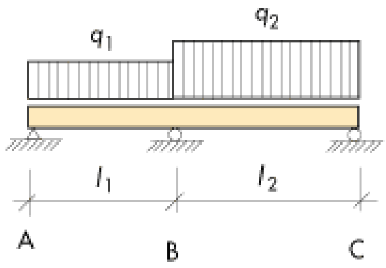


Figure 6.1. Continuous beam on three supports

For continuous beams on three supports with different spans and with uniformly distributed loading (see figure 6.3), moment and reactions at supports can be calculated using the following formulae:

$$M_B = -\frac{q_1 l_1^3 + q_2 l_2^3}{8(l_1 + l_2)}$$

Formula 6.1

$$R_A = \frac{q_1 l_1}{2} + \frac{M_B}{l_1} \text{ where } M_{AB} = \frac{R_A^2}{2q_1}$$

Formula 6.2

$$R_C = \frac{q_2 l_2}{2} + \frac{M_B}{l_2} \text{ where } M_{CD} = \frac{R_C^2}{2q_2}$$

Formula 6.3

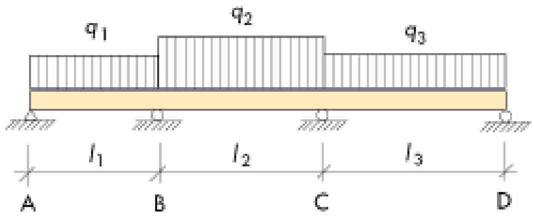


Figure 6.2. Continuous beam on four supports

For continuous beams on four supports with different spans and with uniformly distributed loading (see figure 6.2), moments at intermediate supports can be calculated using the following formulae:

$$M_B = -2q_1 \frac{l_1^3(l_2 + l_3)}{L^2} - q_2 \frac{l_2^3(l_2 + 2l_3)}{L^2} + q_3 \frac{l_3^3 \cdot l_2}{L^2}$$

Formula 6.4

$$M_C = q_1 \frac{l_1^3 l_2}{L^2} - q_2 \frac{l_2^3 \left(2l_1 + l_2\right)}{L^2} - q_3 \frac{l_3^3 \left(l_1 + l_2\right)}{L^2}$$

where

$$L^2 = 16(l_1 + l_2)(l_2 + l_3) - 4l_2^2$$

Formula 6.5

7.1.2.6.1.2 Shear

The following design condition apply:

$$\tau = \frac{1,5V}{bh} \le k_v f_v$$

Formula 6.6

where

V= design shear force

f_V= design shear strength

k_V= reduction factor with regard to notches in beam end (see 4.4).

For a beam supported on its underside and loaded along the top, the shear force(but not the reaction at support) may normally be calculated without regard to that part of the load which acts at a shorter distance than the beam depth from the theoretical support. This may be applied both to beams simply supported at two points and to continuous beams on three or more supports.

The condition can also be formulated as the required beam depth with regard to shear strength

$$h_v = \frac{1,5V}{bk_v f_v}$$

Formula 6.7

For beams simply supported at two points with a uniformly distributed load, the condition can be written as

$$h_v = l\,\frac{0,75q / b}{k_v f_v + 1,5q / b}$$

Formula 6.8

7.1.3.6.1.3 Bearing strength

The following design condition apply:

$$\sigma_{c90} = \frac{R}{bl_{eff}} \leq f_{c90}$$

Formula 6.9

where

l_{eff}= effective length of support in accordance with 4.3

R= design reaction at support

F_{c90}= design compression strength perpendicular to the grain according to current regulations.

7.1.4.6.1.4 Bending and buckling

Under loading in the stiff plane of the beam, the following design condition apply:

$$\sigma_m = \frac{6M}{bh^2} \leq k_{crit} f_m$$

Formula 6.10

where

M= design moment

f_m= design bending strength

k_{crit}= modification factor with regard to buckling (see 4.2)

With continuous beams the support moment may be calculated assuming that the support reactions are uniformly distributed over the support length.

The support moment (6.1.1) can then be reduced by:

$$\Delta M = Ra/8$$

Formula 6.11

where

R = design support reaction

a = length of support

The design condition can also be expressed as the required beam depth with regard to bending strength:

$$h_m = \sqrt{\frac{6M}{bk_{crit}f_m}}$$

Formel 6.12

For beams freely supported at two points with a uniformly distributed load, the following applies:

$$h_m = l\sqrt{\frac{0{,}75ql/b}{k_{crit}f_m}}$$

Formel 6.13

Since both $k_{crit}$and $f_m$normally vary with the beam depth h_m, they must usually be determined by trial and error.

If bending occurs simultaneously about the y and the z axes, conditions (5.7) and (5.8) apply. The formulae apply to sections of any form.

7.1.5.6.1.5 Deflection

The design condition can usually be expressed as:

$$w_S = k_1 \frac{Ml^2}{Ebh^3} + k_2 \frac{M}{Gbh} \leq w_R$$

Formula 6.14

where

$k_1$and k_2= factors whose value depends on support conditions and type of loading

M = design moment

E, G = design values for material properties

WS, WR = calculated and maximum permitted deflection, e.g. as in table 3.1

If the shear deformations are ignored, which is satisfactory for beams where l/h> 10, the design condition can be expressed as the beam depth required with regard to deflection:

$$h_w = \sqrt[3]{\frac{k_1 Ml^2}{Ebw_R}}$$

Formula 6.15

For beams simply supported at two points with a uniformly distributed load, the following applies:

$$h_w = l \cdot \sqrt[3]{\frac{5}{32} \cdot \frac{q/b}{E} \cdot \frac{l}{w_R}}$$

Formula 6.16

For continuous beams, deflection is, as a rule, not critical. Maximum deflection usually occurs in end bays. If shear deformation is ignored (see above) it can be calculated by integration of the differential equation of the elastic line:

$$w'' = -M/EI$$

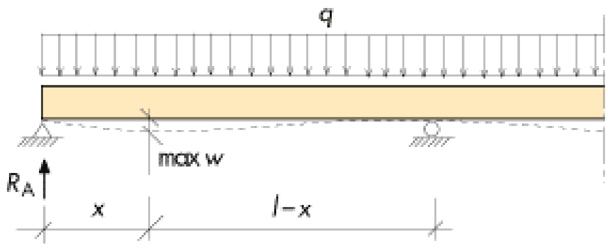


Figure 6.3. Deflection in end bay. Symbols.

Provided the cross-section is constant and the load uniformly distributed, the maximum deflection in the end bay is (symbols as in figure 6.3):

$$w = \frac{x}{EI}\left[\frac{R_A}{6}\left(l^2 - x^2\right) - \frac{q}{24}\left(l^3 - x^3\right)\right]$$

$$\text{där } x \approx l \cdot \sqrt{\frac{R_A - 0,25ql}{3R_A - 0,46ql}}$$

Formula 6.17

Note that design regarding stiffness usually applies to the serviceability limit state and that design load and material properties shall be determined on the basis of that state.

7.2.1.6.2.1 Internal forces and reactions

Internal moments and forces in statically indeterminate double tapered or single tapered beams cannot be determined by using table 6.1. which assumes constant beam depth. A satisfactory calculation must take into account the fact that variation in the depth affects the pattern of deformation and thus the distribution of internal forces. This type of calculation can advantageously be made using a computer.

7.2.2.6.2.2 Shear

As in beams of constant depth, the design condition applies:

$$\tau = \frac{1,5V}{bh} \leq k_v f_v$$

Formula 6.18

where

V= design shear force

f_V= design shear strength

k_V= reduction factor with regard to notches etc in beam ends (see 4.4).

The shear force can also in this case be calculated without regard to that part of the load which acts on the top of the beam at a shorter distance than the beam depth from the support. The beam is then assumed to be supported on its underside.

As with a beam of constant depth, the condition can be expressed as the required depth at the support with regard to shear strength

$$h_v = \frac{1,5V}{bk_v f_v}$$

Formula 6.19

For a beam with uniformly distributed load simply supported at two points, the condition can be written as:

$$h_v = l\,\frac{0,75q / b}{k_v f_v + 1,5q / b}$$

Formula 6.20

7.2.3.6.2.3 Bearing strength

As with beams of constant depth. The following design conditions apply:

$$\sigma_{c,90} = \frac{R}{bl_{eff}} \leq f_{c,90}$$

Formula 6.21

where

l_{eff}= effective length of support in accordance with 4.3

R= design reaction at support

$f_{c,90}$= design compression strength perpendicular to the grain according to current regulations

7.2.4.6.2.4 Bending

In principle, the same design rules apply as for beams with constant depth. However, the varying sectional depth does affect the distribution of bending stresses within the section (see figure 6.6). In addition, the bending strength is reduced in the tapered edge (see 4.6).

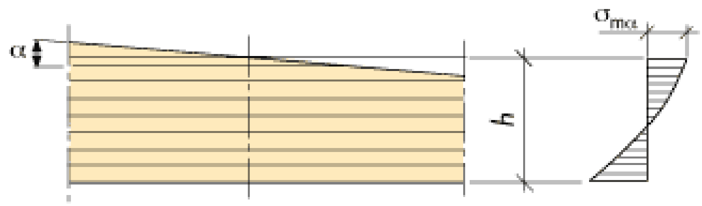


Figure 6.4. Distribution of bending stresses in beam of varying depth

For beams simply supported at two points, the combination of actions means that stresses in the (tapered) top side of the beam are usually critical in the design. When designing continuous beams it is necessary to check both the top and bottom edges of the beam at the support.

$$\sigma_m = k_{\sigma\alpha} \frac{6M}{bh^2} \le k_{f,\alpha} f_m$$

Formula 6.22

where

M= design moment in the cross-section

f_m= design bending strength

K_Kα/sub> = reduction factor due to tapering of laminates (see 4.6)

$$k_{\sigma\alpha} = \begin{cases} 1 - 4\tan^2\alpha & \text{at the top, tapered edge} \\ 1 + 4\tan^2\alpha & \text{at the lower edge (non - tapered)} \end{cases}$$

Since both the moment and the depth vary along the axis of the beam, maximum bending stress as a rule occurs not where the moment is greatest but at a section nearer the supports. The position of this section can be determined analytically by deriving the expression

$$\sigma(x) = \frac{M(x)}{W(x)}$$

and making the result equal to 0.

For simply supported single tapered beams or symmetrical double tapered beams with uniformly distributed loads, the critical section is at a distance of

$$x = \frac{h}{2h_m} l$$

Formula 6.23

from the support, where h is the depth of the beam at the support and h_m the beam depth at mid-span.

The maximum bending stress in the critical section is:

$$\sigma_m = k_{\sigma,\alpha} \frac{0,75ql^2}{bh(h + l \tan \alpha)}$$

Formula 6.24

The design condition can be expressed as the required beam depth at mid-span:

$$h_m = \frac{h_0^2 + h^2}{2h}$$

Formula 6.25

where

$$h_0 = \sqrt{\frac{0,75ql^2}{bf_m} \cdot \frac{k_{\sigma,\alpha}}{k_{f,\alpha}}}$$

Formula 6.26

At a specified roof slope, the following applies instead:

$$h_m = \sqrt{h_0^2 + \left(\frac{l}{2} \tan \alpha\right)^2}$$

Formula 6.27

When designing continuous double-tapered beams and double-tapered beams on two supports with a steep roof slope, the ridge should also be checked. The greatest bending stress occurs in the underside of the beam (see figure 6.5) and can be calculated using the following expression:

$$\sigma_m = \left(1 + 1,4 \tan \alpha + 5,4 \tan^2 \alpha\right) \cdot \frac{6M_{apex}}{bh_{apex}^2}$$

Formula 6.28

where

M_{apex} and h_{apex} are moment and beam depth respectively at the ridge. The design bending strength f_m need not be reduced because of the taper since the check in this case applies to the underside of the beam.

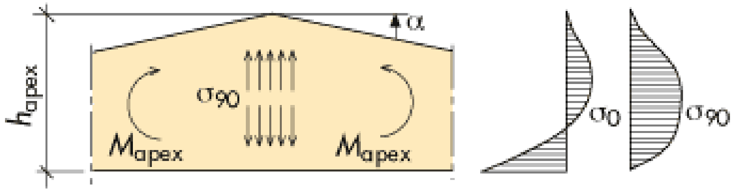


Figure 6.5 Distribution of bending stresses and tension stresses perpendicular to the grain at the ridge of a double tapered beam.

7.2.5.6.2.5 Lateral buckling

The carrying capacity with regard to lateral buckling is checked in accordance with 4.2. The following design conditions apply

$$\sigma_m = \frac{6M}{bh^2} \leq k_{crit} f_m$$

Formula 6.29

where

M= design moment

f_m= design bending strength

k_{crit}= reduction factor with regard to lateral buckling (see 4.2)

In double tapered and single tapered beams, loaded through purlins which restrain the top of the beam against lateral deflection, the reduction factor k_{crit}can be determined on the basis of a slenderness ratio based on the effective length of the beam $l_e$in accordance with table 4.2 and the beam depth $h$at the critical section.

7.2.6.6.2.6 Tension perpendicular to the grain

In double tapered beams whose load acts downwards, the tension perpendicular to the grain at the ridge can be critical. If the roof slope is under 10° and the top of the beam is laterally restrained, the design condition is:

$$\sigma_{90} = 0,1 \tan \alpha \frac{6 M_{apex}}{b h_{apex}^2} \leq k_{vol} f_{t,90}$$

Formula 6.30

where

$f_{t,90}$= design tension strengt perpendicular to the grain

k_{vol}= reduction factor with regard to the size effect on tension strengt perpendicular to the grain (see 4.8).

7.2.7.6.2.7 Deflection

Deflections in double-tapered and single tapered beams can most easily be calculated by computer.. Manual calculation with working equations involves a considerable amount of calculation.

Deflections in simply supported single tapered beams, or symmetrical double tapered beams, can be estimated using the following formula:

$$w_q = \frac{5}{384} \cdot \frac{ql^4}{EI_e} + 0{,}35 \frac{ql^2}{Gb\left(h + h_{\max}\right)}$$

Formula 6.31

where

$I_e = bh_e^3/12$

$h_e = h + 0{,}33 l/x \tan \alpha$ for double tapered beams

$h_e = h + 0{,}45 l/x \tan \alpha$ for single tapered beams

For beams where $2l/(h+h_{max}) > 15$ the second term in the above expression, which corresponds to the contribution of shear deformation to the deflection, can be ignored.

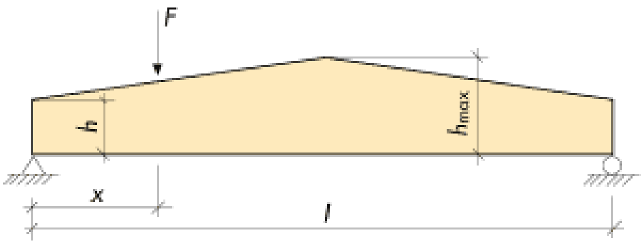


Figure 6.6 Simply supported, double tapered beam with concentrated load.

For a simply supported single tapered beam, or a symmetrical double tapered beam, loaded with a concentrated load F (see figure 6.6), the deflection at mid-span can be calculated using the formula:

$$w_F = k_F \frac{Fl^3}{48EI}$$

Formula 6.32

where

k_F = constant dependent on the ratio h/h_{max} and the point of action of the load which can be found in table 6.3.

$I = bh^3/12$

Double tapered beam

x/l	h/h_{max}				
	0,2	0,4	0,6	0,8	1,0
0,1	0,0104	0,045	0,105	0,188	0,296
0,2	0,0190	0,080	0,194	0,356	0,568
0,3	0,0206	0,104	0,259	0,489	0,792
0,4	0,0224	0,117	0,299	0,574	0,944
0,5	0,0229	0,121	0,312	0,604	1,0

Single tapered beam

x/l	h/h_{max}				
	0,2	0,4	0,6	0,8	1,0
0,1	0,0256	0,081	0,149	0,221	0,296
0,2	0,0430	0,147	0,278	0,421	0,568
0,3	0,0543	0,192	0,375	0,579	0,792
0,4	0,0548	0,213	0,430	0,663	0,944
0,5	0,0518	0,210	0,439	0,708	1,0
0,6	0,0446	0,187	0,399	0,657	0,944
0,7	0,0358	0,149	0,325	0,544	0,792
0,8	0,0238	0,103	0,228	0,385	0,568
0,9	0,0120	0,053	0,117	0,199	0,296

Table 6.3 Deflection coefficient k_F

7.3.1.6.3.1 Bending

The following design conditions apply within the curved part of the beam:

$$\sigma_m \le k_r f_m$$

Formula 6.33

where

k_r= reduction factor with regard to laminate curvature (see 4.7)

If the load is uniformly distributed, the cross-section at mid-span is critical. Maximum bending stress occurs at the bottom edge of the beam.

$$\sigma_m = k_l \frac{6M}{bh_{nock}^2}$$

Formula 6.34

The value of the coefficient $k_l$depends on the radius of curvature and the type of beam, i.e. constant or varying depth within the curved part of the beam. Different regulations give somewhat different design formulae, most of them based on the finite element method. EC 5 gives the following formulae for material with

E_0/E_{90} ≈30 and h_{apex}/r_m ≤0,3:

$$k_l = k_1 + k_2\left(\frac{h_{apex}}{r_m}\right) + k_3\left(\frac{h_{apex}}{r_m}\right)^2 + k_4\left(\frac{h_{apex}}{r_m}\right)^3$$

Formula 6.35

where

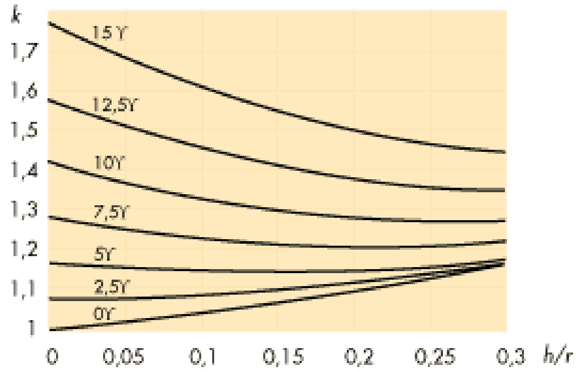
$$k_1 = 1 + 1,4 \tan\alpha + 5,4 \tan^2\alpha$$

$$k_2 = 0,35 - 8 \tan\alpha$$

$$k_3 = 0,6 + 8,3 \tan\alpha - 7,8 \tan^2\alpha$$

$$k_4 = 6 \tan^2\alpha$$

For beams of constant depth within the curved part, the formulae apply with α= 0 and h_{nock} = h. Figure 6.8 shows how $k_l$varies with roof slope and curvature.



Figur 6.8 Coefficient k_l for curved beam with varying depth and with different roof slopes and curvature.

For curved beams with varying depth, the bending stresses at the tangent points must also be checked. These can be calculated using the above expression, where:

$$k_l = \max \begin{cases} 1,05 \\ 1 + 3,7 \tan^2(\alpha - \beta) \end{cases}$$

Formula 6.36

7.3.2.6.3.2 Lateral buckling

The carrying capacity with regard to lateral buckling is checked as in 4.2. Compare also the corresponding instructions in 6.2.5 for straight beams of varying depth.

7.3.3.6.3.3 Tension perpendicular to the grain

Tension stresses perpendicular to the grain occur within the curved part of the beam under vertical, downward loading. The design condition is then:

$$\sigma_{90} \leq k_{vol} f_{t,90}$$

Formula 6.37

where

$f_{t,90}$= design tension strength perpendicular to the grain

k_{vol}= reduction factor with regard to the size dependence of the tension strength perpendicular to the grain (see 4.8)

Design tension stresses occur as a rule at mid-span and can be calculated using the expression:

$$\sigma_{m} = k_{p} \frac{6M}{bh_{apex}^{2}}$$

Formula 6.38

The value of the coefficient k_p depends, like the value of k_l above, on the radius of curvature and the type of beam, i.e. constant or varying depth.

EC 5 gives the following formula for material where:

E_0/E_{90} ≈30 and h_{apex}/r_m ≤0,3:

$$k_{p} = k_{5} + k_{6}\left(\frac{h_{apex}}{r_{m}}\right) + k_{7}\left(\frac{h_{apex}}{r_{m}}\right)^{2}$$

Formula 6.39

where

$$k_5 = 0,2 \tan \alpha$$

$$k_6 = 0,25 - 1,5 \tan \alpha + 2,6 \tan^2 \alpha$$

$$k_7 = 2,1 \tan \alpha - 4 \tan^2 \alpha$$

For beams with constant depth within the curved part, the formula applies with α= 0 and h_{apex}

= h. Figure 6.9 shows how k_p varies with roof slope and curvature.

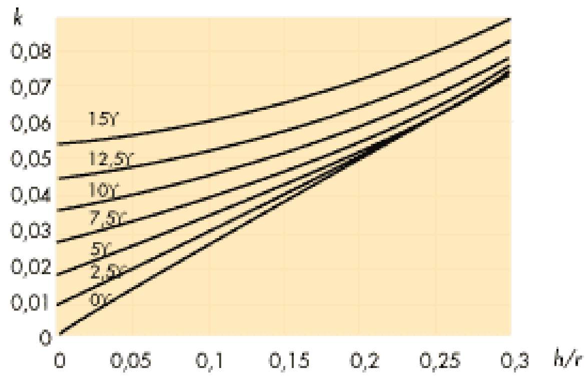


Figure 6.9 The coefficient $k_p$for curved beam with varying depth and with various roof slopes and curvature.

7.3.4.6.3.4 Deflection

Vertical deflection at mid-span can be estimated using the expression:

$$w = \frac{w_0}{\cos \frac{\alpha+\beta}{2}}$$

Formula 6.40

where

w_0= calculated deflection (see 6.2.7) for a symmetrical double tapered beam with the same span and the same cross-section at supports and at mid-span.

The horizontal movement at the free support can be estimated using the expression:

$$w_h = w \cdot \frac{f + 0,8h}{l/4}$$

Formula 6.41

where

f= vertical distance between neutral zone at support and ridge

h= beam depth at support

l= span

w= vertical deflection at mid-span

10.1.1.9.1.1 Internal forces and support reactions

The three-pin frame is statically determinate and the internal forces in a cross-section can therefore be calculated with equilibrium equations. However, the geometric form often involves a great deal of calculation work which can suitably be carried out with the aid of a computer.

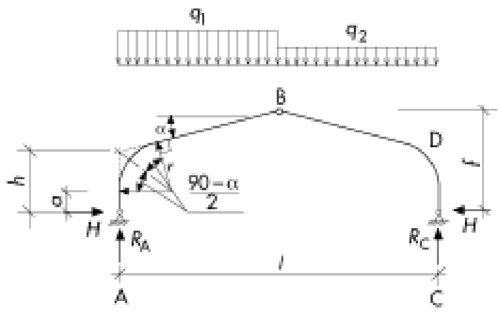


Figure 9.2 Three-pin frame with curved haunches. Symbols.

Under uniformly distributed, unsymmetrical loading the reaction and internal forces acting in a three-pin frame can be calculated using the following expressions (compare equations 8.1-8.3).

Vertical support reaction:

$$R_A = \frac{(3q_1 + q_2) \cdot l}{8}$$

Formulal 9.1

$$R_C = \frac{(q_1 + 3q_2) \cdot l}{8}$$

Formula 9.2

Horizontal support reaction:

$$H = \frac{(q_1 + q_2) \cdot l^2}{16f}$$

Formulal 9.3

Maximum moment at haunch:

$$M_D = H \cdot a + (N - R_C) \cdot r + \frac{q_2 r^2}{2}\left(1 - \frac{R_C}{N}\right)^2$$

Formulal 9.4

Adherent longitudinal force:

$$N \approx \sqrt{R_C^2 + H^2}$$

Formula 9.5

Shear force at ridge (vertical):

$$V_B = -\frac{(q_1 - q_2) \cdot l}{8}$$

Formulal 9.6

10.1.2.9.1.2 Preliminary design

A preliminary, rough design can be carried out as follows:

1. Determine the main sizes and critical values for the relevant loads and climatic conditions.

2. Sketch the approximate centre lines of the frame based on the following experience-based figures;

- Cross-sectional depth at haunch $h_r \approx h/15 + l/30$
- Cross-sectional depth at support $h_f \approx 0{,}7\ h_r$
- Cross-sectional depth at ridge $h_n \approx 0{,}3$ hr≈250 mm
- Radius of curvature at haunch $r_m \approx 170\ t$, where t is the thickness of laminate.

3. Calculate the support reactions and internal forces at the foot and ridge for various combinations of loading.

4. Determine the required cross-section at the foot with regard to the maximal longitudinal force or the maximal shear force.

5. Determine the required cross-section at the ridge using the same criteria as at the foot. The design of the ridge fixture can also affect the cross-sectional size.

6. Correct the sketch in point 2 and determine the bending moment and the internal forces at the haunch for various combinations of loading.

7. Determine the required cross-sectional sizes at the haunch with regard to stability and to the detail design according to the instructions given below.

8. Check the frame rafter for compression and simultaneous bending in accordance with 5.2. Normally the sectional depth has a linear variation from the ridge to the haunch, so critical cross-sections do not coincide with the section where the maximum field moment occurs. It is generally sufficient to check at a couple of randomly selected cross-sections.

10.1.3.9.1.3 Stability control

The risk of buckling in the plane of the frame or three-dimensional buckling is taken into account in the same way as in a laterally loaded column (see 5.2).

The design criterion is written as:

$$\frac{\sigma_c}{k_c f_c} + \frac{\sigma_m}{k_{crit} f_m} \le k_r$$

Formula 9.7

where

the reduction factor k_c is determined with regard to the risk of flat buckling in or perpendicular to the plane of the frame, depending on which is worst. k_{crit} is determined with regard to the risk of buckling as in 4.2, and k_r with regard to the curvature of laminates as in 4.7.

In buckling in the plane of the frame, the reduction factor k_c can be determined as for an axially loaded strut (see 5.1), where the slenderness ratio can be calculated with sufficient accuracy for normal conditions on the basis of a fictitious buckling length.

$$l_c = h\sqrt{4 + 1,6 \cdot c}$$

Formula 9.8

where

$$c = \frac{I}{I_0} \cdot \frac{2s}{h}$$

The moments of inertia I and I_0 are determined on the distances 0,65h and 0,65s from the foot and the ridge respectively (see figure 9.3). The expression, which is in the German timber code, can also be used for two-pin frames.

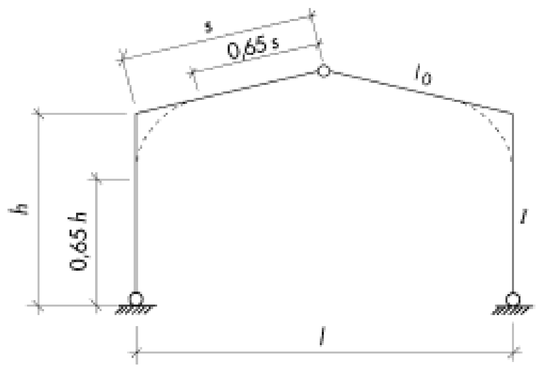


Figure 9.3. Fictitious buckling length for frames. Symbols.

10.1.4.9.1.4 Deformation

The calculation is simplest to carry out using a computer and one of the calculation programmes for timber structures that exist on the market.

10.2.1.9.2.1 Curved haunches

Curved haunches are usually made with a constant cross-section and the addition of the outer corner as in figure 9.1a. High requirements on appearance, e.g. in churches, can demand fully glued corners as in figure 9.4. The corner wedge (outside the broken line in the figure) can then either co-act with the cross-section or be simply nailed on. Radial tension stresses must be checked if load combinations give positive moments (inner edge in tension) at the haunch usually in combinations with wind load. Forces are calculated and checked in accordance with the instructions in 6.3. This means that a fully glued glulam haunch can be treated like a pitched cambered beam.

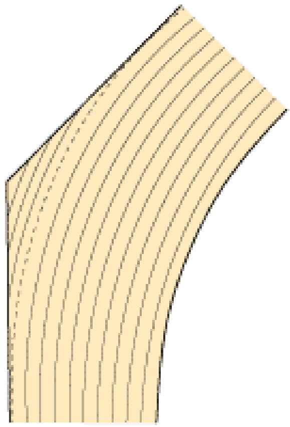


Figure 9.4. Curved haunch with fully glued corner piece.

10.2.2.9.2.2 Finger jointed haunches

Finger jointed haunches are usually designed with a jointing piece (see figure 9.5). The angle between the force and the grain at the joints is limited, which is favourable for the loading capacity of the haunch. The laminates shall be parallel with the underside of the frame.

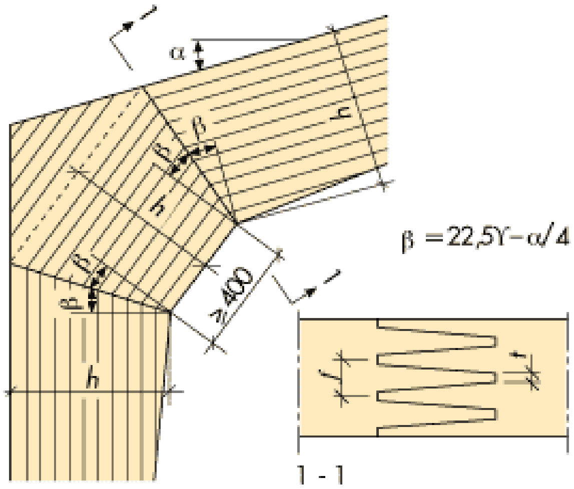


Figure 9.5 Finger jointed haunch with jointing piece.

The joints are checked according to the following empirical method:

• Moment M_{joint} and longitudinal force N_{joint} perpendicular to the joints is calculated at the centre of each joint

• Effective cross-sectional area and effective section modulus at the joints are calculated. With symbols as in the figure 9.5, the following expressions apply:

$$A_{joint} = \frac{bh}{\cos\beta}\left(1-\frac{t}{f}\right)$$

Formula 9.9

$$W_{joint} = \frac{bh^2/6}{\cos^2\beta}\cdot\left(1-\frac{t}{f}\right)$$

Formula 9.10

where

t= width of tip of finger

f= spacing of fingers centre to centre at base

• The design value of the compressive strength $f_{c\beta}$ is determined from the angle β between longitudinal forces and the grain (see 4.3) and based on the values applying to timber in the outer laminates without reduction for contact end grain/end grain. The design value shall not be reduced with regard to the risk of buckling.

• In loading combinations which produce compression at the inside edge of the haunch, the following critical conditions apply:

$$\frac{N_{joint}}{A_{joint}} + \frac{M_{joint}}{W_{joint}} \leq f_{c,\alpha}$$

Formula 9.11

• In loading combinations which produce tension at the inside edge of the haunch, usually in combinations with wind load, the following design conditions apply:

$$\frac{N_{joint}}{A_{joint}} + \frac{M_{joint}}{W_{joint}} \leq k_{\alpha} \cdot f_{c,\alpha}$$

Formula 9.12

where

the coefficient k_kα/sub> depends on the angle βbetween longitudinal force and the direction of grain in the jointing piece.

For β ≤11,25° k_kα/sub> = 0,333

11,25° β 18,75° k_kα/sub> = 0,533-0,0178β

18,75° ≤ β ≤22,5° k_kα/sub> = 0,200

10.2.3.9.2.3 Bolted haunches

Bolted haunches are normally designed with double leg members (see figure 9.6). With this design the risk of lateral buckling can be assumed to be eliminated. Alternatively, the haunch can be designed with single leg members and external joint plates of steel. In this case the risk of buckling shall be checked.

Frame legs and frame rafters can suitably be given the same depth, but each half of the leg should have a width of 0,6-0,7 times that of the rafter.

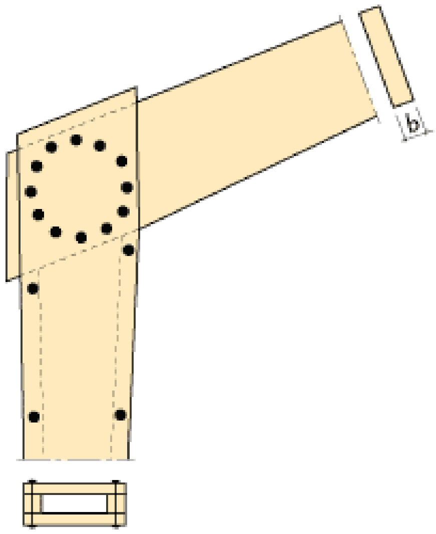


Figure 9.6 Bolted haunch

The bolts are placed with regard to current regulations on minimum distance between centres. To increase stiffness and carrying capacity at the haunch, the bolts can be combined with connectors.

Bolts without connectors can be replaced by steel dowels, which are normally cheaper and simpler to place. However, there should always be at least four bolts in a joint.

The carrying capacity is calculated on the assumption that all parts of the joint have the same stiffness. The forces at the joint are calculated and distributed using the following expressions (see figure 9.7).

$$F_{i,x} = \frac{H}{n} + \frac{M}{I} y_i$$

Formula 9.13

$$F_{i,y} = \frac{V}{n} + \frac{M}{I} x_i$$

Formula 9.14

where

$$I = \quad \Sigma(x_i^2 + y_i^2)$$

is the polar moment of inertia of the parts of he joint in relation to the centre of the joint.

The resultant force in each screw is then

$$F_i = \sqrt{F_{i,x}^2 + F_{i,y}^2}$$

Formula 9.15

in the direction α= arctan ($F_{i,y}$/$F_{i,x}$)

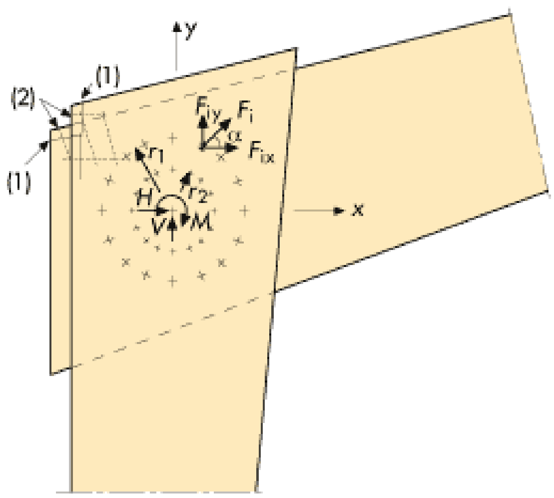


Figure 9.7 Bolted frame haunch. Symbols.

(1) Split

(2) Reinforcement using wood screw

The carrying capacity of individual bolts is determined taking into account the angle between the direction of force and the grain.

The bolts can with advantage be placed in concentric circles as in figure 9.6 and 9.7. More than two circles are however not recommended. The design carrying capacity of bolts in the inner circle should be reduced by 15%.

It has been shown in tests that shear stresses in the haunch are often critical to the load carrying capacity. Necessary checks can be limited to the following:

$$\tau = \frac{1,5V}{A} \leq f_v$$

Formula 9.16

where

$$V = \frac{M}{\pi} \cdot \frac{n_1 r_1 + n_2 r_2}{n_1 r_1^2 + n_2 r_2^2} - \frac{V_{rafter}}{2}$$

Formula 9.17 for frame rafter

$$V = \frac{M}{\pi} \cdot \frac{n_1 r_1 + n_2 r_2}{n_1 r_1^2 + n_2 r_2^2} - \frac{V_{leg}}{2}$$

Formula 9.18 for frame leg

A= cross-sectional area of the frame rafter or frame leg (NB double frame legs) at the section through the centre of gravity of the bolted joint.

n_1, n_2= number of bolts in outer and inner circle respectively

r_1, r_2= radius of outer and inner circle respectively.

$V_{rafter}$and $V_{leg}$are the shear forces which in the section through the centre of gravity of the bolted joint act on the frame rafter and the frame leg respectively (see figure 9.8).

The tests also show that a common reason for failure under positive moments, particularly in frames with the screws arranged in circles, are shear or flaking in the outer parts of the ends of the frame rafter and the frame leg (see figure 9.7). Reinforcement of the outer laminates in the haunch with nails or bolts (see figure 9.7) is therefore recommended. Reinforcement of the frame rafter and the frame leg is designed for a force of F_{haunch}as shown below:

$$F_{haunch} = \frac{M}{12} \cdot \frac{n_1 r_1}{n_1 r_1^2 + n_2 r_2^2}$$

Formula 9.19

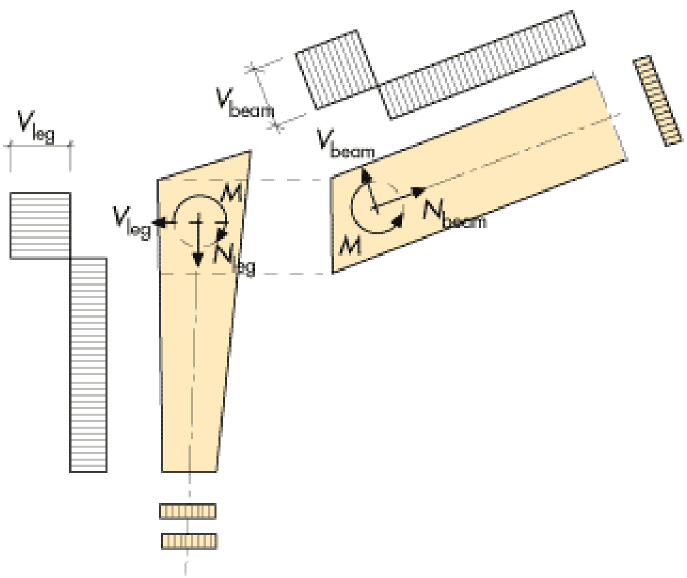


Figure 9.8 Cross-section forces and lateral force diagram for frame beam and frame leg.

10.2.4.9.2.4 Haunches in built-up frames

Built-up haunches can be designed in many different ways. In knee braced frames as in figure 9.9, section A is checked for compression and simultaneous bending as in 5.2, with reduced design values of compression and bending strength due to lateral buckling and buckling in the plane of the frame. Further, shear stresses at section B are checked.

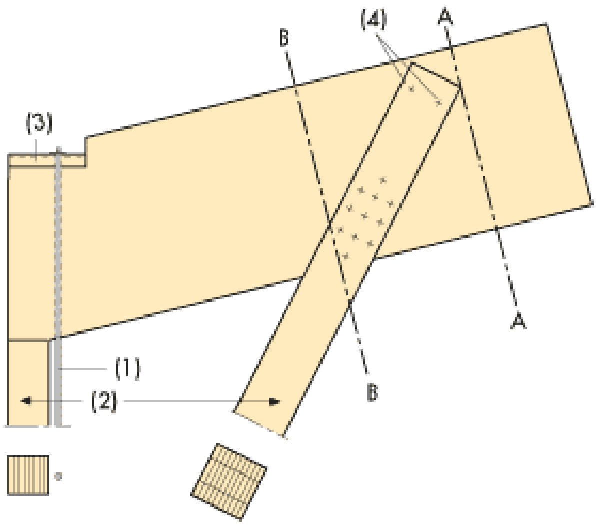


Figure 9.9 Built-up haunch with bolted connection between rafter and inner leg.

(1) Steel tension member

(2) Timber compression member

(3) Steel C section

(4) Elongated bolt holes

The outer frame leg is designed for axial loading, possibly with simultaneous wind moments. As a rule there will be large tension forces in the outer frame leg. These can be taken down to the foundations with the aid of a steel tension member as in figure 9.9, while moment and compression forces are taken up by a simple timber strut. The tension member should be fixed in the top of the frame rafter to minimize the risk of splitting.

The inner frame leg is designed as a column subject to compression and bending. The bending moment is normally caused by eccentric loading (see e.g. figure 9.10). The connection between the frame rafter and the frame legs is so designed that transfer of compression forces occurs mainly at the lower edge of the frame rafter, preferably through contact pressure. The inclined inner frame leg can be assembled by a frame joint as in figure 9.10. The distribution of forces in this well-tried form of connection is unclear but according to established practice the forces can be checked as follows:

- The compression force $N$is divided into its components N1/sub> and N_2, perpendicular to the areas of contact.
- $N_1$is assumed to be uniformly distributed across the area BC. The depth of the notch is such that:

$$\sigma_{c,\alpha,1} = \frac{N_1}{ba}\cos\left(\beta / 2\right) \leq f_{c,\alpha,1}$$

Formula 9.20

where

$f_{c,a,1}$is determined as in 4.3 with regard to angle

α= β/ 2 between the direction of the force and the grain. For a right-angled notch α= *h*/ 2.

•$N_2$is assumed to be uniformly distributed across the area AB, whose size is determined by the condition:

$$\sigma_{c,\alpha,2} = \frac{N_2}{bd} = f_{c,\alpha,2}$$

Formula 9.21

where

$f_{c,a,2}$is determined as in 4.3.2 with regard to angle α= 90° - β/2 between the direction of the force and the grain.

•Further, it must be checked that:

$$\tau = \frac{N_1 \cos\left(\beta / 2\right)}{bs} \leq f_v$$

Formula 9.22

where

*s*shall be at least 200 mm.

Length in excess of *s*= 8*a*can not be utilised.

• If the inclination β ≥60° the depth of the notch is limited to:

$$a \leq \frac{H}{6}$$

Formula 9.23

•The cross-sectional size of the strut is checked:

$$h \geq \left[a \cdot \tan(\beta / 2) + d \cdot \cos(\beta / 2)\right] \cdot \sin\beta$$

Formula 9.24

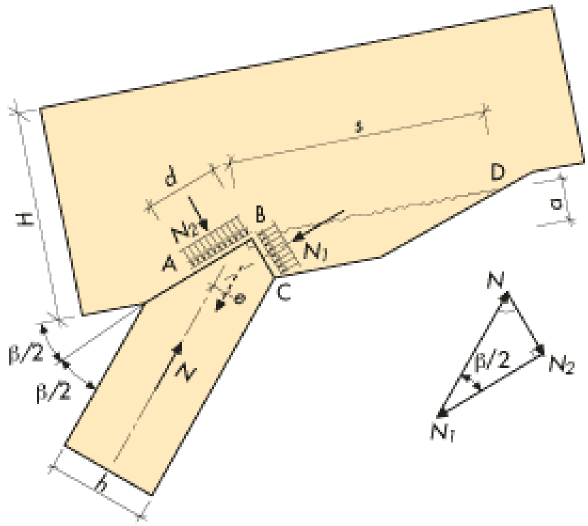


Figure 9.10 Framed joint. Force distribution and symbols.

11.1.1.10.1.1 Internal forces and support reactions

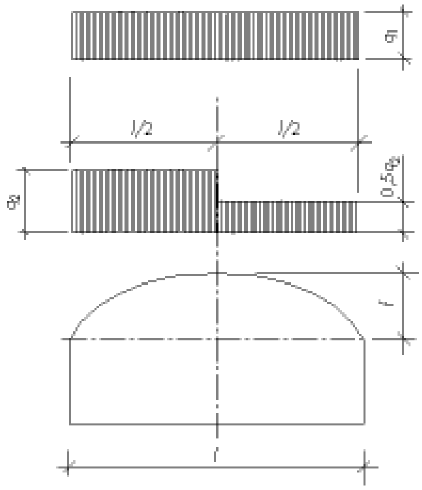


Figure 10.4 Example of various load distributions on an arched roof.

For uniformly distributed loads, support reactions and internal moments and forces in a 3-pin parabolic arch can be calculated from the following expressions:

Vertical reaction at support:

$$R = \frac{q_1 \cdot l}{2}$$

Formula 10.1

Horizontal reaction at support:

$$H = \frac{q_1 \cdot l^2}{8f}$$

Formula 10.2

Maximum longitudinal force:

$$N = \sqrt{R^2 + H^2}$$

Formula 10.3

Longitudinal force at random section:

$$N = \sqrt{\left(R - q_1 x\right)^2 + H^2}$$

Formula 10.4

Moment at random section:

$$M_{(x)} = 0$$

Formula 10.5

Shear force at random section:

$$V_{(x)} = 0$$

Formulal 10.6

For 3-pin circular arches with 0,14 *f*/ *l*0,20, the following approximation applies:

Maximum moment

$$M \approx \frac{q_1 \cdot f^2}{11}$$

Formula 10.7

Otherwise, the expressions for 3-pin parabolic arches can be used for rough estimates.

For non-uniformly distributed load as in figure 10.4 the reactions and cross-sectional forces in a 3-pin parabolic arch can be calculated from the following expressions:

Vertical reaction at support:

$$R_A = \frac{7}{16} \cdot q_2 l$$

Formula 10.8

$$R_B = \frac{5}{16} \cdot q_2 l$$

Formula 10.9

Horizontal reaction at support:

$$H = \frac{3q_2 \cdot l^2}{32f}$$

Formula 10.10

Maximum moment (for x = 0,25*l*):

$$M = \frac{q_2 \cdot l^2}{128}$$

Formula 10.11

Relevant longitudinal force:

$$N = H \cdot \sqrt{1 + \left(\frac{2f}{l}\right)^2}$$

Formula 10.12

Maximum shear force (at ridge):

$$V = \frac{1}{16} \cdot q_2 l$$

Formula 10.13

For 3-pin circular arches with 0,14 *f*/ *l*0,20, the above expressions can be used for rough estimates.

Reactions- and internal forces for arches can suitably be calculated with the aid of a computer.

11.1.2.10.1.2 Stability control

Arches are as a rule slender structures and the design must therefore, to an even larger extent than with frames, take into account the extra forces which the initial deformation of the arch and its deformation under load cause. According to the 2nd Order Theory these are calculated and added to the internal moments and forces already calculated, without reduction of the design strength due to buckling. Initial deformation should be assumed to be affine to the deformation figure under load and can be assumed to be in the form of a sinus curve with a rise of 0,003 x *l*(see figure 10.5). The calculation work is usually considerable and in practice demands the use of a computer.

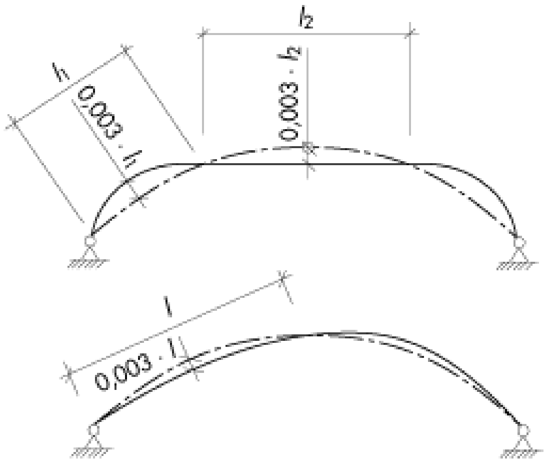


Figure 10.5 Example of an assumed initial deformation under symmetrical and unsymmetrical loadings. l, l1 and l2 are the distances between points which have not moved during the deformation of the arch.

Alternatively the arch can be regarded as an unchanging static system (1st Order Theory) and instead take into account the effect of the additional forces by reducing the design strength values with a slenderness-linked factor determined in the same way as in a strut subject to compression and bending as in 5.2.

The design criterion is the same as for frames, equation (9.7):

$$\frac{\sigma_c}{k_c f_c} + \frac{\sigma_m}{k_{crit} f_m} \leq k_r$$

Formula 10.14

where

the reduction factor $k_c$is determined with regard to the risk of plane buckling in or perpendicular to the plane of the frame, depending on which is worst. $k_{crit}$is determined with regard to the risk of lateral buckling (see 4.2) and $k_r$with regard to curvature of laminates (see 4.7). Where radii of curvature are small it should be noted that the distribution of the bending stresses over the depth is not linear, by taking:

$$\sigma_m = k_p \frac{6M}{bh^2}$$

Formula 10.15

when applying 6.3.1. The coefficient k_p is determined as in formula 6.39 with the angle $\alpha = 0$.

Buckling in the plane of the arch

The reduction factor k_c is determined as for a member in compression (see 5.1). The slenderness ratio is then determined with a fictitious buckling length l_c as starting point. For 3-pin parabolic arches with constant section the following expressions are approximately correct:

$$l_c = \begin{cases} 0{,}58 \cdot l \cdot \sqrt{1+6\left(\frac{f}{l}\right)^2} & \text{om } \frac{f}{l} \le 0{,}3 \\ 0{,}50 \cdot l \cdot \sqrt{1+11{,}6\left(\frac{f}{l}\right)^2} & \text{om } \frac{f}{l} > 0{,}3 \end{cases}$$

Formula 10.16

where

the meaning of f and l can be seen in figure 10.2.

For 2-pin parabolic arches, regardless of the rise ratio f/l, the lower expression applies.

The compressive stress σ_c is calculated from the longitudinal force at the crown of the arch.

The above expressions can also be applied to two- or three-pin circular arches with low rise ratio ($f/l = 0{,}2$). For arches with a higher rise ratio the expressions give results on the dangerous side.

Buckling perpendicular to the plane of the arch

Buckling perpendicular to the plane of the arch is as a rule resisted by the roof construction or by secondary beams, e.g. purlins. The reduction factor k_c is determined as for a strut, with the buckling length taken as being equal to the length of the arc between the points where lateral deflection is prevented.

Lateral buckling

The reduction factor k_{crit} for lateral buckling is determined as in 4.2, with the radius of curvature taken as:

$$r = \begin{cases} \dfrac{l^2}{8f} & \text{för parabelbågar} \\ \dfrac{l^2}{8f} + \dfrac{f}{2} & \text{för cirkelbågar} \end{cases}$$

Formulal 10.17

In areas with large negative moments it can be necessary to brace the inner edge which is under compression.

11.1.3.10.1.3 Tension forces perpendicular to the grain

Radial (tension) forces must be checked in loading conditions which produce positive moments (inner edge in tension) in the arch   normally combinations of wind load or unevenly distributed snow load. The forces are calculated and checked as given in the instructions in 6.3.

Special care should be taken where the lateral tension stresses resulting from the arch form are superimposed by local stresses, e.g. those caused by hanging loads.

11.1.4.10.1.4 Arch length

The arch length *s* of a symmetrical parabolic arch with span *l* and rise *f* can be calculated using the following formula:

$$s = 2f \cdot \sqrt{1+\left(\frac{l}{4f}\right)^2} + 2f\left(\frac{l}{4f}\right)^2 \ln\left[\frac{4f}{l}\left(1+\sqrt{1+\left(\frac{l}{4f}\right)^2}\right)\right]$$

Formula 10.18

for *f*/*l*= 0,14, *s*= 1,0528 x *l*

The arc length *s* of a circular arch with span *l* and rise *f* can be calculated using the following formula:

$$s = 2f \cdot \left(1+\left(\frac{l}{2f}\right)^2\right) \frac{\pi}{360} \arcsin\left(\frac{\frac{l}{f}}{1+\left(\frac{l}{2f}\right)^2}\right)$$

Formula 10.19

for *f*/*l*= 0,14, *s*= 1,0544 x *l*

13.2.1.12.2.1 Wind on the sides of the building

Frames and arches are stable in their own plane, as are column and beam systems with columns rigidly fixed in the foundations. Systems with hinged columns must on the other hand be stabilised by diaphragm action in the roof or by wind trusses in the plane of the roof. The principle of wind bracing with trusses along the side of the building is shown in figure 12.1. The roof beams are often used as posts in the truss, with some of the purlins as bottom and top chords. The latter can in this case not be designed as Gerber beams. Only diagonals are added as stiffeners, often crosses of steel rods or timber diagonals stiff in compression. The joints between the various components forming part of the wind truss must be designed with regard to the forces which arise and to eccentricity. Truss posts and chords are often in different planes.

13.2.2.12.2.2 Wind on the end walls

Bracing against wind on the end walls can suitably be achieved using wind trusses as in figure 12.3. Primary beams are here used as top and bottom chords. Purlins can act as posts but it is often preferred to design the truss with separate posts to get all the members in the same plane and to avoid difficulties with eccentricity at the nodes. The wind truss is often designed so that it can be used for stabilisation during the erection of the structure.

In small buildings a truss in one end wall is normally sufficient. The purlins or the roof diaphragm must then be able to transfer both compression and tension forces from the other end. In long buildings it can on the other hand be suitable, not least with regard to stability during erection, to arrange wind trusses in one or more bays within the building.

At the ends the wind truss can with advantage be placed in the second bay from the end. The design of nodes is then not affected by the end wall structure, which usually differs from that in the rest of the building.

In designing it should be observed that the wind truss is seldom plane. At nodes where the members of the truss change direction, e.g. at the ridge of a double-pitched roof, force components across the roof arise which must be taken care of by the primary structure.

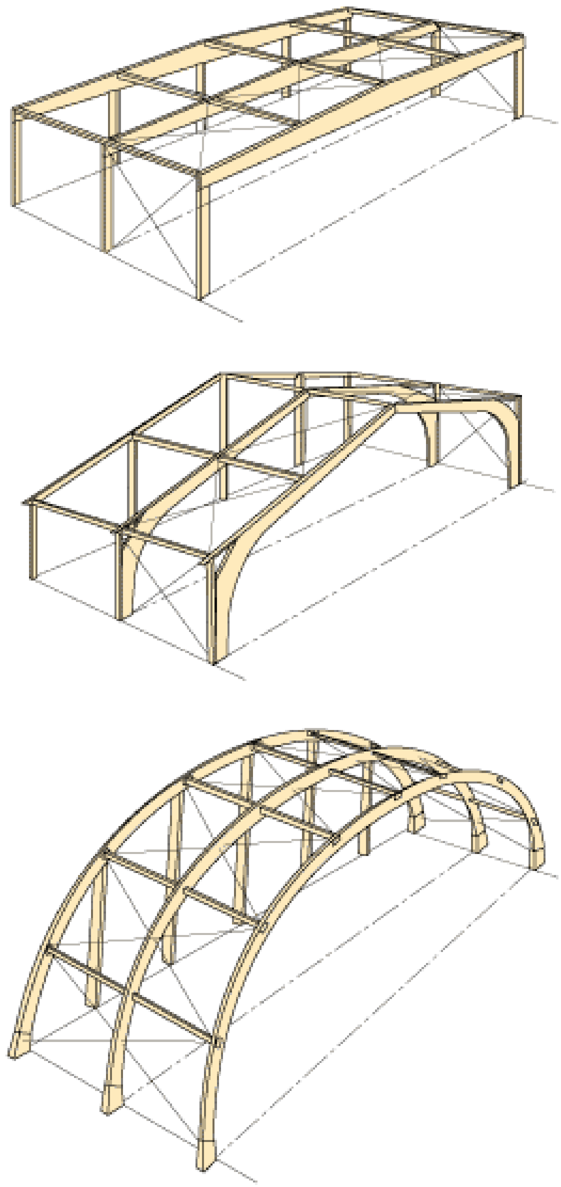


Figure 12.3 Wind bracing of frame and arch structures.

14.2.1.13.2.1 Fishplates of nailplate or flat steel

Pinned fixing with fishplates of nailplate or flat steel fixed to the sides of the column section is simple and functional, and suitable for small and large horizontal forces (see figure 13.1).

The fixture can either be cast into the concrete or welded to a cast-in fixing plate. The transfer of forces between concrete and cast-in fixtures can either take place though adhesion between fishplate and concrete or through contact with a bent end of the fishplate, alternatively against a screw or reinforcement steel passing through a hole in the fishplate. In designing it cannot be assumed that adhesion and contact co-act; the whole force must be taken either by adhesion or by contact.

The transfer of forces between fishplate and glulam column can be made with nails, screws or timber screws. Choice of fixing is made from aesthetic, economic, erection and strength or stiffness requirements. Carrying capacity and stiffness in screw joints can be improved by using connectors, e.g. toothed plates (Bulldog). These should be fitted in the factory as the pressing-in demands special tools. Connectors should therefore be avoided in combination with cast-in fixtures. It is not recommended to press in connectors using through screws, since the thread and any galvanizing can be damaged.

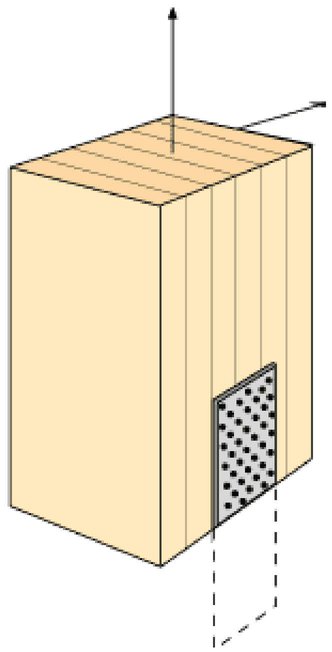


Figure 13.1 Pinned fixing of column foot with fishplates of nailplate. Principle sketch.

Pre-drilled nailplates of galvanized steel are not suitable for welding, as poisonous gases are formed by the heating-up of the zinc coating. They should only be used when the fishplates are fixed by being cast into the foundation.

Nailplates with different hole patterns and thicknesses can be ordered from special manufacturers of perforated sheet steel. The cost is generally lowest when the holes are punched out, which means that the thickness must not exceed the hole diameter. The holes should be about 1 mm larger than the thickness of the nail.

Fishplates for screwed or bolted joints can suitably be made of steel plates. Thicknesses are chosen from the standard range of 6, 8, 10, 12, 15 and 20 mm and shall be at least 0,3 times the bolt diameter (0,4 times for connectors). The holes are drilled 1 mm larger than the bolt diameter.

DESIGN

In the design the fishplate is regarded as a cantilever rigidly fixed in the foundation, loaded with a horizontal force and, where appropriate with a vertical force (uplift), both acting at the centre of the screw, bolt or nail group.

Downward vertical forces in the column are in general assumed to be transferred directly to the foundation by contact pressure. If contact pressure cannot be achieved, the vertical force should also be taken into account in the design of the fishplate.

The design condition for fixing a single fishplate in the column is:

$$\sqrt{F_x^2 + F_y^2} \leq R_{vd}$$

Formula 13.1

where

$F_x = 0,5V_d/n$

$F_y = 0,5H_d/n$

n= number of fixings per fishplate

R_{vd} = design carrying capacity for a nail or screw/bolt in shear

H_d = design horizontal force per fishplate

V_d = design vertical force (uplift) per fishplate

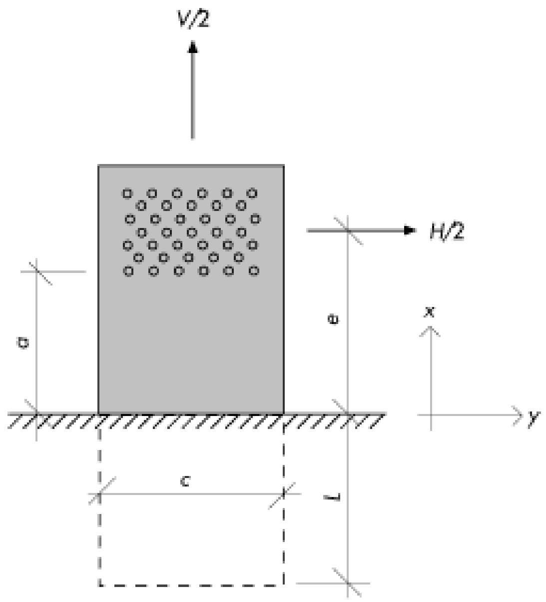


Figure 13.2. Nailplate as a rigidly fixed cantilever.

The design condition for an individual fishplate is:

$$\sqrt{\sigma^2 + 3\tau^2} \leq f_{yd}$$

Formulal 13.2

where

f_{yd} = design value of the tensile yield tension limit

σ = longitudinal stress and

τ = shear stress

calculated at the same point on the fishplate. If $\sigma$and $\tau$are calculated with the Elastic Theory as the starting point at the same time as a biaxial force condition applies, f_{yd}in formula 13.2 can be replaced by 1,1f_{yd}

The maximum longitudinal stress σ_{max}exists in the tension edge of the fishplate and can be calculated from the formula:

$$\sigma_{max} = \frac{0,5H_d \cdot e}{W} + \frac{0,5V_d}{A}$$

Formula 13.3

where

$A$and $W$are the cross-sectional area and the section modulus (respectively) of the fishplate. The distance between the cross-section where it is fixed and the centre of the nail or screw/bolt group is e. The shear stress τat the same cross-sectional edge is 0. The effect of the screw/bolt and nail holes on the capacity of the fishplate can be taken into account in formula 13.2 by replacing $f_{yd}$with $f_{ud}$and calculating the forces $\sigma$and $\tau$from the net values of the cross-section $A_{net}$and W_{net}.

This means that the material is allowed to yield round the holes.

The maximum shear stress σ_{max}acts on the level of the neutral axis in bending about the z-axis and can be calculated from the formula:

$$\tau_{max} = 1,5\frac{0,5H_d}{A}$$

Formula 13.4

If the vertical force is transferred by contact pressure, the longitudinal stress σ= 0. In other cases the longitudinal stress $\sigma$is calculated from the formula:

$$\sigma = \frac{0,5V_d}{A}$$

Formula 13.5

Thin fishplates are also checked for the risk of buckling at the compression edge of the plate. This check can be carried out according to Eurocode 3, Design of Steel Structures (EN 1993). The reduction factor, ω_b, is determined from the slenderness parameter $\lambda_b$of the fishplate, which is calculated from the formula:

$$\lambda_{b} = 0,76 \frac{\sqrt{c \cdot e}}{t} \sqrt{\frac{f_{yk}}{E_{k}}}$$

Formula 13.6

where

f_{yk} = characteristic value of the tensile yield tension limit of the steel

E_k = characteristic value of the Module of Elasticity

t = thickness of fishplate
= width of fishplate.
c

Other symbols, see figure 13.2.

For steel with f_{yk}= 270 N/mm2, the risk of buckling need not be checked if the thickness of the fishplate fulfils the condition.

$$t \geq 0,07\sqrt{c \cdot e}$$

Formula 13.7

For nailplates the distance *e*in the formulae 13.6 and 13.7 can be replaced by the length of the free edge *a*from the cross-section where the fishplate is fixed to the first row of nails (see figure 13.2).

The design condition covering anchorage through bonding of the individual fishplate cast into the foundation is:

$$\left(\frac{V}{V_d}\right)^2 + \left(\frac{H}{H_d}\right)^2 \leq 1,0 \qquad (13.8)$$

Formula 13.8

where

V_d= $2sLf_{cv}$

$$H_d = \frac{4c \cdot L \cdot f_{cv}}{1 + 1,2 \cdot L/c + 2,4 \cdot e/c}$$

$s = 2(c+ t)$

t = thickness of plate

f_{cv}= design value of bond strength

Other symbols as in figure 13.2.

For concrete of strength class K25, the characteristic value of adhesion can be taken as:

f

cvk

= 1,2 N/mm^2 for flat steel and 3,5 N/mm^2 for punched steel plate.

The surrounding concrete structure is designed according to Eurocode 2, Design of Concrete Structures (EN 1992).

If the connection to the foundation consists of welding to a cast-in fixing plate, the conditions for good workmanship should be considered. It is usually impossible to weld from both sides of the fishplate. If the fishplate is chamfered and butt welded throughout, its carrying capacity does not need to be checked. If however the fishplate is fillet welded to the fixing plate the strength of the weld must be checked according to Eurocode 3. The size of the fillet weld shall be at least 3 mm. If it is not over 5 mm, it can be laid in one operation.

If the welded joint between the fishplate and the fixing plate is utilised to more than 70%, some regulations demand that the weld be checked by non-destructive testing, e.g. X-Ray. Due to the risk of lamellar tearing of the fixing plate it is often demanded that, if utilised to over 50%, it must have verified properties in the direction of thickness, or be checked ultrasonically. It is therefore often practical to make the fixing plate of (ultrasonically) controlled material and to limit the utilisation in the fishplate in formula 13.2 to 70% while at the same time designing the weld so that it is as strong as the fishplate.

The width and length of the cast-in fixing plate are as a rule determined by the size of the glulam column and the amount of space needed for the fixture and its fixing, with the necessary permitted deviations. The thickness is determined so that the bending stress in the fixing plate does not exceed the design strength value.

14.2.2.13.2.2 Steel angle fixture

A pinned connection with fixtures of steel angle section screwed to the sides of the column is a simple and appropriate method suitable for small horizontal forces (see figure 13.3). The fixtures are bolted into the foundation with expansion bolts or adhesive anchors, which permits exact measurement and reduces the risk of incorrect placing. Symmetrical placing is recommended, with an angle on either side of the column.

The transfer of forces between the angles and the glulam column is by bolts or wood screws, if necessary combined with connectors. Connectors should be factory-fitted.

Various types of angle fixture, intended to be fixed with expansion bolts to the concrete construction below, are manufactured and sold by a number of firms. The fixtures can also be specially made of folded steel sheet or of rolled steel sections.

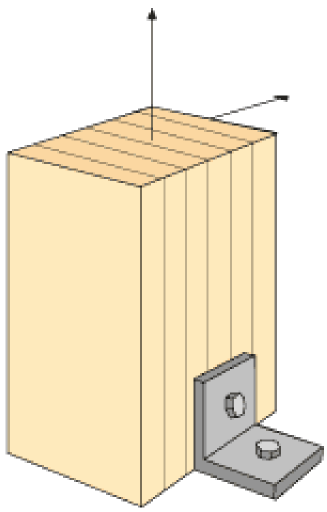


Figure 13.3 Pinned fixing of column base with steel angle fixture. Principles.

DESIGN

In the design the vertical part of the angle is regarded as a cantilever rigidly fixed in the foundation, loaded with a horizontal force and in certain cases with a vertical force (uplift), both acting in the centre of the group of bolts (see figure 13.4). Downward vertical forces in the column are transferred direct to the foundation by contact pressure.

Fixing in the column and the vertical flange of the angle are checked in the same way as for fishplates of nailplate of flat steel (13.2.1).

The design condition for the horizontal flange of the angle is:

$$\sqrt{\sigma^2 + 3\tau^2} \leq f_{yd}$$

Formula 13.9

where

f_{yd} = design value of the yield tension limit of steel

σ = longitudinal stress

τ = shear stress

calculated at the same point on the flange.

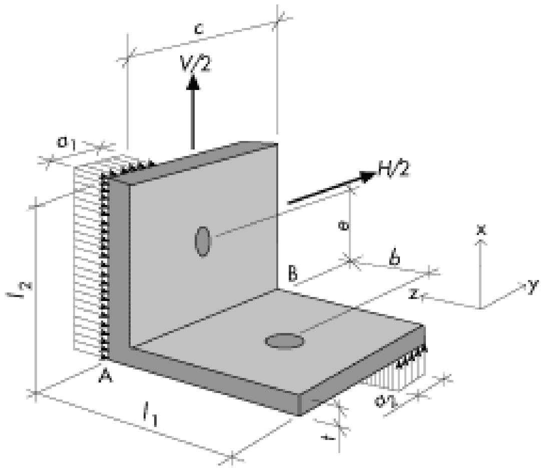


Figure 13.4 Fixture of steel angle. Symbols.

If $\sigma$and $\tau$are calculated with the Elastic Theory as the starting point at the same time as a biaxial stress condition applies, f_{yd}in formula 13.9 can be replaced by 1,1f_{yd}.

The maximum longitudinal stress σ_{max}occurs at points A and B and can be calculated using:

$$\sigma_{max} = \frac{0,5H_d \cdot b}{W_x} + \frac{0,5V_d \cdot b}{W_y}$$

Formula 13.10

where

$H_d$and $V_d$are the total design horizontal reaction and the upward vertical reaction from the column

$W_x$and $W_y$are the section moduli of the flanges

*D*is given in figure 13.4.

The shear stress τ= 0 at points A and B.

If the upward vertical force is small, the maximum shear stress τ_{max}occurs halfway between points A and B and can be calculated from the formula:

$$\tau_{max} = 1,5\frac{0,5H_d}{A} + \frac{0,5H_d \cdot e}{W_v}$$

Formula 13.11

where

A = cross-sectional area

W_v = torsion constant of the flange (=Ax t/3)

*e*is given in figure 13.4.

The longitudinal stress σat the same point on the cross-section can be calculated using the formula:

$$\sigma_{max} = \frac{0,5V_d \cdot b}{W_y}$$

Formula 13.12

If only one expansion bolt per fixture is used the moment of eccentricity $0,5H_d \times b$ is taken up by the column. The design condition regarding the contact pressure between the column and the fixture is then:

$$0,5H_d \cdot b \le f_{c90d} \cdot a_1 \cdot l_2 \left(\frac{c - a_1}{2} \right)$$

Formula 13.13

where

f_{c90d}= design value of the compression strength of the glulam column perpendicular to the grain.

a_1 = the width of the vertical compressive block diagram

Other symbols are given in figure 13.4.

The width of the compressive block diagram $a_1$can be calculated from the formula:

$$\frac{c}{2} \ge a_1 = t \cdot \sqrt{\frac{f_{yd}}{2 f_{c90d}}}$$

Formula 13.14

where

f_{yd}= design value of the yield tension limit of steel

t = thickness of the fixture.

The design condition for the fixing of the horizontal flange of the angle in the foundation is:

$$\left[\frac{F_v}{R_{vd}} \right]^2 + \left[\frac{F_t}{R_{td}} \right]^2 \le 1$$

Formula 13.15

where

$F_v = 0,5H_d/n$and

$F_1 = (0,5V_d + F)/ _n$.

R_{vd} = design strength value in shear for the expansion bolt

R_{td} = design value of the withdrawal strength of the expansion bolt

n = number of expander bolts per fixture

F = withdrawal force caused by eccentricity of horizontal force H_d x e.

With symbols as in figure 13.4, F can be calculated using the formula:

$$F = \frac{0{,}5 H_d \cdot e}{(c - a_2)/2}$$

Formula 13.16

The width of the horizontal compressive block diagram a_2 can be calculated by the formula

$$\frac{c}{2} \geq a_2 = t \cdot \sqrt{\frac{f_{yd}}{2 f_{cc}}}$$

Formula 13.17

where

f_{cc} is the design value of the strength of the concrete in local compression.

The design condition for contact pressure between concrete and the angle fixture is:

$$0{,}5 H_d \cdot e \leq f_{cc} \cdot a_2 \cdot l_1 \cdot \left(\frac{c - a_2}{2}\right)$$

Formula 13.18

For concrete of strength class K25 the characteristic value of compressive strength can be taken as f_{cck} = 30 N/mm^2, provided the distance to the edge or other factors do not reduce the strength.

The surrounding concrete is designed according to Eurocode 2 (EN 1992).

14.2.3.13.2.3 Base shoes

For pinned fixtures out of doors, or on floors where water is regularly present, base shoes are a suitable alternative since they prevent absorption of water through the butt end of the column (see figure 13.5). The fixture should be designed with drainage holes to eliminate the risk of standing water in the fixture itself.

The fixture consists of a short piece of U-section and a fixing rod. As a rule, part of the rod is cast into the slab, direct or in a pocket, but it can also be welded to a fixing plate cast into the slab. If the forces are small, a solid rod, e.g. a reinforcement rod, can be used, but round or square hollow sections are common.

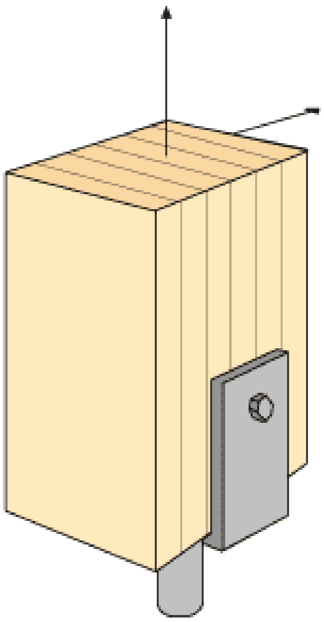


Figure 13.5 Pinned fixing of column foot using steel base shoe. Principle sketch.

Base shoes are available as standard products from several fixture manufacturers.

The transfer of forces between concrete and the cast-in rod or hollow section is either by adhesion between the metal and the concrete or by contact pressure with a washer and nut at the cast-in end of the rod. In designing it cannot be assumed that adhesion and contact pressure will co-act; the whole force must be taken up either by adhesion or by contact pressure.

The transfer of forces between the U-section and the glulam column is by nails or screws/bolts. The latter can if necessary be combined with connectors, which should be fitted in the factory (see 13.2.1 above).

DESIGN

In designing, the vertical part of the base shoe is regarded as a rigidly fixed cantilever, loaded with a horizontal force and in certain cases with a vertical force (uplift), both acting at the centre of the group of nails or screws/bolts. Downward vertical forces are transferred from the column to the base shoe by contact pressure and further through the rod/section to the foundation. The rod/section is designed with regard to buckling and to bending moments caused by horizontal forces.

The connection between the column and the sides of the U-section is checked in the same way as with fishplates of nailplate or flat steel in 13.2.1.

If the U-section is loaded with horizontal forces and upward vertical forces the same check of the web of the U-section is carried out as for angle fixtures in 13.2.2. If downward vertical forces exist the carrying capacity of the web is checked against the condition:

$$\sqrt{\sigma^2 + 3\tau^2} \leq f_{yd}$$

Formula 13.19

where

f_{yd} = design value of the yield tension limit of the U-section

σ = longitudinal stress and

τ = shear stress calculated at the same point on the U-section

If σ and τ are calculated with the Elastic Theory as the starting point at the same time as a biaxial stress condition applies, f_{yd} in formula 13.19 can be replaced by 1,1 f_{yd}.

The maximum longitudinal stress σ_{max} occurs at the edge of the fixture in cross-section A-A and can be calculated using:

$$\sigma_{max} = \frac{0{,}5H_d \cdot \left(\frac{b-d}{2}\right)}{W_x} + \frac{\frac{0{,}5V_d}{b}\left(\frac{b-d}{2}\right)^2}{W_y}$$

Formula 13.20

where

H_d and V_d

are critical horizontal and downward vertical forces respectively, calculated per column

W_x and W_y

are the section modulus of the web.

Other symbols are given in figure 13.6.

The shear stress τ at the same point is 0.

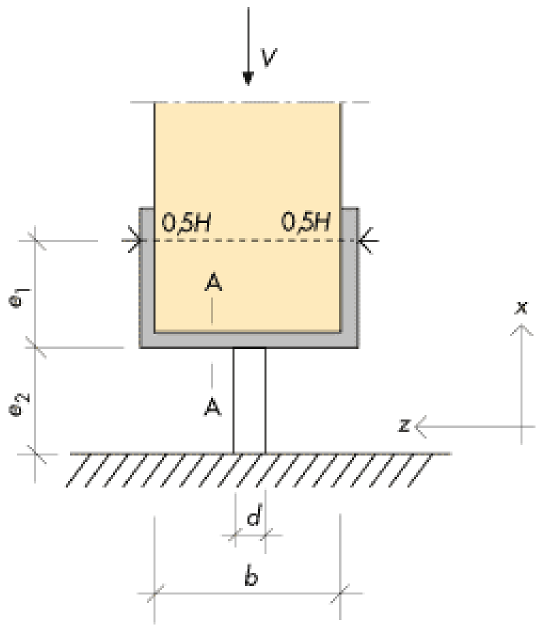


Figure 13.6. Base shoe in the form of a U-section. Symbols.

The maximum shear stress τ_{max}occurs at the same level as the neutral zone in bending about the y-axis and can be calculated from the formula:

$$\tau_{max} = 1{,}5\frac{0{,}5V_d}{A}\left(\frac{b-d}{b}\right)$$

Formula 13.21

where

*A*is the area of the web of the U-section.

The longitudinal stress σat the top and bottom of the web, calculated at a cross-section through the rod/hollow section is:

$$\sigma = \frac{\frac{1}{2}\cdot\frac{0{,}5V_d}{b}\left(\frac{b-d}{2}\right)^2}{W_y}$$

Formula 13.22

For the shear stress τat the same point, the following applies:

$$\tau = 1{,}5\frac{0{,}5H_d}{A} + \frac{0{,}5H_d \cdot e_1}{W_v}$$

Formula 13.23

where

$W_v$is the torsion constant of the web (=*A*x*t*/3)

Distance $e_1$is shown in figure 13.6.

The design condition with regard to contact pressure between the column and the fixture is:

$$V_d \leq f_{cd} \cdot b \cdot c$$

Formula 13.24

where

f_{cd} = critical value of the compressive strength of the glulam column parallel with the grain.

c = length of the U- section

b = width of column

The risk of punching-through in the U-section is checked using formula 13.19. The longitudinal stress is assumed to be 0 and the shear stress calculated from the formula:

$$\tau = \frac{V_d - f_{cd} \cdot d^2}{s \cdot t}$$

Formula 13.25

where

s= circumference/perimeter of the rod/section

t= thickness of the web of the U-section

The weld between the U-section and the rod/hollow section is a butt weld and a calculation check of its strength is not required. If the utilisation of the weld is limited to 70%, the workmanship need not be checked ultrasonically or by other means.

The carrying capacity of the rod/section is checked with the aid of the design condition (13.19), The maximum longitudinal stress occurs at the fixing section (intersection of the rod/ screw with the concrete surface) and is calculated using the formula:

$$\sigma_{max} = \frac{H_d(e_1 + e_2)}{W_2} + \frac{V_d}{A}$$

Formula 13.26

where

A = cross-sectional area of rod/section

W_2 = section modulus of the rod/section

$e_1$and $e_2$are shown in figure 13.6.

The shear stress at the point of maximum longitudinal stress is 0.

Maximum shear occurs in the neutral zone and is calculated using the formula:

$$\tau_{max} = k_\tau \frac{H_d}{A}$$

Formula 13.27

where

A= cross-sectional area

k_kτ/sub> = 1,5 for square solid cross-section

k_kτ/sub>1,3 for circular, solid cross-section

k_kτ/sub>2,0 for circular tube

k_kτ/sub>2,5 for square tube

Longitudinal stress at the same point is 0.

Buckling of the rod/section is checked against Eurocode 3, Design of Steel Structures (EN 1993). The reduction factor for buckling $\omega_c$depends on the slenderness of the rod/section $\lambda_c$which is:

$$\lambda_c = 0,67 \frac{e_2}{i} \sqrt{\frac{f_{yk}}{E_k}}$$

Formula 13.28

where

f_{yk} = characteristic value of yield tension limit

E_k = characteristic value of modulus of elasticity

i = radius of inertia

For steel whose f_{yk}= 270 N/mm^2, the risk of buckling need not be checked if the free length of the rod/section fulfils the condition:

$$e_2 \leq 8,3 \cdot i$$

Formula 13.29

The design condition regarding fixing by adhesion of the rod/section cast into the foundation is:

$$\left(\frac{V}{V_d}\right)^2 + \left(\frac{H}{H_d}\right)^2 \leq 1,0 \qquad (13.30)$$

Formula 13.30

where

$V_d = f_{cv} \cdot s \cdot L$

s = perimeter of rod/section

L = length cast-in

f_{cv} = design value of the adhesion strength

$$H_d = 1,75d \cdot f_c \left(\sqrt{(e_1 + e_2)^2 + \frac{2W_{pl} \cdot f_{st}}{d \cdot f_{cc}}} - e_1 - e_2 \right)$$

W_{pl} = plastic section modulus of the rod/section

d = diameter of the rod/section

f_{cc} = design value of the compressive strength of concrete regarding local pressure

f_{st} = design value of bending strength of rod/section material

Other symbols are shown in figure 13.6.

For concrete of strength class K25 the characteristic value of adhesive strength can be taken as follows, provided the edge distance or other factors do not reduce the strength:

f_{cvk} = 1,2 N/mm^2 for plain rods or sections without fixing at the end

f_{cvk} = 4,2 N/mm^2 for ribbed rods

f_{cck} = 30 N/mm^2

The surrounding concrete construction is designed in accordance with current concrete regulations.

If the connection to the foundation is made with a cast-in fixing plate the weld and the fixing plate must be checked as for fishplates of nailplate or flat steel (see 13.2.1).

14.2.4.13.2.4 Glued-in bolts

Pinned fixing with glued-in bolts is a completely hidden fixing, something which can be desirable for aesthetic, fire and other reasons (see figure 13.7).

Glued-in bolts cannot be used in climate class 3, or in structures subject to dynamic loads or fatigue loads. The gluing operation is always carried out in the factory.

The connection to the foundation is as a rule designed with an end plate on the column which is welded to a fixing plate cast into the foundation. The end plate is threaded on to the glued-in bolt. If several glued-in bolts per column end are required the end plate can suitably be welded to them. If the connection is designed with projecting bolts intended to be cast into pockets, the bolts must be protected against damage during transport. The column must be braced until the concrete has hardened.

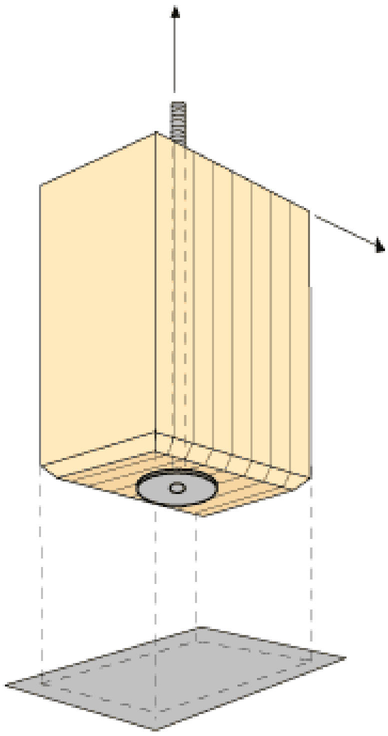


Figure 13.7 Pinned fixing of column base with glued-in bolt. Principle.

The normal version with only one glued-in bolt per column hardly permits moments to be taken up and assumes that the column is braced until the structure has been stabilised.

If several glued-in bolts are required, they shall be placed as near each other as possible as the shortest permitted distance allows.

DESIGN

The number of bolts, their dimensions and glued-in lengths are determined with regard to horizontal forces and any uplift forces. Downward vertical forces can be transferred direct to the foundation by contact pressure.

The design condition for a bolt glued into a column is:

$$\left(\frac{F_v}{R_{vd}}\right)^2 + \left(\frac{F_t}{R_{td}}\right)^2 \leq 1{,}0$$

Formula 13.31

where

$F_v = H_d / n$ and $F_t = V_d / n$

H_d = design horizontal force per column

V_d = design vertical force (uplift) per column

n = number of glued-in bolts per column

R_{vd} = design carrying capacity of bolt in shear, calculated as below or according to Eurocode 5 (EN 1995-1-1)

R_{td}= design carrying capacity of bolt in axial tension, calculated as below or according to Eurocode 5.

The characteristic carrying capacity in shear R_{vk} can be calculated by the following formula:

$$R_{vk} = 12\, d^2 \sqrt{\frac{f_{yk}}{240}}$$

Formula 13.32

where

d = diameter of the bolt

f_{yk} = characteristic value of the yield tension limit of the bolt

Formula 13.32 applies on condition that the glued-in length of the bolt is at least 8d, the distance to the edge at least 4d and the distance between bolts at least 4d.

Formulae for calculating the characteristic value of the carrying capacity R_{tk} against withdrawal are given e.g. in Eurocode 5.The critical carrying capacity may be calculated from the following expression:

$$R_{td} = \min\begin{cases} 10 \cdot d \cdot L \cdot \kappa_r / \gamma_n \\ 0{,}5 \cdot A_{net} \cdot f_{buk} / \gamma_n \end{cases}$$

Formula 13.33

where

d = nominal bolt diameter

L = glued in length (but max 350 mm)

A_{net} = stress area

f_{buk} = characteristic value of ultimate strength of the bolt material

κ_r = modification factor which takes into account effects of moisture content and duration of load

γ_n = partial coefficient for safety class if relevant

The glued-in length should be sufficiently large for the bolt to yield before it is withdrawn. For steel with a yield tension limit of 350 N/mm^2 this means that the glued-in length must be at least 19*d*. This assumes that the distance between bolts is at least 3*d*. This can be reduced to 2*d*, but check that the whole group of bolts is not pulled out.

The design condition with regard to the strength of concrete for a bolt glued into the foundation without end anchorage can be written:

$$\left(\frac{F_v}{R_{vd}}\right)^2 + \left(\frac{F_t}{R_{td}}\right)^2 \leq 1,0 \qquad (13.34)$$

Formula 13.34

where

$R_{td} = f_{cv}.\ s.\ L$

$$R_{vd} = 2,5\sqrt{W_{pl} \cdot d \cdot f_{cc} \cdot f_{st}}$$

s = circumference of bolt

L = length of cast-in part

F_{cv} = design value of strength of adhesion

$W_{pl} = d^3/6$ = plastic section modulus of bolt

d = diameter of bolt

f_{cc} = design value of the compressive strength of concrete in local pressure

f_{st} = design value of bending strength in the material of the bolt

The characteristic value of the strength of adhesion can be taken as f_{cvk}= 3,5 N/mm^{2}for threaded bolts, cast into concrete of strength class K25. The cover and the clear space between rods are assumed to be 2,5 Ø and 5 Ø respectively. If these distances are reduced, the adhesion is also reduced, which usually gives unpractical long casting-in lengths. If there is more than one glued-in bolt in the fixture it is therefore often suitable to provide the bolts with end anchorage designed to take the whole withdrawal force.

The surrounding concrete structure is designed in accordance with Eurocode 2 (EN 1992).

If the connection with the foundation is made with a circular end plate threaded on to the bolt, which is welded to a cast-in fixing plate as in figure 13.8, this should be designed to ensure workmanship which is as good as possible. The diameter of the end plate should then be not more than 20 50 mm less than the smaller dimension of the column. For the same reasons the thickness of the plate should be at least 15 mm.

The design condition for the fillet weld between the end plate and the cast-in fixing plate is:

$$\left(\frac{H_d}{F_{R//}}\right)^2 + \left(\frac{V_d}{F_{Ra}}\right)^2 \leq 1$$

Formula 13.35

where

$F_{R//}$and $F_{Ra}$are the design carrying capacities of the weld in longitudinal and transverse directions respectively. The characteristic value of the carrying capacity longitudinally and laterally can be calculated using the following formulae:

$$F_{R//} = 0{,}3\, a \cdot \pi \cdot D \cdot f_{wd}$$

Formula 13.36

$$F_{Rz} = \frac{a \cdot \pi \cdot D \cdot f_{wd}}{\sqrt{2}}$$

Formula 13.37

where

f_{wd} = design strength value of the weld

a = size of the weld

D = diameter of the end plate.

Because of the risk of lamellar tearing in the fixing plate and to any requirements on checking of the weld, the degree of utilisation in formula 13.35 should not exceed 70% and if possible be limited to 50%. As a rule the weld between the end plate and the fixing plate will not be critical if the size is 3 mm.

The cast-in fixing plate is checked as for fishplates of nailplate or flat steel (see 13.2.1)

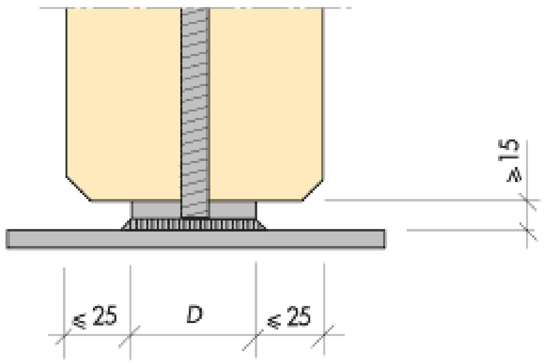


Figure 13.8 Fixing with end plate welded to fixing plate cast into the foundation.

14.3.1.13.3.1 Fishplates of nailplate or flat steel

Fixing with fishplates of nailplate or flat steel is a simple and cheap method which can be used for large moments. In contrast to the design of pinned column bases the fishplates are placed on the narrow sides of the column (see figure 13.9).

The fixture can be designed with a baseplate for fixing with anchor bolts or without a baseplate for welding to a cast-in fixing plate. When fixing with baseplates each fishplate can suitably have its own baseplate. The same fixtures can then be used for columns of different cross-sectional sizes.

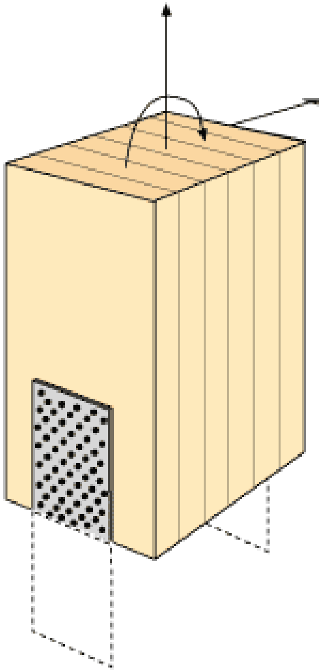


Figure 13.9. Rigidly fixed column base with fishplates of nail plate. Principle.

The fixture can also be designed to be cast into the foundation itself or into pockets in the foundation. To avoid problems with concreting tolerances this solution should preferably be used in conjunction with nails or wood screws.

The transfer of forces between fishplate and glulam column takes place with the aid of nails, bolts or wood screws. To increase carrying capacity, bolts and wood screws can suitably be combined with connectors. These should be fitted at the factory, since the pressing-in demands special equipment. The nailed joint is the stiffest alternative and provides the most effective transfer of forces per unit of area. The risk of splitting is however great if the specified nail spacing is not followed. In a screwed/bolted fixture, bolts should be used if the depth (*h*) of the column section is less than 500 mm. Wood screws are preferable for larger columns.

Pre-drilled nailplates of galvanized steel are not suitable for welding, as poisonous gases are formed by the heating-up of the zinc coating. They should only be used when the fishplates are fixed by being cast into the foundation. Nailplates with different hole patterns and thicknesses can be ordered from special manufacturers of perforated sheet steel. The cost is generally lowest when the holes are punched out, which means that the thickness must not exceed the hole diameter. The holes should be about 1 mm larger than the thickness of the nail.

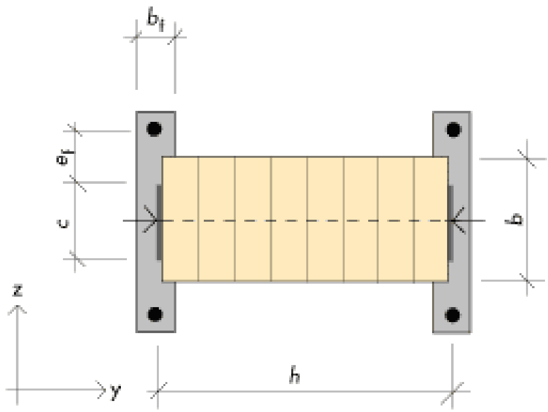


Figure 13.10. Fishplates with baseplates. Symbols.

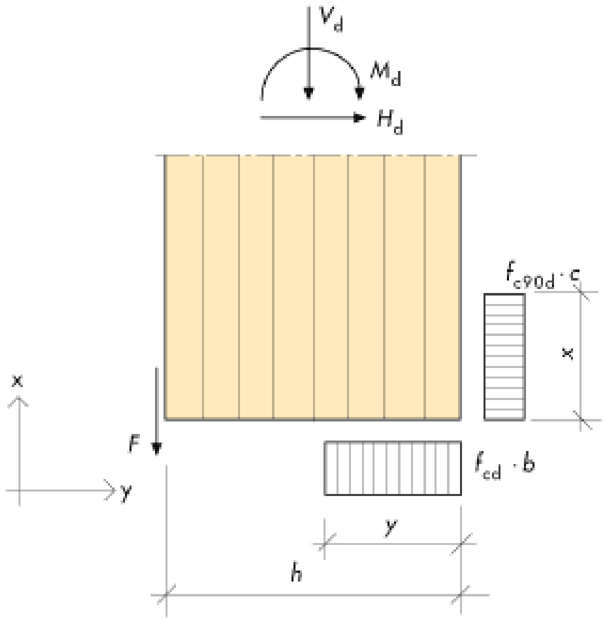


Figure 13.11. Rigid fixing with fishplates. Calculation model with compression stress blocks. Symbols.

Fishplates for screwed or bolted joints can suitably be made of flat steel. Thicknesses are chosen from the standard range of 6, 8, 10, 12, 15 and 20 mm and shall be at least 0,3 times the bolt diameter (0,4 times for connectors). The holes are drilled 1 mm larger than the screw/bolt diameter.

DESIGN

In designing it is assumed that the tension force from the fixing moment, and in certain cases the vertical uplift, is transferred via the nail or screw/bolt joint to the fishplates and further to the foundation. Compression force from the fixing moment and downward vertical force is transferred to the foundation through contact pressure. Horizontal forces are transferred from the column to the fishplate on the leeward side by contact pressure perpendicular to the grain.

The design condition for the fixing of the fishplate on the windward side is:

$$\frac{F}{n} \le R_{vd}$$

Formula 13.38

where

R_{vd} = design value of carrying capacity of a nail or screw/bolt loaded in shear

n = number of nails or screws/bolts per fishplate

F = design tension force in the fishplate

Fcan be calculated using the formula:

$$F = f_{cd} \cdot b \cdot y - V_d$$

Formula 13.39

where V_d= design value of vertical force per column (positive for compression, negative for tension)

f_{cd} = design value of compressive strength of glulam column in direction of grain

c = width of fishplate

For concrete of strength class K25, f_{cck}can be taken as 30 N/mm^2.

The length $y$of the compression stress block is calculated from the condition for moment equilibrium at the end surface of the column, figure 13.11, and can be written:

$$y = h\left(1 - \sqrt{1 - \frac{2M_d + V_d \cdot h}{f_{cd} \cdot b \cdot h^2}}\right)$$

Formula 13.40

where

M_d = design value of end moment

h = cross-sectional depth of column

The condition that formulae 13.39 and 13.40 shall apply is that longitudinal stresses resulting from the longitudinal force are considerably smaller than the bending stresses which are caused by the end moment. Otherwise the contact pressure under the column will not be concentrated to the compressed edge as has been assumed when deriving the formulae.

The design condition for the fishplates on the windward side is:

$$F \le A \cdot f_{yd}$$

Formula 13.41

where

f_{yd} = design value of the yield tension limit of the fishplate

A = cross-sectional area of fishplate

The possible effect of screw/bolt and nail holes is taken into account by replacing A with A_{net} and f_{yd} with f_{ud}, which means that the fishplate is allowed to plasticise.

The critical condition for the fishplates on the leeward side is:

$$H_d \leq f_{c90d} \cdot c \cdot x$$

Formula 13.42

where

H_d = design horizontal force per column

f_{c90d} = design value of the compressive stress of the glulam column perpendicular to the grain

c = width of the fishplate

The height of the compression stress block x is calculated using the Theory of Plasticity, based on two plastic hinges being formed in the fishplate, and can be written:

$$x = t\sqrt{\frac{f_{yd}}{f_{c90d}}}$$

Formula 13.43

where

t = thickness of the fishplate.

If the fishplate is fillet welded to the baseplate from both sides, the weld between the fishplate on the windward side and the baseplate shall fulfil the condition:

$$F \leq F_{R\alpha}$$

Formula 13.44

where

F_{Ra} = design value of the lateral carrying capacity of the weld

F_{Ra} can be calculated using the formula:

$$F_{R\alpha} = \frac{2a \cdot c \cdot f_{wd}}{\sqrt{2}}$$

Formula 13.45

where

f_{wd} = design value of the strength of the weld

a = size of the weld, not less than 3 mm. Up to 5 mm can be laid in one run.

c = width of fishplate

If the welded joint between the fishplate and the baseplate is utilised to more than 70%, some regulations demand that the weld be checked by non-destructive testing, e.g. X-Ray. Due to the risk of lamellar tearing of the baseplate it is often demanded that if it is utilised to over 50%, it must have verified properties in the direction of thickness, or be checked ultrasonically. It is therefore often practical to make the fixing plate of (ultrasonically) controlled material and to limit the utilisation in the fishplate in formula 13.41 to 70% while at the same time designing the weld so that it is as strong as the fishplate.

The width and length of the baseplate are as a rule determined by the size of the glulam column and the free space requirement and edge distance of the anchor bolts. The thickness is determined so that the bending stress in the baseplate caused by tension forces in the anchor bolts does not exceed the design strength value. For the baseplate in figure 13.10 the condition is:

$$0{,}5\,F\cdot e_f \leq f_{yd}\cdot W_p$$

Formula 13.46

where

e_f = the lever as defined in figure 13.10

f_{yd} = design value of the yield tension limit of the baseplate

W_p = plastic section modulus of the baseplate

(W_p= b_fx$t_f^2/4$)

The carrying capacity of the anchor bolts with reference to the strength of the concrete can be checked with the following formula:

$$\left(\frac{F_v}{R_{vd}}\right)^2 + \left(\frac{F_t}{R_{td}}\right)^2 \leq 1,0 \qquad (13.47)$$

Formula 13.47

where

$F_v = 2H_d/n$

$F_t = F/n/2$

H_d = design horizontal force per column

F = design force in bolt group in tension as in formula 13.39

n = number of glued-in bolts per column

The carrying capacity of the bolts can be calculated as for cast-in, glued-in bolts (see 13.2.4).

$R_{td} = f_{cv}$xsxL

$$R_{vd} = 2,5\sqrt{W_{pl}\cdot d\cdot f_{cc}\cdot f_{st}}$$

s= circumference of bolt

L= cast-in length

F_{cv} = design value of bond strength

$W_{pl} = d^3/6$ = Plastic section modulus of bolt

d = diameter of bolt

f_{cc} = design value of the strength of the concrete in local compression

f_{st} = design value of the strength of the material of the bolt in bending

The characteristic value of the bonding strength can be taken as f_{cvk}= 3,5 N/mm^{2}for bolts with annular threads, cast into concrete of strength class K25. The characteristic value of compressive strength can be taken as f_{cck}= 30 N/mm^2.

If the fishplates are cast direct into the foundation the horizontal force is transferred through contact pressure with the fishplate on the leeward side. The carrying capacity can be regarded as sufficient if the conditions in formula 13.42 are satisfied.

If the withdrawal load is moderate the bond between concrete and steel can be utilised. Where the tension forces are large both cast-in fishplates and cast-in ground bolts must be provided with some type of end anchorage, in which case the bond is ignored and the whole load taken by the anchorage.

The design condition regarding anchorage for a fishplate cast into the foundation on the windward side (without end anchorage) is:

$$F \leq f_{cv} \cdot s \cdot L$$

Formula 13.48

where

s= perimeter of the fishplate

L= cast-in length

f_{cv} = design value of the bond strength.

For concrete of strength class K25, it can be assumed that

f_{ck}= 1,2 N/mm^{2}for flat steel and 3,5 N/mm^{2}for punched nail plate.

14.3.2.13.3.2 Glued-in bolts

When glued-in bolts are used for fixing column bases the fixing is completely hidden (see figure 13.12), which in certain cases can be desirable for aesthetic, fire and other reasons. The necessary edge distances and distances between bolts mean, however, that the moment capacity of the column is poorly utilised. Fixing with glued-in bolts is therefore only suitable for relatively small end moments.

Glued-in bolts may not be used in service class 3 or in structure subject to dynamic loads or fatigue loads. The gluing-in process is always carried out in the factory.

The connection to the foundation can be designed either for casting-in direct in pockets in the concrete, or with a steel base for welding to a cast-in fixing plate. If the connection is designed with projecting bolts for direct casting-in, the bolts must be protected during transport and erection. The column shall be braced until the concrete has hardened.

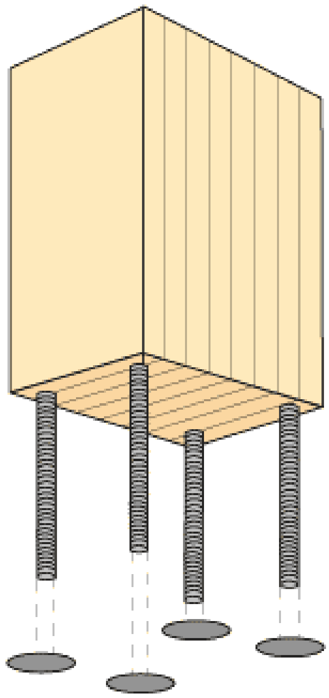


Figure 13.12 Rigidly fixed column base with glued-in bolt. Principles.

DESIGN

The number of bolts, their dimensions and gluing-in lengths are determined with regard to end moments, horizontal forces and any uplift. Downward vertical forces in the column are transferred direct to the foundation by contact pressure.

The design condition for an individual glued-in bolt is:

$$\left[\frac{F_v}{R_{vd}}\right]^2 + \left[\frac{F_t}{R_{td}}\right]^2 \leq 1$$

Formula 13.49

where

$F_v = H_d/n$

$F_t = F/n/2$

H_d = design value of horizontal force per column

F = design value of withdrawal force acting on the group of bolts in tension

n = number of glued-in bolts

R_{vd}= design value of capacity per bolt in shear

R_{td} = design value of carrying capacity per bolt against withdrawal

$R_{vd}$and R_{td}can be calculated with the aid of formulae 13.32 and 13.33 respectively.

If the column is placed directly on the concrete structure, Fcan be calculated from the formula:

$$F = f_{cd} \cdot b \cdot y - V_d$$

Formula 13.50

where

V_d= design value of vertical force per column.

($V_d$is positive for compression and negative for tension)

f_{cd} = design value of the strength of the glulam column in compression in the direction of the grain, but not more than the corresponding strength of concrete f_{cc}in local compression

Other symbols, see figure 13.13.

For concrete of strength class K25 it can be assumed that f_{cck}= 30 N/mm^2.

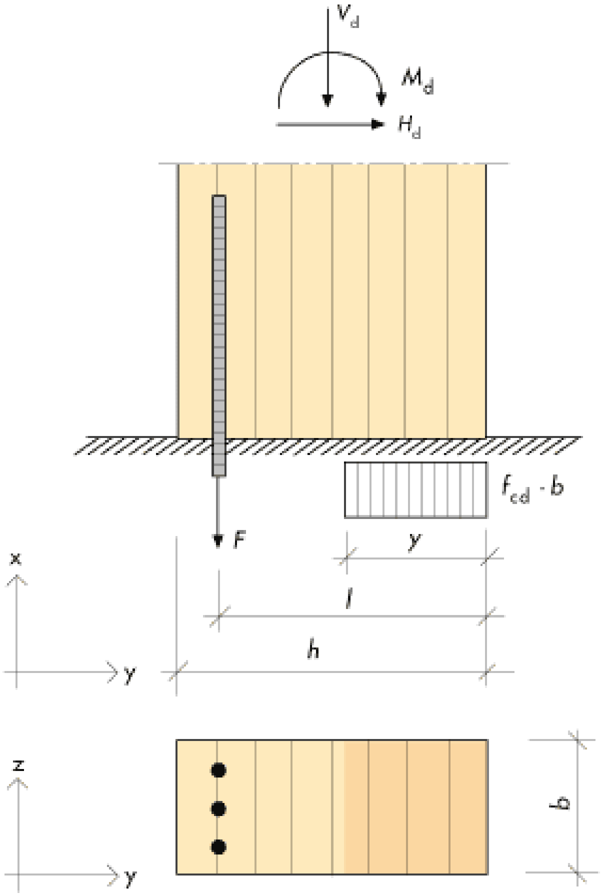


Figure 13.13 Rigidly fixed column base with glued-in bolt. Calculation model.

The extent *y*of the compression stress block is calculated from the condition for moment equilibrium at the end surface of the column, which can be written:

$$y = l \cdot \left(1 - \sqrt{1 - \frac{2M_d + V_d(2l - h)}{f_{cd} \cdot b \cdot l^2}} \right)$$

Formula 13.51

where

$M_d$is the design value of the end moment.

If the connection is not direct with the concrete structure but with a steel base, the contact pressure will be concentrated to its edges which are stiffened by side plates. In this case *F*can instead be calculated from the formula:

$$F = f_{cd}\left(2c \cdot y + b \cdot c - 2c^2\right) - V_d$$

Formula 13.52

where

f_{cd} = design value of the compression strength of the glulam column parallel to the grain.

Other symbols are to be found in figure 13.14.

The extent *y* of the compression stress block can be calculated from the condition for moment equilibrium at the end surface of the column, which in this case is:

$$y = l \cdot \left(1 - \sqrt{1 + A - \frac{2M_d + V_d(2l - h)}{f_{cd} \cdot 2c \cdot l^2}}\right)$$

Formula 13.53

where

$$A = \frac{(b - 2c)(2l - c)}{2l^2}$$

Formula 13.54

$$c = t\left[1 + \sqrt{\frac{f_{yd}}{2f_{cd}}}\right]$$

Formula 13.55

t = thickness of the steel base

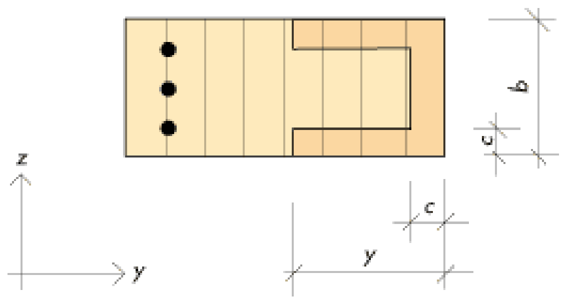


Figure 13.14 Rigidly fixed column base with glued-in bolts and steel base. Calculation model.

A condition for the application of the formulae 13.50 13.55 is that the longitudinal stresses resulting from longitudinal forces are small in relation to the bending stresses which the end moment causes in the contact surface. When moments are small, the contact pressure under the column is not concentrated to the compression edge as has been assumed when deriving the formulae.

If the bolts are cast-in direct in pockets in the concrete, the carrying capacity can be checked regarding the strength of concrete as in 13.3.1, design condition (formula 13.47) and associated formulae.

The cover and the clear space between bolts are assumed to be 2,5 *d* and 5 *d* respectively. If these distances are reduced, the bond is also reduced, which usually gives unpractical long casting-in lengths. If there is more than one glued-in bolt in the fixture it is therefore often suitable to provide the bolts with end anchorage designed to take the whole withdrawal force.

The surrounding concrete structure is designed in accordance with Eurocode 2 (EN 1992).

If the connection to the foundation is in the form of a steel plinth this can with advantage be made from half hollow square steel sections, which are available in a number of sizes and thicknesses (see figure 13.5).

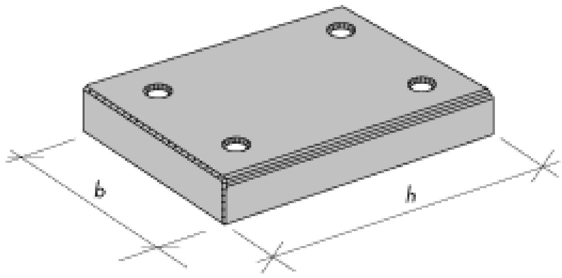


Figure 13.15 Steel base for rigidly fixed columns with glued-in bolts.

The tension in the bolts causes bending moments in the base. The carrying capacity with regard to bending stresses shall satisfy the condition:

$$m_p \leq f_{yd} W_p$$

Formula 13.56

where

f_{yd} = design value of the yield tension limit of the steel plinth

W_p = plastic section modulus of plinth ($W_p = t^2/4$)

m_p = bending moment in plinth per unit of width

The bending moment m_p is calculated using the Theory of Plasticity (see figure 13.16) and can be written:

$$m_p = \frac{F \cdot c}{8c + 4b - n \cdot d}$$

Formula 13.57

where

F = design value of withdrawal force as in formula 13.52

c and b can be found in figure 13.16.

Welding between the steel plinth and the cast-in fixing plate can only be done from the outside. If the plinth is chamfered and butt welded throughout, the carrying capacity of the weld need not be checked. If however the plinth is fillet welded to the cast-in fixing plate the stresses in the welds must be checked. Tension and compression forces can then be assumed to be transferred by the edge welds and lateral forces of the web welds.

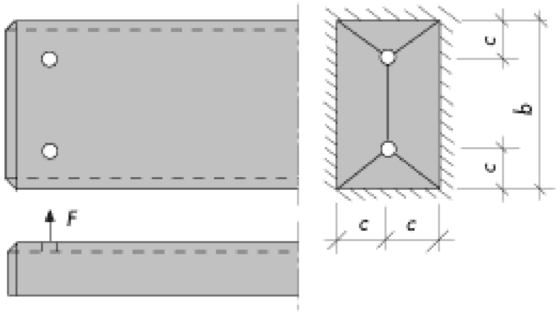


Figure 13.16 Steel base. Calculation model.

The design condition for the weld between the edge and the fixing plate is:

$$\frac{M_d}{h} + 0,5V_d \le F_{R\alpha}$$

Formula 13.58

where

F_Fα/sub> = design value of carrying capacity of the weld in the lateral direction

F_Fα/sub> can be calculated using the formula:

$$F_{R\alpha} = \frac{a \cdot b \cdot f_{wd}}{\sqrt{2}}$$

Formula 13.59

where

f_{wd} = design value of the strength of the weld

The design condition for the weld between one edge and the fixing plate is:

$$0,5H_d \le F_{R//}$$

Formula 13.60

where

$F_{R//}$ = design value of carrying capacity of the weld in longitudinal direction

$F_{R//}$can be calculated using the formula:

$$F_{R||} = 0,6a \cdot h \cdot f_{wd}$$

Formula 13.61

where *a* = size of the weld.

If the degree of utilisation in formulae 13.60 and 13.58 is more than 70%, some regulations demand that the weld be checked by non-destructive testing, e.g. X-Ray. Due to the risk of lamellar tearing of the baseplate it is often demanded that if it is utilised to over 50%, it must have guaranteed and verified properties in the direction of thickness, or be checked ultrasonically. It is therefore often practical to make the fixing plate of (ultrasonically) controlled material and to limit the utilisation in the welds in the formula to 70%. As a rule the weld between steel base and fixing plate is not critical if the size is chosen as 3 mm.

The width and length of the cast-in fixing plate are as a rule determined by the size of the glulam column and the amount of space needed for the fixture and its fixing, with the necessary permitted deviations. The distribution of forces under the fixing plate is assumed (simplified) to be comparable to a compression zone centred round the compression edge of the base, and a tension force in the anchoring bolts. The thickness is determined so that the bending stress in the fixing plate does not exceed the critical value of the yield tension limit of the fixing plate. With symbols as in figure 13.17 the condition can be written as:

$$F^* \cdot e \leq f_{yd}\, W_p$$

Formula 13.62

where

f_{yd} = design value of the yield tension limit of the material of the fixing plate

W_p = $b_f x t_f^2/4$ plastic section modulus of the fixing plate

F^* = design value of the withdrawal force acting in the centre of the group of anchor bolts.

F^* can be calculated using the formula:

$$F^* = f_{cc} \cdot b_f \cdot y - V_d$$

Formula 13.63

where

f_{cc} = design value of the strength of the concrete in local compression

b_f = width of the fixing plate

For concrete of strength class K25, it can be assumed that F_{cc}= 30 N/mm^2.

The extent of the compression zone $y$is calculated from the condition for equilibrium of moments and can, with symbols as in figure 13.17, be written:

$$y = \frac{M_d + 0{,}5V_d\left(2l_f - h\right)}{f_{cc} \cdot b_f \cdot l_f}$$

Formula 13.64

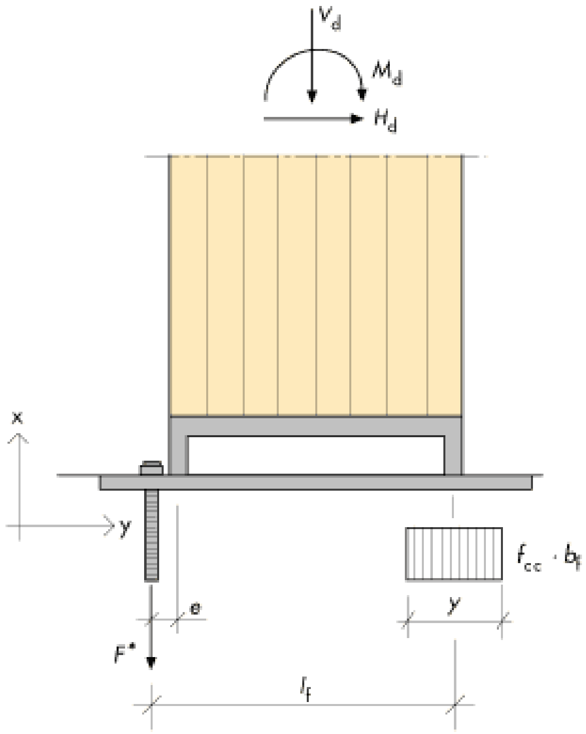


Figure 13.17 Rigidly fixed column base with glued-in bolts and steel base. Calculation model.

The compression zone must lie entirely within the edges of the fixing plate. Otherwise the contact pressure between the fixing plate and the concrete will be critical and the fixing plate must be increased in size.

The surrounding concrete structure is designed in accordance with Eurocode 2 (EN 1992).

The carrying capacity of the anchor bolts with regard to the strength of concrete can be checked in the same way as when fixing with fishplates (see 13.3.1). Design condition formula 13.47 applies, as in the related formulae, if *F*is replaced by *F**.

14.4.1.13.4.1 Fishplates of nailplate or flat steel

Pinned connections with fishplates of nailplate or flat steel, fixed to the wide faces of the column's cross-section, are a simple and functional method, suitable both for large and small forces (see figure 13.18).

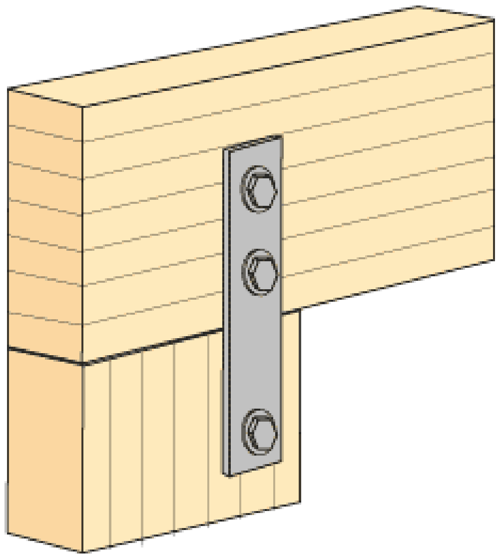


Figure 13.18 Pinned connection between beam and column with fishplates of flat steel. Principles.

The transfer of forces between the fishplate and glulam column or beam takes place with the aid of nails, bolts or wood screws. Which type of fastening shall be used is decided on the basis of aesthetic, economic, erection and strength requirements.

The fishplates should be placed as near as possible to the inner side of the column so as not to restrict rotation of the beam. A suitable distance between the inner side of the column and the centre of the bolt is 4*d*if the joint only transfers horizontal forces or 2*d*if only vertical forces are transferred. If nailplates are used the corresponding distances are 10*d*and 5*d*respectively.

Pre-drilled nailplates of galvanized steel, 1,5 5,0 mm thick, are a cheap alternative for moderate loads. Nailplates with various hole patterns and thicknesses can be ordered from special manufacturers of perforated sheet steel. The cost is generally lowest when the holes are punched out, which means that the thickness must not exceed the hole diameter. The holes should be about 1 mm larger than the thickness of the nail.

Fishplates for bolted joints can suitably be made of flat steel. Thicknesses are chosen from the standard range of 6, 8, 10, 12, 15 and 20 mm and shall be at least 0,3 times the bolt diameter. The holes are drilled 1 mm larger than the bolt diameter.

DESIGN

In the design the fishplate is regarded as a beam rigidly fixed at both ends, loaded with a horizontal force and in certain cases with a vertical uplift. The forces are assumed to act at the centres of the screw/bolt or nail groups (see figure 13.19).

The eccentricity moment 0,5*H*× *e*is taken into account in the design. Downwards vertical forces are transferred to the column by contact pressure and are not loading the fishplates.

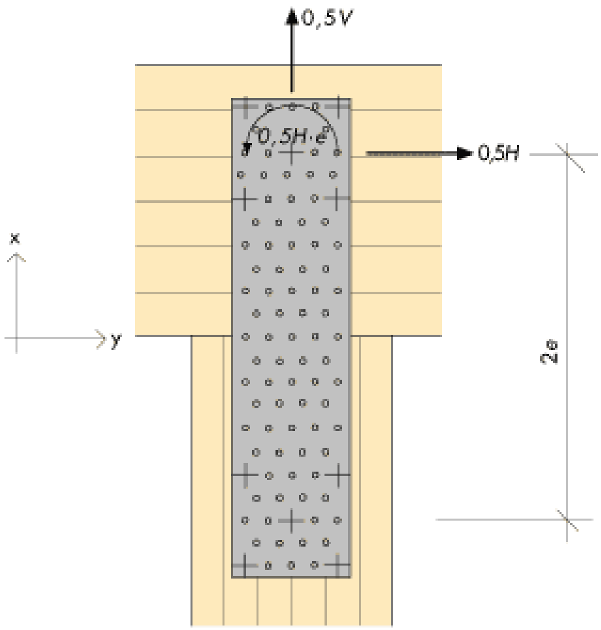


Figure 13.19. The fishplate is regarded as a beam rigidly fixed at both ends. Symbols.

The design condition for the fixing of an individual fishplate in the beam is:

$$\sqrt{F_x^2 + F_y^2} \leq R_{vd}$$

Formula 13.65

where

R_{vd} = design value of carrying capacity of a nail or screw/bolt in shear.

The carrying capacity of an individual connection is determined with regard to the angle between the force and the grain.

$F_x$and $F_y$are calculated using the formulae:

$$F_x = \frac{0{,}5V_d}{n} + \frac{0{,}5H_d \cdot e \cdot r_y}{I_p}$$

Formula 13.66

$$F_y = \frac{0{,}5H_d}{n} + \frac{0{,}5H_d \cdot e \cdot r_x}{I_p}$$

Formula 13.67

where

H_d = design value of horizontal force from the beam

V_d = design value of uplift from the beam

n = number of fasteners between each fishplate and the beam

$2e$ = distance between centres of gravity of nail/screw/bolt groups (figure 3.19)

r_y, r_x = distance in y and x direction respectively between centre of gravity of nail/screw/bolt group and the individual nail, bolt or screw

I_p = polar moment of inertia of nail/screw/bolt group

The polar moment of inertia is calculated using the formula:

$$I_p = \sum (r_x^2 + r_y^2)$$

Formula 13.68

The design condition for an individual fishplate is:

$$\sqrt{\sigma^2 + 3\tau^2} \leq f_{yd}$$

Formula 13.69

where

f_{yd} = design value of the yield tension limit of the fishplate

σ = axial stress at a certain point on the fishplate

τ = shear stress at the same point

If σ and τ are calculated in accordance with the Theory of Elasticity at the same time as a biaxial stress condition exists, f_{yd} in formula 13.69 can be replaced by $1{,}1 \cdot f_{yd}$.

The largest normal stress σ_{max} exists in the tension edge of the fishplate and a value on the safe side can be calculated using the formula:

$$\sigma_{max} = \frac{0{,}5 H_d \cdot e}{W} + \frac{0{,}5 V_d}{A}$$

Formula 13.70

where

A = cross-sectional area of the fishplate

W = section modulus of the fishplate

The shear stress τ at the point of maximum bending stress is 0.

The maximum shear stress acts at the same height as the neutral axis in bending about the z axis and is calculated from the formula:

$$\tau_{max} = 1{,}5 \frac{0{,}5 H_d}{A}$$

Formula 13.71

The axial stress σ at the same point on the cross-section is calculated from the formula:

$$\sigma = \frac{0{,}5 V_d}{A}$$

Formula 13.72

The effect of screw or nail holes on the capacity of the fishplate is taken into account by inserting stresses in formula 13.69 based on the net cross-section, i.e. with A_{net} and W_{net} and f_{yd} instead of f_{ud}. This means that the fishplate is allowed to plasticise round the holes.

For thin fishplates the risk of buckling at the compression edge of the plate must be checked in accordance with Eurocode 3 (EN 1993).

The slenderness parameter α_b can then be determined as in 13.2.1.

The fixing of an individual fishplate in the column is checked in the same way as fixing in the beam but with regard to the fact that the direction of the grain differs between beam and column.

The design condition regarding contact pressure between beam and column is:

$$V_d \leq f_{c90d} \cdot b \cdot h$$

Formula 13.73

where

V_d = design value of vertical force (downward)

F_{c90d} = design value of the compression strength of the glulam beam across the grain.

b = cross-sectional width of the column

h = cross-sectional depth of the column

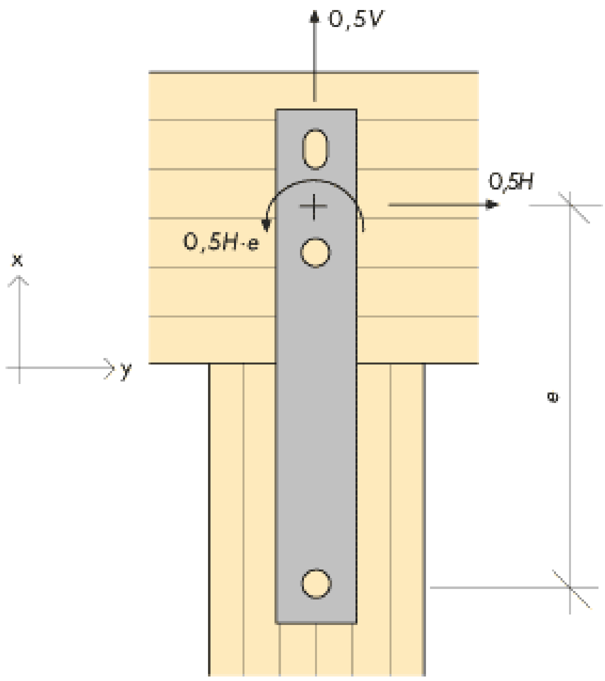


Figure 13.20.The fishplate is regarded as a beam rigidly fixed at one end and freely supported at the other. Symbols.

The fishplate in figure 13.20 is assumed in the design to be rigidly fixed to the beam and pinned to the column. The moment of eccentricity is taken into account when designing the fixing to the beam. The upper bolt hole in the beam can with advantage be vertically elongated to allow some moisture movement and rotation of the beam. When designing it is important to remember that this bolt cannot assist in taking up vertical forces.

14.4.2.13.4.2 Glued-in bolts

In a pinned connection with a glued-in bolt the fixture is completely hidden, which can be desirable for aesthetic, fire or other reasons (see figure 13.21). Nuts and washers can be recessed into the top of the beam and be covered with a timber plug, which provides effective fire protection.

The gluing operation is always carried out at the factory and the columns are thus delivered with projecting bolts. These must be protected from shocks and mechanical damage during transport and erection.

Glued-in bolts may not be used in service class 3 or in structures subject to dynamic loads or fatigue loads. Due to moisture movement in the beam the use of glued-in bolts should be limited to beams less than 500 mm deep.

Glued-in bolts provide insufficient lateral bracing. The top of the beam must therefore be specially braced. Glued-in bolts should be placed as close as possible to the inner side of the column so as not to restrict rotation of the beam. A suitable distance between the inner side of the column and the centre of the bolt is 4*d*if the joint only transfers horizontal forces or 2*d*if only vertical forces are transferred. If several glued-in bolts are required, they should for the same reason be placed beside one another.

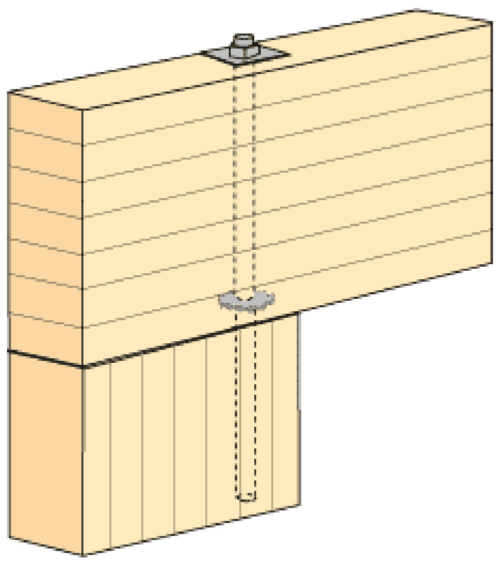


Figure 13.21 Pinned connection between beam and column with glued-in bolt. Principles.

DESIGN

The number of bolts, their dimensions and gluing-in lengths are determined with regard to horizontal forces and any uplift forces. Downward vertical forces are transferred to the column by contact pressure.

The design condition for the bolt glued into the column is:

$$\left[\frac{F_v}{R_{vd}}\right]^2 + \left[\frac{F_t}{R_{td}}\right]^2 \leq 1$$

Formula 13.74

where

$F_v = H_d/n$

$F_t = V_d/n$

H_d = design value of horizontal force on column

V_d = design value of vertical force (uplift)

n = number of glued-in bolts per column

R_{vd} = design bearing capacity of one bolt loaded in shear

R_{td} = design withdrawal loading capacity of one bolt

The characteristic values of R_{vd} and R_{td} are calculated using formulae 13.32 and 13.33.

To improve the load bearing capacity in shear, single-sided toothed plate connectors (e.g. Bulldog) can be fitted to the end surface of the column. This should be done at the same time as the gluing-in of the bolt in the factory. The bearing capacity of the glued-in bolt in shear can then be increased by the design extra capacity of the connector.

The size of the washer on the top of the beam is chosen so that the contact pressure between it and the top of the beam does not exceed the design value.

The design condition is:

$$F_t \le k_c f_{c90d} A_b$$

Formula 13.75

where

f_{c90d} = design value of the compression strength of the glulam beam perpendicular to the grain

A_b = area of connector with reduction for the hole

Since the compression is local, the value of f_{c90d} may be increased by a factor k_c. The factor can be determined either from 4.3 or with the expression:

$$k_c = \sqrt[4]{\frac{150}{D}} \le 1.8$$

Formula 13.76

where

D= size of the side of the connector in mm.

The thickness of the connector t_b is chosen so that the connector does not exceed the critical value of the yield tension limit of the material. The design condition for a square washer whose side is D is:

$$t_b \ge D \cdot \sqrt{\frac{D}{D-d}} \cdot \sqrt{\frac{f_{c90d}}{2 f_{yd}}}$$

Formula 13.77

where

f_{yd} = the design value of the yield tension limit of the washer.

The contact pressure between beam and column is checked in the same way as for fishplates of nailplate or flat steel as in formula 13.73.

14.4.3.13.4.3 Recessed beam

Recessed beams are often used with columns at an end wall to transfer the horizontal force of the column to the beam. For aesthetic and erection reasons, the recess in the column is of the same size as the width of the beam.

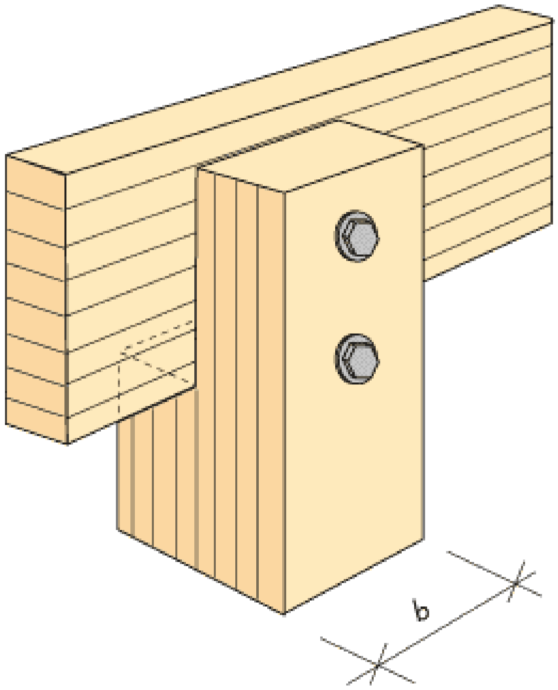


Figure 13.22 Pinned connection between beam and column with the beam recessed in the top of the column. Principles.

The transfer of forces between beam and column is usually made with the aid of bolts (see figure 13.22). Fixing with angles of steel plate (see figure 13.23) can be used when the horizontal forces are small.

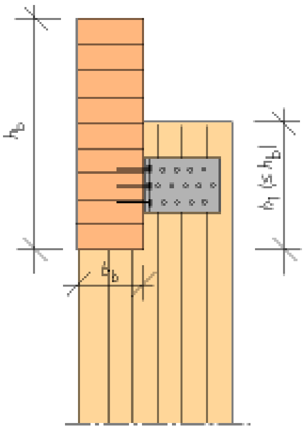


Figure 13.23

DESIGN

Bolts or angles of steel sheet are designed for uplift forces and for such horizontal forces as pull the column from the beam. Downward vertical forces and horizontal forces pushing the column towards the beam are transferred by contact pressure. The recess in the column is designed in accordance with the instructions in 4.4. The vertical reaction from the beam acts eccentrically on the glulam column, and the extra moment which this causes must be taken into account in the design of the column.

In the text below, bolts are assumed. It is further assumed that horizontal forces in the direction of the beam do not load the bolts.

The design condition for the fixing of the column in the beam is:

$$\left[\frac{F_v}{R_{vd}}\right]^2 + \left[\frac{F_t}{R_{td}}\right]^2 \leq 1$$

Formula 13.78

Where

$F_v = V_d/n$

$F_t = H^*/n$

V_d = design value of vertical force (uplift)

H^*_d = design value of the horizontal force pulling the column away from the beam

R_{vd} = design value of the load bearing capacity for a bolt in shear

R_{td} = design witdrawal capacity of the bolt

n = number of bolts in the connection

The size and thickness of the washer is checked as for glued-in bolts (see formulae 13.75-13.77).

The design condition regarding contact pressure between beam and column is:

$$V_d \leq f_{c90d} \cdot b \cdot b_b$$

Formula 13.79

$$H_d \leq f_{c90d} \cdot b \cdot h_b$$

Formula 13.80

where

V_d = design value of vertical force (downward)

H_d = design value of horizontal force pushing column against bean

b = width of cross-section of column

b_b = width of cross-section of beam

h_b = depth of cross-section of beam

The capacity with regard to downward vertical force, formula 13.79, can be increased by increasing the bearing area with the aid of a heavy flat steel with a thickness of t_p, placed between beam and column. If the projection of the steel is $4t_p$on each side of the column, $b$in formula 13.79 can be replaced by $b + 8t_p$. The carrying capacity of the flat steel need not be checked if the projection is maximum $4t_p$.

The design condition regarding recess in top of column is:

$$1{,}5\frac{H_d}{b(h-b_b)} \leq k_v \cdot f_{vd}$$

Formula 13.81

where

H_d= design value of the horizontal force pushing the column against the beam

f_{vd} = design value of the shear strength of the glulam column

h = depth of cross-section of column

k_v = reduction factor taking into account the effect of the recess on the carrying capacity and which is calculated as in 4.4.

The size of the recess is limited to 0,5h, but max 500 mm. It can be reinforced with e.g. plywood or glued-in screws (see 4.4).

14.4.4.13.4.4 Bracing of column top

In buildings of hall type the connection between column and beam is often made as shown in figure 12.21, i.e. with the beam placed on top of the column. As a rule the horizontal frame bracing, e.g. wind truss or roof slab, is at the level of the top of the beam and measures must be taken to prevent the top of the column from being pushed sideways. At the outer walls the wall itself can often be utilised, but inside columns must normally be braced, e.g. as in figure 13.24.

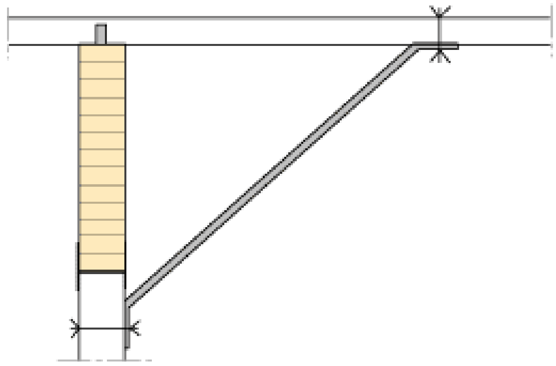


Figure 13.24 Side bracing of column top.

The bracing is designed for a horizontal force which depends both on the vertical support reaction and the moment in the beam:

$$H_d = H_{column} + H_{beam}$$

Formula 13.82

$$H_{column} = N_d / 100$$

Formula 13.83

$$H_{beam} = \frac{M_d}{h \cdot \beta_T}$$

Formula 13.84

where

N_d= compression force in column

M_d= maximum passive moment in beam

β_T= factor depending e.g. on geometry and side bracing of beam.

For a simply supported beam with evenly distributed load acting on the top of the beam, the factor $\beta_T$can be assessed with the aid of the following formulae:

β_T= 100 if the beam is unbraced between supports

β_T= 80 + 9h/b+ 0,6$(h/b)^2$if the top of the beam is braced for its entire length.

For interior columns H_{beam} from adjacent bays are added. If the beam is continuous over the inner support, the figure for two freely supported spans can be used an approximation on the safe side.

14.4.5.13.4.5 Bracing of continuous beam

The critical load for a continuous beam with the respect to lateral buckling can be increased by bracing the underside near the intermediate support, e.g. as in figure 13.25. See also 4.2. The bracing is designed for a horizontal force of:

$$H_d = \frac{M_d}{70h}$$

Formula 13.85

where

M_d is the moment at support.

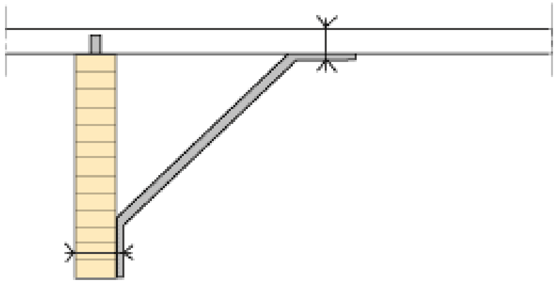


Figure 13.25 Bracing of continuous beam.

14.5.1.13.5.1 Fishplates of nailplate

Pinned ridge joints with fishplates of nailplate are simple and functional, suitable for both small and large forces (see figure 13.26).

Transfer of forces between fishplate and glulam beam takes place with the aid of ring shank nails. The fishplates should be placed as near the undersides of the beams as possible, so as not to restrict the free changes of angle. A suitable distance from the underside of the beam to the first row of nails is 10*d*.

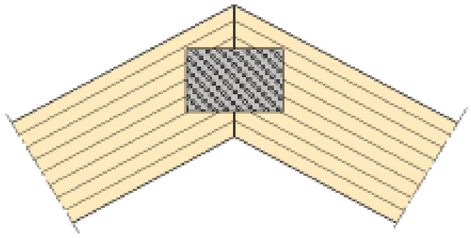


Figure 13.26 Pinned ridge joint with fishplates of nailplate. Principles.

Pre-drilled nailplates of galvanized steel sheet, 1,5 5,0 mm thick are available from stock in a large number of variations. Nailplates with free choice of hole patterns, thicknesses and finish can be ordered from manufacturers of perforated steel sheet. The cost is generally lowest when the holes are punched out, which means that the thickness must not exceed the hole diameter. The holes should be about 1 mm larger than the thickness of the nail.

DESIGN

In designing, the fishplate is regarded as a beam rigidly fixed at each end, loaded with a vertical force and a horizontal force, both acting at the centre of the nail group (see figure 13.27). The moment of fixture is taken into account when designing the fixture and the nailed joint.

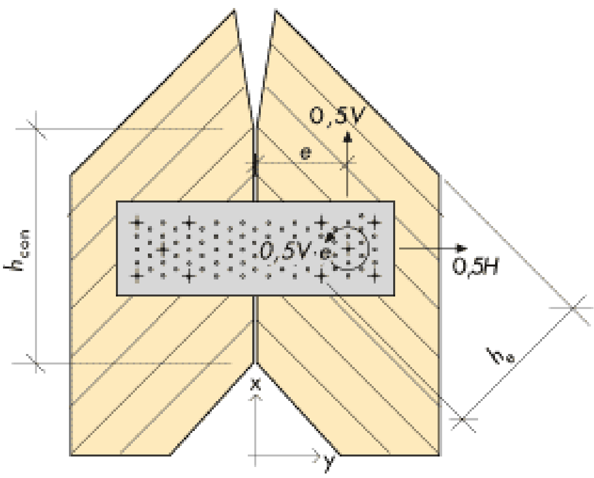


Figure 13.27 Pinned ridge joint with fishplates of nailplate. Symbols.

Two thirds of the compressive horizontal forces may be transferred by contact pressure between the beams. To reduce the risk of splitting, the ridge joint should be designed so that the contact pressure does not act nearer the top of the beam than 1/6 of the depth, e.g. through diagonal sawing (see figure 13.27) or a distance piece.

The design condition for the fixing of an individual fishplate in the beam is:

$$\sqrt{F_x^2 + F_y^2} \leq R_{vd}$$

Formula 13.86

where

R_{vd} = design carrying capacity of a nail loaded in shear

$$F_x = \frac{0{,}5V_d}{n} + \frac{0{,}5V_d \cdot e \cdot r_y}{I_p}$$

Formula 13.87

$$F_y = \frac{0{,}5H_d}{n} + \frac{0{,}5V_d \cdot e \cdot r_x}{I_p}$$

Formula 13.88

where

V_d = design value of vertical force from beam

H_d = design value of horizontal tension force from beam (if compression, insert $H_d/3$ instead of $H_d$in formula)

n = number of fasteners per beam and fishplate

e = distance in figure 13.27

r_y,r_x = distance in y and x direction between nail group centre of gravity and each nail

I_p = polar moment of inertia of nail group

The polar moment of inertia of the nail group is calculated using the formula:

$$I_p = \sum (r_x^2 + r_y^2)$$

Formula 13.89

The design condition for an individual fishplate is:

$$\sqrt{\sigma^2 + 3\tau^2} \leq f_{yd}$$

Formula 13.90

where

f_{yd} = design value of the yield tension limit of the fishplate

σ = axial stress at a certain point in the fishplate

τ = shear stress at the same point.

If $\sigma$and $\tau$are calculated in accordance with the Theory of Elasticity at the same time as a biaxial stress condition exists, f_{yd}in formula 13.90 can be replaced by 1,1.f_{yd}.

If the horizontal force is a tension force, the maximum stress $s_{t,max}$occurs in the tension edge of the fishplate. A value which is on the safe side can be calculated from the formula:

$$\sigma_{max} = \frac{0,5V_d \cdot e}{W} + \frac{0,5H_d}{A}$$

Formula 13.91

where

A = cross-sectional area of one fishplate

W = section modulus of the fishplate

If on the other hand the horizontal force is a compression force, it is transferred partly by contact pressure and the greatest axial stress $\sigma_{c,max}$can instead be calculated using the formula:

$$\sigma_{max} = \frac{0,5V_d \cdot e}{W} + \frac{1}{3} \cdot \frac{0,5H_d}{A}$$

Formula 13.92

where

$V_d$and $H_d$are inserted with their absolute values.

The shear stress τat the point of maximum bending stress is 0.

The maximum shear stress τ_{max}acts at the same height as the neutral axis in bending about the z axis and is calculated from the formula:

$$\tau_{max} = 1,5\frac{0,5V_d}{A}$$

Formula 13.93

The normal stress σat the same point on the cross-section is in tension:

$$\sigma = \frac{0,5H_d}{A}$$

Formula 13.94

and if the horizontal force is instead a compression force:

$$\sigma = \frac{1}{3} \cdot \frac{0,5H_d}{A}$$

Formula 13.95

The possible effect of bolt holes on the capacity of the fishplate is taken into account by replacing $f_{yd}$with f_{ud}in formula 13.90 and calculating the stresses $\sigma$and τbased on the net cross-section, i.e. with $A_{net}$and W_{net}. This means that the fishplate is allowed to plasticise round the holes.

The design condition with regard to the contact pressure between the beams is:

$$\frac{2}{3} \cdot H_d \le f_{cad} \cdot A_{contact}$$

Formula 13.96

where

H_d = the design value of the compressive horizontal force

f_{cad} = design value of the compression strength at an angle to the grain (see 4.3). Note that the strength shall be reduced if there is no distance piece.

A_{anl} = contact area, calculated with regard to possible diagonal cut in the top of the beam.

At steep roof slopes the contact pressure can lead to splitting of the beams.

The design condition with regard to delamination is

$$H \sin\alpha + V \cos\alpha \le \frac{2}{3} \cdot b \cdot h_e \cdot f_{vd}$$

Formula 13.97

where

α = slope of roof

b = width of beam

h_e = effective depth of beam (see figure 13.27)

f_{vd} = design value of shear strength

14.5.2.13.5.2 Fishplates of nailplate and shear connectors

When the shear force is so large that the moment of eccentricity V×e can cause difficulties, the sheaar force can instead be transferred with the aid of shear connectors. The shear connector can be designed in many ways, e.g. a piece of I-beam. In the text below, however, a cruciform as shown in figure 13.28 is assumed. The whole of the lateral force is assumed to be transferred by the cruciform and the nailplates are designed only for horizontal tension. However, they help distribute the shear force over the whole depth of the beam and counteract splitting at the cruciform. Horizontal compression is transferred by contact pressure between the end of the beam and the vertical legs of the cruciform.

DESIGN

Nailplates and nailed joints are designed according to the previous paragraph, with V_d= 0 inserted into the formulae.

The design condition with reference to contact pressure between the end of the beam and the cruciform is:

$$\frac{H_d}{b(2h)} + \frac{V_d(l+t_2)}{b \cdot h(h+t_1)} \le f_{cad}$$

Formula 13.98

where

V_d = design value of the shear force in the ridge

H_d = the associates horizontal (compression) force

b = length of cruciform

f_{cad} = design value of compressive strength of glulam beam at an angle to grain (see 4.3)

Symbols for the various cross-sectional sizes of the cruciform are in figure 13.28.

The design condition regarding the strength of the steel plate is:

$$\sqrt{\sigma^2 + 3\tau^2} \le f_{yd}$$

Formula 13.99

where

f_{yd} = design value of the yield tension limit of the steel

σ = normal stress at a certain point in the cruciform

τ = shear stress calculated at the same point

If $\sigma$and $\tau$are calculated in accordance with the Theory of Elasticity at the same time as a biaxial stress condition exists, f_{yd}in formula 13.99 can be replaced by 1,1f_{yd}.

The maximum bending and shear stresses can be calculated using the following formulae:

$$\sigma_{max} = \frac{V_d \cdot l/2}{b \cdot t_1^2/6}$$

Formula 13.100

$$\tau_{max} = 1{,}5\frac{V_d}{b \cdot t_1}$$

Formula 13.101

Note that maximum bending stress does not occur at the same point as maximum shear stress. The same design conditions as above (formula 13.99) exist for welds between horizontal and vertical legs.

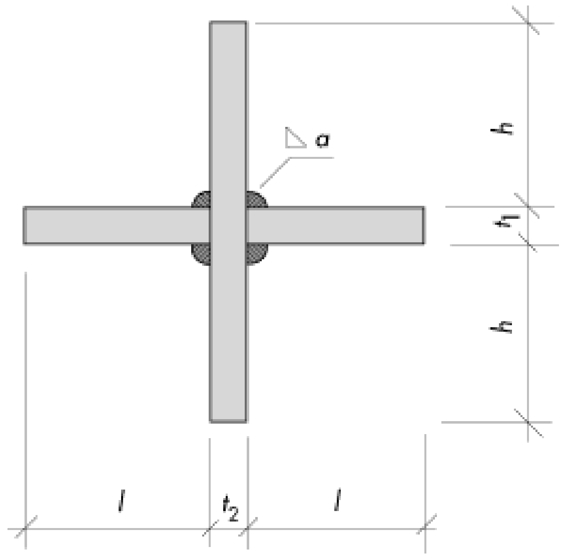


Figure 13.28 Cruciform connector of welded flat steel for transfer of shear forces.

If the cruciform connector is made with fillet welds, the bending and shear stresses in the welds can be calculated using the following formulae:

$$\sigma = \frac{V_d \cdot l/2}{b \cdot t_1 \cdot a\sqrt{2}} + \frac{V_d}{b \cdot a\sqrt{2}}$$

Formula 13.102

$$\tau = \frac{V_d \cdot l/2}{b \cdot t_1 \cdot a\sqrt{2}} - \frac{V_d}{b \cdot a\sqrt{2}}$$

Formula 13.103

If the degree of utilisation in formulae 13.60 and 13.58 is more than 70%, some regulations demand that the weld be checked by non-destructive testing, e.g. X-rays. Due to the risk of lamellar tearing of the baseplate it is often demanded that if it is utilised to over 50%, it must have guaranteed and verified properties in the direction of thickness, or be checked ultrasonically. It is therefore often practical to limit the utilisation in the welds in the formula to 50%.

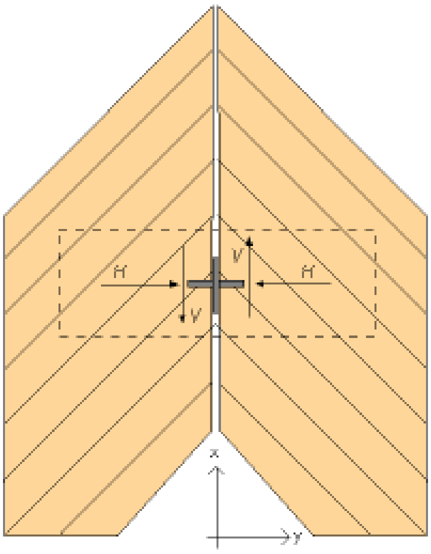


Figure 13.29 Cruciform connector of welded flat steel. Symbols.

14.5.3.13.5.3 Ridge fixture, type BMF

Ridge fixtures of pressed steel are available from stock and can be used for moderate shear forces. The fixture consists of two identical plates with pressed ribs and a short welded-on tube, plus one large and two smaller steel dowel pins (see figure 13.30). The fixture is made in four sizes with height and width from 175 x 65 to 350 x 90 mm.

Shear forces are transferred with the aid of ring shanked nails in the end surface of the timber together with the large middle pin, while the smaller pins take up any twisting moments. Horizontal compressive forces are transferred by contact between the pressed-out ribs. The fixture is combined with nailplates on the sides of the beams or, if desired for aesthetic reasons, on the tops of the beams.

The carrying capacity regarding shear forces varies according to the manufacturer's figures, from 6 kN for the smallest fixture to 10 kN for the largest. For details on the carrying capacity the manufacturer should be consulted.

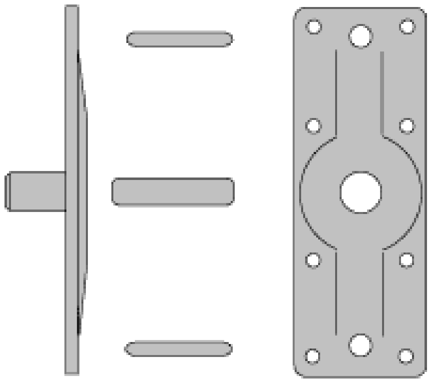


Figure 13.30 BMF ridge fixture.

14.6.1.13.6.1 Fishplates of nailplate

Pinned beam joints with fishplates of nailplate are a simple and functional joint which is suitable for moderate forces (see figure 13.31).

The fishplates should be placed centrally about the centre line of the glulam beam. Transfer of forces takes place with the aid of ring shanked nails.

Pre-drilled galvanized steel nailplates 1,5 5,0 mm thick are stocked in a large number of types. A cheap alternative for moderate forces. Nailplates with a wide choice of hole patterns and thicknesses can also be ordered from manufacturers of perforated steel plate (see 13.2.1).

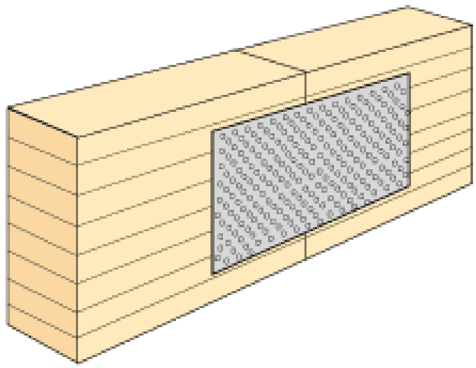


Figure 13.31. Pinned beam joint with fishplates of nailplate. Principles.

DESIGN

In designing, the fishplate is regarded as a beam rigidly fixed at both ends, and loaded with a vertical force and a horizontal force. The forces are assumed to act at the centre of the nail group. Two thirds of the compressive horizontal forces are transferred by contact pressure between the beams. The moment of fixture is taken into account when designing fixtures and nailed joints.

The fixing of individual fishplates in the beam, the fishplates themselves and the contact pressure between the beams are checked as for ridge joints with fishplates of nailplate (see 13.5.1).

14.6.2.13.6.2 Welded Gerber fixtures

Pinned beam joints with Gerber fixtures are recommended if large shear forces are to be transferred and if the force always has the same direction. Small shear forces in the opposite direction can be transferred through the screw/bolt joints in the vertical side plates. If, additionally, the Gerber fixture must transfer tension it is complemented by welded-on steel fishplates. Welded Gerber fixtures are usually designed as in figure 13.32, but other types exist, e.g. with a slotted-in central plate instead of two external ones. This type has aesthetic and fire advantages.

So as not to resist angular movement in the beams, the side screws are placed as near as possible to the top and bottom plates. A suitable edge distance is 2*d*if the screws only transfer horizontal forces, and 4*d*if they also transfer vertical forces.

For small shear forces, ready-made Gerber fixtures can be used with advantage (see 13.6.3).

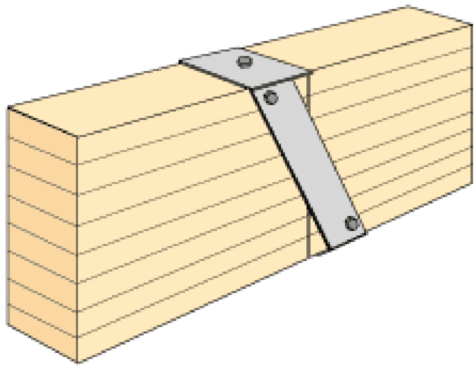


Figure 13.32 Pinned beam joint with Gerber fixture. Principle.

DESIGN

When designing it is assumed that shear force is transferred by contact with the top and bottom plates of the Gerber fixture.

The design condition regarding contact pressure between the Gerber fixture and the beam is:

$$V_d \leq f_{c,90,d} \cdot \beta \cdot b \cdot L$$

Formula 13.104

where

V_d= design shear force, calculated for the whole beam

$f_{c,90,d}$= design value of the compression strength of the beam perpendicular to the grain

b = width of the beam

L = length of the top or bottom plate, figure 13.33.

The contact pressure is assumed to act only against a part, βxb, of the Gerber fixture's length:

$$\beta \cdot b = 2t_f \sqrt{\frac{f_{yd}}{2 f_{c90d}}}$$

Formula 13.105

where

f_{yd} = design value of the yield tension limit of the top or bottom plate

t_f = thickness of the top or bottom plate.

The eccentricity of the shear forces causes a moment, which in the underside is taken up by

contact pressure between the back plate and the end of the beam and in the top by the screws/bolts in the side plates. If necessary the screw/bolts in the side plates can be complemented by screws through the top plate.

The design condition regarding the screws/bolts in the side plates (assuming that there are no screws in the top plate) is:

$$F \leq R_{vd}$$

Formula 13.106

where

R_{vd} = the design load bearing capacity of the screw/bolt in shear.

$F$is calculated from the formula:

$$2F = f_{cd}\, b\, x$$

Formel 13.107

where

f_{cd} = design value of the compression strength of the glulam beam in the direction of grain

Distance $x$is determined from the Equilibrium Condition with respect to the moment and is:

$$x = h_1\left(1 - \sqrt{1 - \frac{2 V_d L}{f_{cd} \cdot b \cdot h_1^2}}\right)$$

Formula 13.108

where distance $h_1$is shown in figure 13.33.

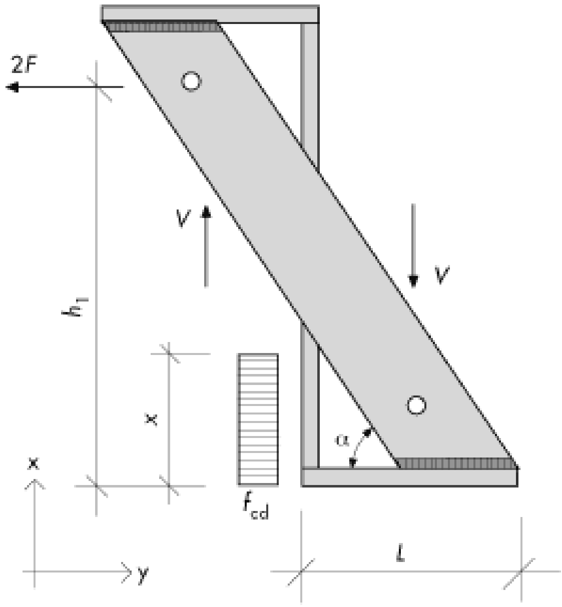


Figure 13.33. Welded Gerber fixture. Symbols.

The design condition for the side plates of the Gerber fixture is:

$$\sqrt{\sigma^2 + 3\tau^2} \leq f_{yd}$$

Formula 13.109

where

f_{yd} = design value of the tensile yield tension limit of the side plates

σ = normal stress at a certain point in the side plate

τ = shear stress calculated at the same point.

If $\sigma$and $\tau$are calculated in accordance with the Theory of Elasticity at the same time as a biaxial stress condition exists, f_{yd}in formula 13.109 can be replaced by 1,1.f_{yd}.

The maximum normal tension stress $\sigma_{max}$is calculated from the formula:

$$\sigma_{max} = \frac{F \cdot 0,5x}{W} + \frac{0,5V_d \sin\alpha + F\cos\alpha}{A}$$

Formula 13.110

where

A = cross-sectional area of the side plate

W = section modulus of one side plate.

Other symbols are given in figure 13.33.

If the distance between screw/bolt and bottom plate is greater than 0,5*x*, this distance is replaced in formula 13.110 by the edge distance of the screw/bolt.

The shear stress τ= 0 at the edge of the plate.

The maximum shear stress τ_{max}occur at the same level as the neutral axis in bending about the z-axis and can be calculated from the formula:

$$\tau_{max} = 1{,}5 \cdot \frac{F \sin \alpha - 0{,}5 V_d \cos \alpha}{A}$$

Formula 13.111

The axial stress σat the same point is calculated from:

$$\sigma = \frac{0{,}5 V_d \sin \alpha + F \cos \alpha}{A}$$

Formula 13.112

The effect of screw/bolt holes on the bearing capacity can be taken into account in formula 13.109 by replacing $f_{yd}$with $f_{ud}$and calculating stresses $\sigma$and τstarting from the net figures for the cross-section $A_{net}$and W_{net}. This means that the fishplate is allowed to plasticise round the screw holes.

The design condition for the fillet welds between the side plates of the Gerber fixture and its top and bottom plates is

$$\left(\frac{F}{F_{R//}}\right)^2 + \left(\frac{0{,}5V_d}{F_{R\perp}}\right)^2 \leq 1$$

Formula 13.113

where $F_{R//}$ = design load bearing capacity of the weld in the longitudinal direction

$F_{R\text{^}}$ = design load bearing capacity of the weld in the transverse direction

$F_{R//}$and $F_{R\text{^}}$are calculated using the formulae:

$$F_{R||} = 0{,}6a \cdot l_s \cdot f_{wd}$$

Formula 13.114

$$F_{R\perp} = \frac{a \cdot l_s \cdot f_{wd}}{\sqrt{2}}$$

Formula 13.115

where

f_{wd} = design value of the strength of the weld

l_s = is the length of the welded joint

a = the size of the weld; to be chosen as at least 3 mm.

It is in many cases advantageous to limit the utilisation in formula 13.109 to 70% and to design the weld so that is as strong as the side plate (compare comments under 13.2.1).

14.6.3.13.6.3 Gerber fixture type BMF-W

For moderate shear forces, factory-made fixtures of galvanised steel plate can be used with advantage. These are available from several manufacturers, e.g. BMF.

Factory-made Gerber fixtures can be either whole or in parts. The whole fixtures only fit certain beam dimensions, while those in parts are as a rule independent of the cross-sectional sizes of the beams. Forces are transferred mainly with the aid of ring shanked nails. For detailed information on carrying capacity etc, see the manufacturers' product catalogues. When using two angle fixtures of the type shown in figure 13.34, the risk of splitting must be taken into account.

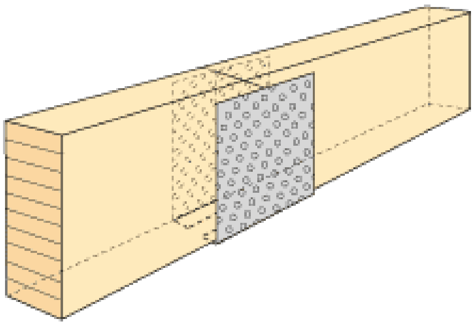


Figure 13.34 Gerber fixture type BMF-W.

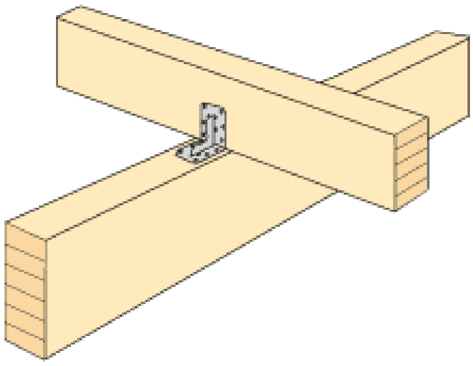


Figure 13.35 Fixing purlins with stiffened angle fixture of cold-formed steel plate.

14.7.1.13.7.1 Fixing purlins

The fixing of purlins in the primary beam is usually carried out using factory-made fixtures of cold-formed galvanized steel sheet. The fixtures can be designed as angle fixtures (figure 13.35), with or without a stiffening groove, or as hangers as in figure 13.36. Transfer of forces is mainly by contact pressure, assisted by ring shanked nails. Sheet steel fixtures are made by a number of firms, e.g. BMF. For detailed information on carrying capacity etc., see the companies' product catalogues. For side bracing of purlins in pitched roofs, see 12.3.

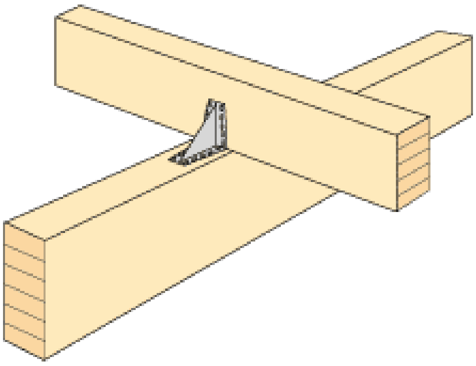


Figure 13.36 Fixing purlins with angle of cold-formed steel.

14.7.2.13.7.2 Welded beam hanger

Connection of a secondary beam with a hanger is a simple and functional solution, especially when the top sides of the beams are to be at the same level (see figure 13.37).

For small forces there are hangers of cold-formed galvanized steel sheet made and marketed by a number of firms, e.g. BMF. The selection of hangers with sizes adapted to glulam sizes is however very limited. Manufacturers can however offer tailor-made hangers according to the customer's specification. Transfer of forces between the secondary beam and the hanger is mainly by contact pressure, while that from hanger to primary beam is by nails, bolts or wood screws. The carrying capacity can be increased considerably with the aid of connectors between hanger and primary beam. The connectors should in this case be fitted in the factory.

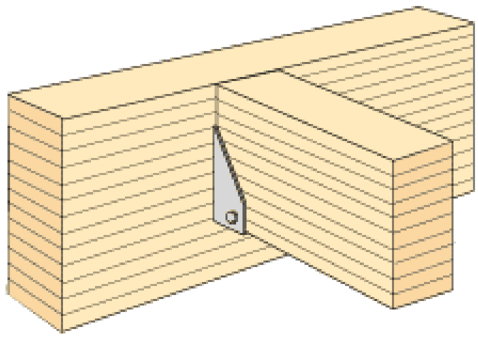


Figure 13.37. Pinned connection between secondary and primary beam with welded hanger. Principle.

A secondary beam on one side only of the primary beam causes a torsional moment in the primary beam. This moment must be taken into account in the design. Note that the risk of

splitting increases the further down the side of the primary beam the secondary beam is fixed.

Bracing against lateral buckling of the secondary beam can be improved by fixing its top laterally at the supports, e.g. with the aid of angles.

Hangers can be designed in different ways, depending on the aesthetic and other requirements which are made. If the back plate does not extend outside the side plates the fixing to the primary beam is shielded from direct fire action. On the other hand, a projecting back plate provides space for several rows of fixings and the load carrying capacity can be increased. Side plates on either side of the secondary beam can further be replaced by a single routed-in plate which is invisible and protected from direct fire attack. This does not affect the load bearing capacity, but may increase the price somewhat.

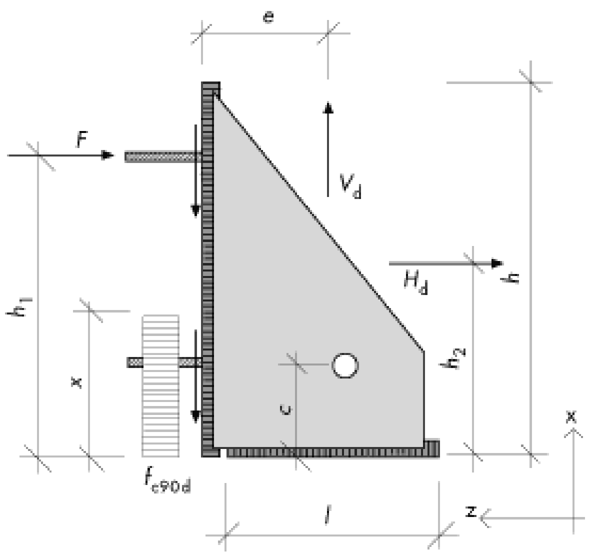


Figure 13.38 Welded beam hanger. Symbols.

DESIGN

In the design it is assumed that downward vertical forces are transferred to the hanger by contact pressure against the bottom plate. Uplift and horizontal forces pulling the secondary beam away from the primary beam are transferred to the side plate with the aid of screws/bolts. Horizontal forces pushing the secondary beam against the primary beam are transferred by contact pressure against the back plate.

In the following design instructions for the hanger in figure 13.38 it is assumed that the uplift force is small in relation to the downward vertical force.

The design condition for the fixing of the secondary beam in one of the side plates of the hanger is:

$$\sqrt{F_x^2 + F_z^2} \leq R_{vd}$$

Formula 13.116

where

$F_x = 0{,}5V_d$

$F_z = 0{,}5H_d$

V_d= design value of the vertical uplift reaction from the secondary beam

H_d = design value of horizontal force pulling the secondary beam away from the primary

R_{vd} = design value of load bearing capacity of a laterally loaded screw/bolt

The formula applies to the case where there is one screw/bolt in each side plate.

The design condition regarding contact pressure between the secondary beam and the bottom plate of the hanger is:

$$V_d \leq f_{c90d} \cdot \beta \cdot b \cdot l$$

Formula 13.117

where

V_d = design value of (downward) vertical support reaction from the secondary beam

f_{c90d} = design value of the compression strength of the secondary beam perpendicular to the grain

l = length of the bottom plate

b = width of the beam

β= factor taking into account that compression at support is concentrated to the edges of the hanger.

The effective width βx $b$is calculated using the formula:

$$\beta \cdot b = 2t\sqrt{\frac{f_{yd}}{2f_{c,90,d}}}$$

Formula 13.118

where

f_{yd} = design value of the yield tension limit of the bottom plate

t = thickness of bottom plate.

The side plates of the hanger will not be critical for the load bearing capacity if the thickness is at least half of that of the bottom plate.

The design condition for the back plate of the hanger is:

$$\sigma \leq f_{yd}$$

Formula 13.119

where

f_{yd} = design value of the yield tension limit of the back plate

σ = axial stress in back plate.

The axial stress σcan be calculated from the formula:

$$\sigma \approx \frac{3F}{2t^2}$$

Formula 13.120

where

F = design value of withdrawal force in the upper back screw/bolt

t = thickness of back plate.

With two horizontal rows of screws/bolts as in figure 13.38, *F* can be calculated using the formula:

$$F = f_{c90d} \cdot b \cdot x - H_d$$

Formula 13.121

where

f_{c90d} = design value of the compression strength of the beam perpendicular to the grain

H_d = design value of the horizontal force

x = height of compression stress block, see figure 13.38 and formula 13.122.

The horizontal force is inserted as positive if the force pushes the secondary beam against the primary, and negative in the opposite direction. Distance *x* is calculated from the condition of moment equilibrium and can be written:

$$x = h_1\left(1-\sqrt{1-2\frac{V_d \cdot e + H_d\left(h_1 - h_2\right)}{f_{c90d} \cdot b \cdot h_1^2}}\right)$$

Formula 13.122

where

V_d = design value of (downward) vertical support reaction

$e \approx 2t + l/2$

$h_2 = h/2$ if the horizontal force is compressive

$h_2 = c$ if the horizontal force is tensional.

The condition for formulae 13.121 and 13.122 to apply is that the vertical force is large in relation to the horizontal force. In other cases the contact pressure is not concentrated to the lower edge, which has been assumed when deriving the formulae.

The design condition for the upper screw/bolt in the back plate (see figure 13.38) of the hanger is:

$$\left[\frac{F_v}{R_{vd}}\right]^2 + \left[\frac{F_t}{R_{td}}\right]^2 \leq 1$$

Formula 13.123

where

$F_v = 0{,}5V_d$

$F_t = F$

R_{vd} = design value of the load bearing capacity of the screw/bolt in shear

R_{td} = design value of the load bearing capacity of the screw/bolt under axial tension

The size of the washer on the back of the primary beam is chosen so that the contact pressure between beam and washer does not exceed the compressive strength across the grain. The design condition is:

$$F \leq \kappa_c f_{c90d} A_b$$

Formula 13.124

where

A_b = area of washer with reduction for the hole

k_c = factor which takes into account the fact that load bearing capacity increases under local pressure.

Factor $k_c$can be determined using the text in 4.3, or from the formula:

$$k_c = \sqrt[4]{\frac{150}{D}} \leq 1{,}8$$

Formula 13.125

where

D= diameter/side dimension of the washer in mm.

The thickness of the washer is determined so that the bending stress in the washer does not exceed the design strength value. The design condition for a square connector is:

$$t_b = D\sqrt{\frac{D}{D-d}} \cdot \sqrt{\frac{f_{c90d}}{2 f_{yd}}}$$

Formula 13.126

where

f_{yd} = design value of the yield tension limit of the washer

t_b = thickness of the washer

The design condition for the fillet welds between the side and bottom plates of the hanger is:

$$\left(\frac{0{,}5 H_d}{F_{R//}}\right)^2 + \left(\frac{0{,}5 V_d}{F_{R\perp}}\right)^2 \leq 1$$

Formula 13.127

where

$F_{R//}$ = design load bearing capacity of the weld longitudinally

F_F⊥/sub> = design load bearing capacity of the weld in the transverse direction

$F_{R//}$and F_F⊥/sub> can be calculated using the formulae:

$$F_{R||} = 0{,}6 a \cdot l_s \cdot f_{wd}$$

Formula 13.128

$$F_{R\perp} = \frac{a \cdot l_s \cdot f_{wd}}{\sqrt{2}}$$

Formula 13.129

where

f_{wd} = design value of the strength of the weld

l_s = length of the weld

a = size of the weld; minimum 3 mm.

It can be advantageous to limit the utilisation of the welded joint to about 70%, since some regulations require that the weld must be checked using non-destructive testing (e.g. X-Ray) if the utilisation is higher.

The design condition for fillet welds between the side and back plates of the hanger is:

$$\sqrt{\sigma_\perp^2 + 3\tau_\parallel^2} \leq f_{wd}$$

Formula 13.130

where

f_{wd} = design value of the strength of the weld

σ = longitudinal stress in the weld

$\tau_{//}$ = shear stress in the weld.

Normal and shear stresses in the weld are calculated using the formulae:

$$\sigma_\perp = \frac{0{,}5V_d \cdot e - 0{,}5H_d \cdot h_2}{a \cdot h^2 / 6} \cdot \sqrt{2} \pm \frac{0{,}5H_d}{a \cdot h} \cdot \sqrt{2}$$

Formula 13.131

$$\tau_{//} = \frac{0{,}5V_d}{a \cdot h}$$

Formula 13.132

The size, *a*, of the weld is chosen as at least 3 mm. The utilisation of the welded joint should be restricted to 70% to avoid the requirement on welding control, e.g. X-ray.

If the hanger is designed with a projecting back plate, the degree of utilisation should be further limited, due to the risk of lamellar tearing of the plate (see comments in 13.2.1).

14.8.1.13.8.1 Steel ties

Steel ties are suitable both for small and large tension forces (figure 13.39)

The simplest fixing is that with one tension rod on each side of the beam. For moderate forces this can be replaced by a single tie drawn through a central hole in the beam. If the tension forces are large, the two ties on either side can be complemented by a third, centrally placed. The steel plate against the end of the beam   the anchor plate   should have nail holes to simplify erection.

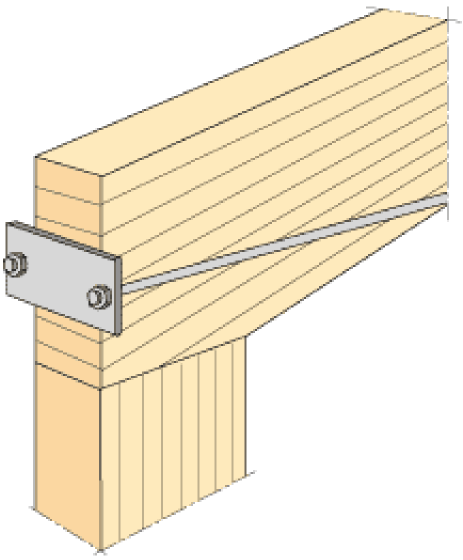


Figure 13.39 Fixing of steel tie. Principle.

DESIGN

In designing the horizontal tension forces are assumed to be transferred by contact pressure between the anchor plate and the end of the beam.

The design condition regarding contact pressure between the anchor plate and the beam end is:

$$H_d \leq f_{c\alpha d} \cdot A_{ef}$$

Formulal 13.133

where

H_d = total design tension force

f_fαd/sub> = design value of the compressive strength of the beam at an angle to the grain

A_{ef} = effective part of contact area between anchor plate and end timber (see figure 13.40).

The compressive strength of the glulam at an angle to the grain is determined as in 4.3.

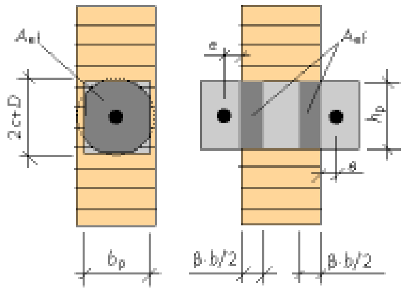


Figure 13.40. Effective contact area with one or two tie rods. Symbols.

With double ties the contact pressure is concentrated to the edges of the beam. The effective part of the contact area between anchor plate and beam end is calculated using the formulae:

$$A_{ef} = \beta b \cdot h_p$$

Formula 13.134

where

$$\beta = \frac{2t_p}{b}\sqrt{\frac{f_{yd}}{2\,f_{c90d}}} \leq 1$$

Formula 13.135

where

f_{yd} = design value of the yield tension limit of the anchor plate

t_p = thickness of anchor plate

Other symbols are given in figure 13.40.

For a single, centrally placed tie, the effective part is limited by a circle whose diameter is 2*c*+ *D*, see figure 13.40. *D*is the diameter of the nut or of a possible extra washer and *c* is given by the formula:

$$c = t_p \sqrt[3]{\frac{E_k}{E_{\alpha k}}}$$

Formula 13.136

where

E_k = characteristic value of the Modulus of Elasticity of the anchor plate (21000 N/mm2)

E_Eαk/sub> = characteristic value of the Modulus of Elasticity of the glulam beam in compression diagonal to the grain

E_Eαk/sub> can be calculated from the formula:

$$E_{\alpha k} = E_{0k} - \left(E_{0k} - E_{90k}\right)\sin\alpha$$

Formula 13.137

where

E_{0k} and E_{90k} are characteristic values of the Modulus of Elasticity of the glulam beam in compression respectively parallel to and perpendicular to the grain.

The design condition regarding bending in the anchor plate is:

$$\sigma \le f_{yd}$$

Formulal 13.138

The maximum bending stress in the plate is calculated using the formulae:

$$\sigma = \frac{3H_d\left(e+\beta \cdot b/4\right)}{2h_p \cdot t_p^2}$$

Formula 13.139. Two tie rods.

$$\sigma = \frac{3H_d}{2t_p^2}$$

Formula 13.140. One tie rod.

The design condition regarding pull-over of the nut through the anchor plate is:

$$\tau = \frac{H_d}{\pi \cdot D \cdot t_p} \le 0,6 f_{yd}$$

Formula 13.141

14.8.2.13.8.2 Glulam ties

Glulam ties are primarily suitable for small tension forces (figure 13.41).

Fixing of the tie in the beam can be carried out with flat steel which is either taken round the end of the beam or ends a little way in from the beam end. For small forces the flat steel can be replaced by nailplates.

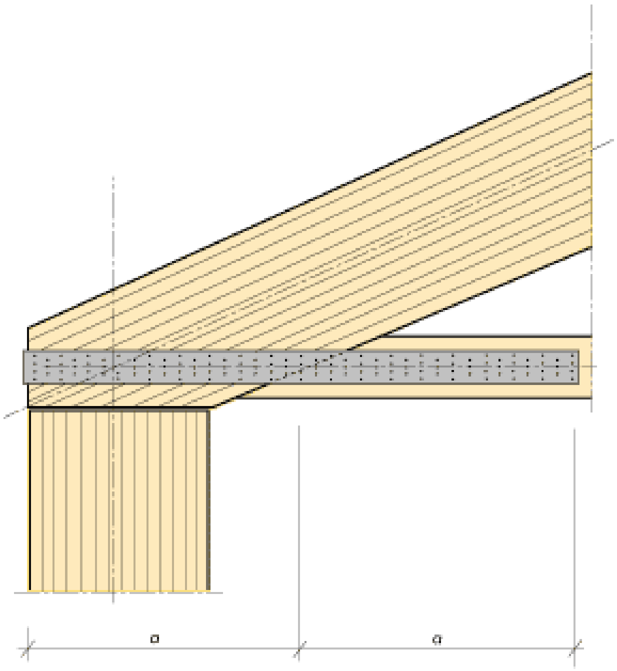


Figure 13.41 Fixing of glulam ties. Principles.

DESIGN

In the design of the tie fixing in figure 13.41 it is assumed that horizontal tension forces are transferred from the tie to the side plates by nailed joints. The side plates are fillet welded to the anchor plate, which transfers the force to beam or arch end by contact pressure.

The design condition for the fixing of side plates to the tie is:

$$F_y \leq R_{vd}$$

Formula 13.142

'where

$F_v = H_d/2n$

H_d = design value of tension force for the whole tie

R_{vd} = design value of load bearing capacity for a nail loaded in shear

n = number of nails per side plate.

The design condition for the side plates is:

$$\sigma \leq f_{yd}$$

Formula 13.143

where

f_{yd} = design value of the yield tension limit of the side plate

σ = axial stress in the plate.

The axial stress is calculated from the formula:

$$\sigma = \frac{H_d}{2A}$$

Formula 13.144

where

$A$is the cross-sectional area of a side plate.

The influence of the nail holes on the carrying capacity of the side plate can be taken into account in formula 13.143 by replacing the yield tension limit $f_{yd}$with the ultimate strength $f_{ud}$and calculating the normal stress σbased on the net area A_{net}. This means that the side plate is allowed to plasticise around the holes.

The fillet welds between the side plates and the anchor plate must be checked against the design condition (13.130). Here $\tau_{//}$is given the value 0 and σ⊥/sub> is calculated using the formula:

$$\sigma_{\perp} = \frac{H_d}{2a \cdot h_p}\sqrt{2}$$

Formula 13.145

where

a = the size of the weld (minimum 3 mm)

h_p = height of the side plates.

The utilisation of the welded joint should be restricted to 70% to avoid the requirement on welding control, e.g. X-Ray.

The contact pressure between the anchor plate and the beam end is checked with the design condition (13.133).

Here again the contact pressure is concentrated to the edges of the beam and the effective part of the contact area can be calculated with the formulae 13.134 and 13.145.

14.9.1.13.9.1 Fishplates of nailplate or flat steel

Anchorage with external fishplates of nailplate or flat steel (see figure 13.42) is a simple and functional solution for arches with spans of up to about 25 m. Its use is limited to indoor structures which are not regularly subject to the action of water, e.g. cleaning.

The fishplates can either be cast into the concrete foundation or be welded to a cast-in fixing plate. In the first case, the ends of the beam must have some kind of damp proofing, e.g. an oil tempered hardboard sheet glued and nailed to the end of the arch.

Design can be according to the instructions for hinged column base in 13.2.1.

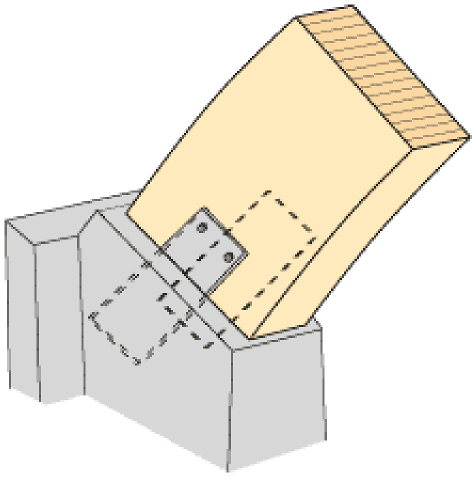


Figure 13.42 Springing point of arch with flat steel and screws/bolts. Principles.

14.9.2.13.9.2 Welded support fixture with hinged connection

A welded support fixture with a true hinge function is as a rule necessary for larger spans. Designed as in figure 13.43, the transmitted moment is limited and need not be taken into account in the design of the arch. The design permits use out of doors, i.e. in service class 3.

Axial and shear forces from the arch are transferred by contact pressure to the steel shoe and thence via the hinge down into the concrete foundation (see figure 13.44). The moment, Vx h_1, which arises when the shear force is transferred from the centre of gravity of the contact area to the hinge is taken up by screws/bolts and/or steel dowels. The moment Vx $h_2$which is necessary to transfer the shear force down into the concrete foundation is taken up by contact pressure between the hinge plate and the concrete. The shear force itself is taken up by anchor bolts cast into the foundation. Alternatively, the lower hinge plate can be designed with downwards projections which transfer the shear force to the concrete foundation.

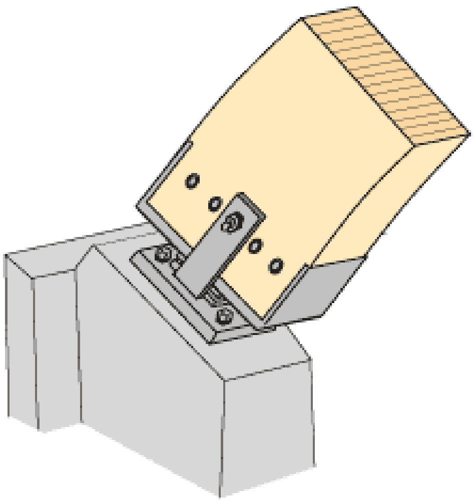


Figure 13.43 Welded support fixture with hinged connection. Principles.

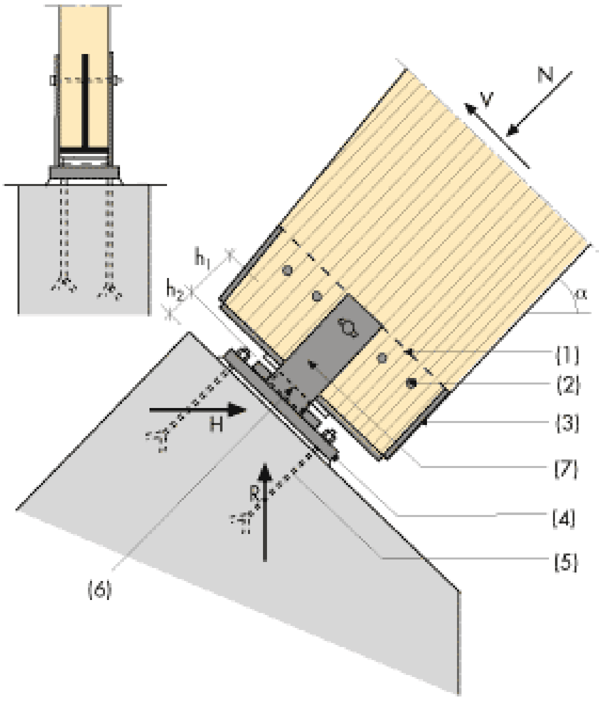


Figure 13.44 Symbols. (1) ^ section, welded or cut I beam. (2) Steel dowels and/or screw/bolt. (3) End plate. (4) Lower hinge plate with steel projections. (5) Anchor bolts. (6) Upper hinge plate with steel projections. (7) Side plates (NB: elongated screw/bolt holes).

DESIGN

Cross-sectional forces at the springing point are calculated with the following formulae:

$$N = H\cos\alpha + R\sin\alpha$$

Formula 13.146

$$V = H\sin\alpha - R\cos\alpha$$

Formula 13.147

The design condition with regard to the shear strength of the glulam is:

$$V_d \leq \frac{2}{3} b \cdot h \cdot f_{vd}$$

Formula 13.148

where

V_d = design value of the shear force in the end of the arch

b = cross-sectional width of the arch

h = cross-sectional depth of the arch

f_{vd} = design value of shear strength

The design condition with regard to contact pressure between glulam and the end plates of the steel shoe is:

$$V_d \leq b \cdot c_1 \cdot f_{c90d}$$

Formula 13.149

where

c_1 = length of the end plate (see figure 13.45)

f_{c90d}= design value of the compressive strength across the grain, possibly increased with regard to local compression as in 4.3.

The design condition with regard to contact pressure against the upper hinge plate is:

$$N_d = (b-t)c_2 \cdot f_{c0d}$$

Formula 13.150

where

c_2 = length of the hinge plate

t = thickness of the web of the ^ section

$f_{c,o,d}$ = critical value of the compressive strength parallel to the grain.

The transferring moment M_1= V+ h_1, is taken up by screws/bolts and/or dowels (see figure 13.45).

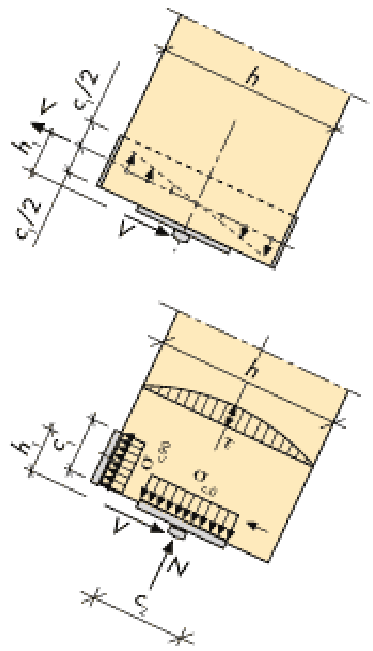


Figure 13.45. Transfer of forces between fixture and glulam. Calculation models.

Design conditions with regard to carrying capacity of an individual screw, bolt or dowel are:

$$\sqrt{F_x^2 + F_y^2} \leq R_{vd}$$

Formula 13.151

where

R_{vd} = design load bearing capacity per shear plane for a fastener in shear

$$F_x = \frac{0{,}5V_d \cdot h_1 \cdot r_y}{I_p}$$

Formula 13.152

$$F_y = \frac{0{,}5V_d \cdot h_1 \cdot r_x}{I_p}$$

Formula 13.153

V_d = design value of shear force

h_1 = distance as in figure 13.45

r_y, r_x = distance in y and x directions between screw/bolt and/or dowel groups' centre of gravity and the individual fastener

I_p = polar moment of inertia of the group.

The polar moment of inertia of the group of fasteners is calculated using the formula:

$$I_p = \sum_1^n (r_x^2 + r_y^2)$$

Formula 13.154

where

n= number of fasteners.

The design condition with regard to bearing strength between screw/dowel/bolt and the web of the ⊥- section is:

$$2\sqrt{F_x^2 + F_y^2} \leq d \cdot t \cdot f_{bd}$$

Formula 13.155

where

d = diameter of the screw/bolt/dowel

f_{bd} = design value of the bearing strength of steel.

The side plates of the fixture have a mainly guiding function and can be designed with practical considerations as the starting point. Thus the screw/bolt holes should be elongated so as not to restrict changes of angle in the arch.

The design condition regarding contact pressure between the lower hinge plate and the concrete foundation is:

$$0 \le \frac{N_d}{b \cdot c_3} \pm \frac{V_d \cdot h_2}{b \cdot c_3^2 / 6} \le f_{ccd}$$

Formula 13.156

where

c_3 = length of lower hinge plate

h_2 = distance as in figure 13.46

f_{ccd} = design value of the strength of concrete under local pressure.

If the concrete stress in formula 13.156 is negative, the tension must be taken up by the anchor bolts instead (see below).

The anchor bolts can be designed as in 13.2.4 glued-in bolts. Normally the bolts are only loaded in shear. The resultant of the contact pressure between concrete and bolt can be assumed to lie at a depth of *d*(bolt diameter) under the surface of the concrete.

Steel parts and welds are designed in accordance with Eurocode 3 (EN 1993).

The surrounding concrete structure is designed in accordance with Eurocode 2 (EN 1992).

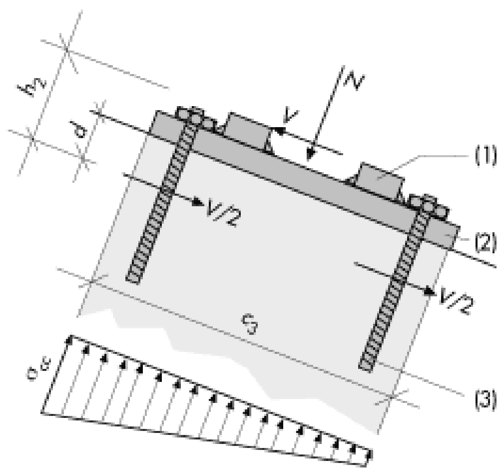


Figure 13.46 Transfer of forces between fixture and concrete. Calculation model. (1) Steel projection (2) Hinge plate (3) Anchor bolt

15.4.1.14.4.1 Design by classification

When designing by classification, the structure is built up of fire-classified components. The required class in each case is given in current regulations and depends inter alia, as previously mentioned, on the use of the building, its height, the fire load density and the importance of the part of the building in question to the total load bearing capacity of the structure.

Classification is based on fire testing or calculations, or a combination of both. In design the starting point is a fire exposure corresponding to the standard fire curve. This is stipulated in an international standard and is in universal use in fire testing.

Most regulations provide only incomplete instructions on how the fire class of timber structures shall be calculated. Two methods of calculating load bearing capacity as described in Eurocode 5 parts 1-2 are given below. The following applies to them both:

- Shear and compression at right angles to the grain need not be considered. Notched beams shall be designed so that the residual cross-section at the notch is at least 60% of the cross-section required at normal temperature.

- It must be remembered that the slenderness of the construction in regard to buckling increases in fire and that bracing members can disappear during the fire. Such bracing members can be assumed to function during the fire if their residual section is at least 60% of the section required at normal temperature.

SIMPLIFIED METHOD

According to the first, highly simplified, method an effective residual section is calculated by reducing the cross-section by an effective charred depth on those sides exposed to fire (see figure 14.1).

The effective charred depth is the sum of the actual charred depth and a zone with greatly reduced strength and stiffness. On the inner side of this zone it is assumed that strength characteristics are unaffected by the fire. The residual section is assumed to be rectangular, without the rounded corners which in practice always arise.

The effective charred depth is calculated using the formula:

$$d_{ef} = d_{char} + d_0$$

Formula 14.1

where

$d_{char} = \beta_0 \cdot t$= real charred depth in mm

β_0 = 0,7 mm/minute for glulam of spruce or pine

t = duration of fire in minutes

d_0 = zone with greatly reduced strength = 0,35$t$mm, max 7 mm.

The expression assumes that the charred depth:

$$\beta_0 t \leq \min \begin{cases} b/4 \\ h/4 \end{cases}$$

The size of the residual section is then calculated. Finally the load carrying capacity is calculated in the ultimate limit state, without consideration of fire exposure (see 3.2.5).

The values of the reduced, cross-sectional properties of the reduced, effective cross-section are then calculated, and finally the load-bearing capacity in the same way as when designing in the ultimate limit state, without regard to the effect of fire (see 3.2.5).

The partial coefficients in formula 3.3 are normally taken as (gn=) gm = 1,0 when designing for fire. The modification factor for service class and duration of load is also usually taken as 1,0. This means in practice that unreduced characteristic short-term values of stiffness and strength are used.

REFINED METHOD

According to the other, more refined method, the true residual section is used, without a weakened zone as above. This is compensated by reducing the stiffness and strength with regard to the effects of temperature.

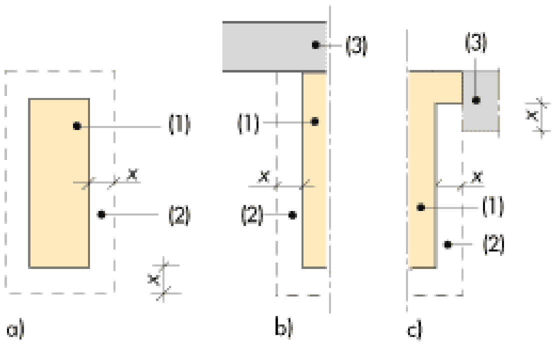


Figure 14.1 Effective residual section after fire (1) original section (2) To the left: Four sided fire exposure. Centre and to the right: Three sidedfire exposure, where the top is protected by material with better fire resistance (3).

The residual section can, as in the first method, be approximated as a rectangle or be calculated more accurately with regard to the rounding of corners. In the first case rectangular section   the charred depth is calculated as in 14.1 but with d_0= 0. In the second case   with rounded corners   the charred depth is calculated:

$$d_{char} = 0,64 \cdot t$$

Formula 14.2

The rounded corners are assumed to have the form of an arc whose radius (mm) increases thus with time:

$$r = \begin{cases} 0,67 \cdot t \text{ mm} & \text{for } t \leq 30 \text{ min} \\ 0,33 \cdot t + 10 \text{ mm} & \text{for } t > 30 \text{ min} \end{cases} \leq \min \begin{cases} 0,5 \cdot b_{res} \\ 0,5 \cdot h_{res} \end{cases}$$

Formula 14.3

The residual section can then be calculated using the formulae:

$$A_{rest} = b_{rest} \cdot h_{rest} - 0{,}86 \cdot r^2$$

Formula 14.4

$$I_{rest} = 0{,}0833 \cdot b_{rest} \cdot h_{rest}^3 - 0{,}215 \cdot r^2 \cdot h_{rest}^2 + 0{,}192 \cdot r^3 \cdot h_{rest} - 0{,}073 \cdot r^4$$

The design strength value is calculated, both with an assumed rectangular section and when

the rounding of corners is taken into account, using the following formula:

$$f_{fi,d} = k_{\mathrm{mod},fi} \frac{k_{fi} f_k}{\gamma_{n,fi} \gamma_{m,fi}}$$

Formulal 14.5

The design values of the Moduli of Elasticity and shear are calculated as follows:

The partial coefficients $\gamma_{n,\ fi}$and $\gamma_{m,\ fi}$are taken as γ_{fi}= 1,0. The factor $k_{fi}$is taken as 1,15. The latter has to do with safety questions and the fact that fire design may be based on 20% fractiles while the characteristic strength values in the regulations (f_k) are normally based on 5% fractiles. The modification factor $k_{mod,fi}$takes into account the effect of temperature on strength and stiffness and is calculated using the formula:

$$k_{\mathrm{mod},fi} = 1{,}0 - k \frac{p}{A_{rest}}$$

Formula 14.6

where

k = 0,008 for compression strength

k= 0,005 for bending strength

k= 0,003 for tension strength and E-module

p = circumference in metres of the part of the residual cross-section exposed to fire

A_{rest} = area of the residual section in square metres.

15.4.2.14.4.2 Design for parametric fire

As an alternative to fire design by classification, a more refined method may be used based on the parametric course of the fire instead of the standardized fire curve. Unlike the standard curve this is determined with regard to the geometry, ventilation and surface materials in the room and its surrounding structure. As a rule it is required that sufficient load carrying capacity remains during the complete fire course, including the cooling-down period.

The charring depth after cooling down can be calculated using the following formula:

$$d_{char} = 2\beta_{par} \cdot t_0$$

Formula 14.7

where

$$\beta_{par} = 1{,}05\frac{5F - 0{,}04}{4F + 0{,}08}$$

F= the opening factor (m½)

t_0= 0,006$q_{t,d}$/F= duration of fire (minutes),

$q_{t,d}$= numerical value of the design fire load density, related to area of the envelope of the fire compartment (MJ/m2)

The expressions apply provided that $d_{char} \leq b/4$ and $d_{char} \leq h/4$ where b and h are the width and depth of the original cross-section, and that the opening factor is within the interval $0{,}02 \leq F \leq 0{,}3$ m$^{1/2}$.

Further, the fire is assumed to be fuelled mainly by wood and the duration t_0 is assumed to be a maximum of 40 minutes. Note that the expression for d_{char} above covers the whole course of the fire, including the cooling-down period.

The modification factor $k_{mod,fi}$ with regard to the minimum strength value during the cooling down period can be estimated with the formula:

$$k_{mod,fi} = 1 - 3{,}2 d_{char} / b$$

Formula 14.8

where

b= beam width before fire

Starting from the above relationships a differentiated fire design of glulam beams can be carried out in the following stages. For definition of special concepts such as opening factor, fire load density etc and for further information, see special literature, e.g. Brandteknisk dimensionering av betongkonstruktioner (Anderberg, Pettersson: Swedish Council for Building Research, T13:1992).

1. Determine the critical fire load density for the type of space or building.

2. Determine the opening factor of the fire compartment.

3. Correct the fire load density and opening factor above if the thermal characteristics of the fire compartment differ from those of the standard fire compartment.

4. Calculate the charring depth d_{char} using formula 14.7.

5. Calculate the cross-sectional properties of the residual section of the structural element. Cross-sectional properties are calculated without taking rounded corners into account.

6. Calculate the design load bearing capacity R_d for the residual section in ultimate limit state. Use design strength values as in formula 14.5 with $k_{mod,fi}$ according to formula 14.8.

7. Determine the design load effect S_d under fire action.

8. Check that the design condition $R_d \geq S_d$ is met.

15.4.3.14.4.3 Required sectional depth

An expression can be developed saying how much a certain sectional property can be reduced in case of fire, without the fire case being critical. This can be done by combining the design conditions with and without regard to the action of fire. Using this expression it is then possible to derive a formula for calculating the (unreduced) sectional depth which, with given conditions for utilization factor, partial coefficients etc, is required if the fire case is not to be critical.

For a rectangular section loaded with moments and subjected to 4-sided fire action, the required depth of cross-section can be calculated as follows:

$$h_{erf} = \frac{2d}{1-\sqrt{\frac{b}{b-2d}\frac{K_{fi}}{K_0}}}$$

Formula 14.9

where

b, h_{erf} = sectional sizes before the fire

d = charring depth $d_{ef}$of d_{char}

K_0 = sectional property (section modulus) before fire

K_{fi} = sectional property (section modulus) after fire

K_{fi}/K_0= permitted reduction factor in formula 14.10.

For fire action on three sides as in figure 14.1 half the above value applies.

The quotient K_{fi}/K_0can be calculated using the following formula:

$$\frac{K_{fi}}{K_0} = \mu \frac{k_{mod}}{k_{mod,fi} \cdot k_{fi} \cdot \gamma_m \cdot \gamma_n} \cdot \frac{(\Psi + G_k / Q_k)}{(\gamma_q + \gamma_g G_k / Q_k)}$$

Formula 14.10

where

μ= utilization factor in design for ultimate limit state, without regard to fire

k_{mod} = modification factor for climate class and load duration according to Eurocode 5 (EN 1995)

K_{fi}, $k_{mod,fi}$defined in conjunction with formula 14.5

γ_g, γ_q = partial coefficients for self-weight and variable load respectively

γ_m,γ_n= partial coefficients for material and possible safety class

G_k,Q_k= self-weight and variable loading respectively, characteristic values

Ψ= load reduction factor for variable load.

15.4.4.14.4.4 Fire classes - Required cross section

Table 14.4 Required depth in mm for glulam beams with rectangular cross sektion 4-sided fire action[1].

FIRE CLASS	µ	DEPTH (mm) 90	115	140	165	190	215
R30	0,5	180	180	180	180	180	180
	0,75	225	180	180	180	180	180
	0,90	360	225	180	180	180	180
	1,00	585	270	225	180	180	180
R60	0,5	–	810	360	270	225	225
	0,75	–	–	765	450	360	315
	0,90	–	–	–	630	450	405
	1,00	–	–	–	900	585	450

1) If the top side is fire protected in at least R30 resp. R60 the half table value can be used.

15.5.1.14.5.1 Joints

The characteristic load bearing capacity of steel fasteners (nails, screws, bolts, dowels) can be assumed to fall with increasing temperature in the same way as that of steel structures. Further, it can be assumed that forces cannot be transferred between the fastener and charred timber. In practice this means that joints which transfer forces must as a rule be protected from fire, e.g. as in table 14.3, if a higher fire class than R30 is aimed for.

According to Eurocode 5 symmetrical, shear loaded joints timber to timber or timber to steel are assumed to meet the requirements of class R15 without special protection. Plates sandwiched between glulam members and slotted-in plates, are assumed to be at least 2 mm and side plates 6 mm thick. Unprotected joints with timber sides can reach a higher fire class than R15 if the edge distance (a3) and the spacing (a4) required in design without regard to fire is increased by afi mm (see figure 14.2):

$$a_{fi} = 0{,}7(t_{fi,req} - 15)$$

Formula 14.11

where

$t_{fi,req}$= the required fire resistance in minutes.

The thickness of the sides shall further fulfil the following conditions:

$$t_1 \geq \max \begin{cases} t_{fi,req} / (1{,}25 - \eta) \\ 1{,}6 t_{fi,req} \\ t_{1,\min} + a_{fi} \end{cases}$$

Formula 14.12

where

$t_{fi,req}$ = required fire resistance in minutes

$t_{l,\ min}$ = required thickness of sides without consideration of fire

η = S_d/R_d= utilisation factor in the joint at normal temperature

S_d, R_d = design loading effect and load bearing capacity (respectively) of the joint without regard to fire.

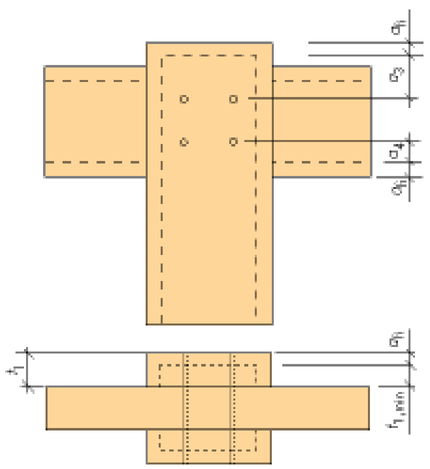


Figure 14.2. Increased sizes of unprotected joints for fire classes higher than R15.

Fire class R30 and higher can alternatively be reached by limiting the degree of utilisation h so that the following condition is fulfilled:

$$\eta \leq \eta_{30}\left(\frac{30}{t_{fi,req}}\right)^2$$

Formula 14.13

where

η_{30} is taken from table 14.1 below. The table also gives the length of fasteners and (for bolted or screwed joints with connectors) minimum screw/bolt diameters and minimum thicknesses of centre and side pieces.

Table 14.1

Factor η_{30} and required length (*l*) of fasteners and minimum thickness of side pieces (t_1) and centre piece (t_2).

Type of fastener		η_{30}	Dimensions
Nail	Timber to timber	0,80	$l \geq t_1 + t_2 + 8d$ $t_1 / d \leq 16$
	Timber to steel	1,00	$l \geq 90$ mm
Dowel	Timber to timber	0,80	$l \leq 2t_1 + t_2$ $l \geq t_2 + 2 \cdot 3d$
	Timber to steel	1,00	$l \leq 2t_1 + t_2$ $l \geq t_2 + 2 \cdot 3d$
Screw with or without connector		0,45	$t_1 \geq 75$ mm $d \geq 12$ mm

15.5.2.14.5.2 Column base

PINNED COLUMN BASE

Pinned fixing of column base, where the transfer of forces mainly takes place by contact pressure, e.g. as in figure 14.3a or b, is judged to fulfil the requirements for class R60 without additional fire protection. If there are horizontal forces, these must be taken up by contact pressure with a concrete projection (see figure 14.4a) if R60 is to apply without extra protection. A column base with steel shoe as in figure 14.4b, a common solution in conjunction with three-pinned portal frames, is judged to meet R30 without fire protection.

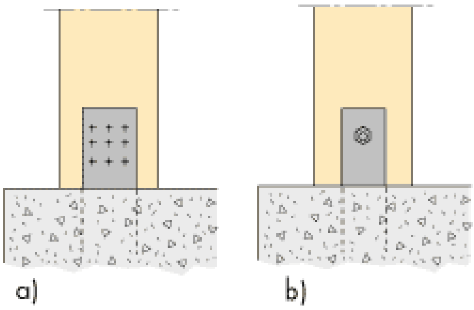


Figure 14.3 Pinned fixing of column base. a) Cast-in nailplate: R60. b) Cast-in flat steel and screw/bolt: R60.

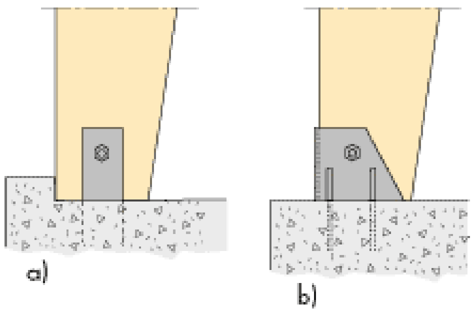


Figure 14.4 Pinned fixing of frame base. a) With concrete projection: R60. b) Steel shoe: R30.

RIGIDLY FIXED COLUMN BASE

A concrete column base (figure 14.5a) is judged to meet the requirements of R60 without additional fire protection.

A rigidly fixed column base with glued-in bolts (figure 14.b) is judged to meet the requirements of R30 or R60. The edge distance of the bolts is chosen so that they are within the effective residual section. When a steel shoe is used it must be given fire protection.

A steel column pocket (figure 14.5c) must be protected (see e.g. table 14.3) if there are requirements on fire class.

When fishplates are used for rigid fixing they must be protected (see e.g. table 14.3) if there are requirements on fire class. Edge distances of nails, bolts and screws with any connectors are chosen so that they are within the residual section.

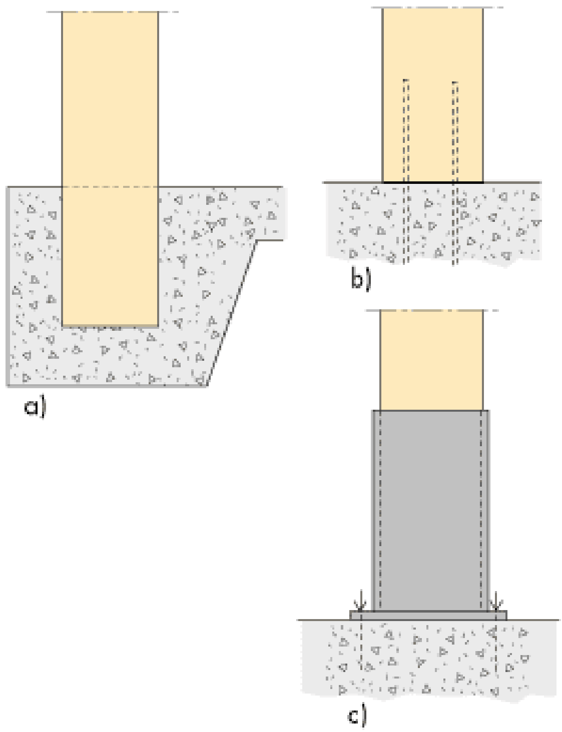


Figure 14.5 Rigidly fixed column base.a) Concrete column pocket: R60. b)Glued-in bolt: R30-R60. c)Steel column pocket: Must be fire protected.

15.5.3.14.5.3 Tops of columns

A forked concrete column top (figure 14.6a) is judged to meet the requirements of R60 provided dimensions are sufficient.

A forked glulam column top (figure 14.6b) can, according to German sources, be judged to meet the requirements of R30 if it is at least 25 mm thick and the width of the beam at least 80 mm, or R60 if the fork is at least 40 mm thick and the width of the beam at least 120 mm. For beams whose depth is at least 4 times the width, the required thickness of the fork is increased to 80 mm or 140 mm respectively, due to the risk of buckling. This design is unsuitable if horizontal forces are to be transferred during a fire.

Fixing with flat steel as in figure 14.6c is judged to meet the requirements of R30 if only downward vertical load is to be transferred and the lateral stability during a fire is secured by special measures, e.g. by utilising adjacent roof or wall structures. To meet class R60, fire insulation is required (see table 14.3).

Corresponding connections using nailplates, e. g. BMF, also require fire insulation in class R30.

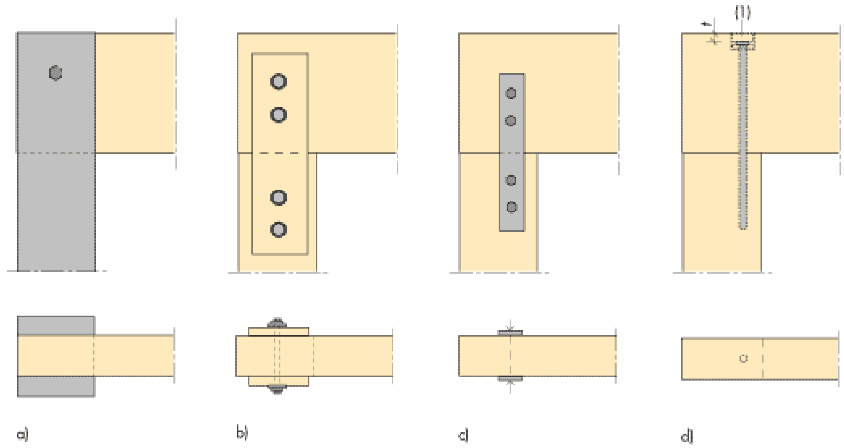


Figure 14.6. Connection between column and beam. a)Concrete fork: R60. b)Glulam fork: R30-R60. c)Steel flats: R30. d)Glued-in screw: R30-R60.

Fixing with glued-in bolts has been tested in Finland and is judged to meet the requirements of R30 or R60. The edge distance of the bolts is chosen so that they are within the residual section. The lateral stability of the beam in a fire must be secured by taking special measures, e.g. via wall or roof structures.

Connections with a slotted-in T-section (figure 14.7) are classified as R30 in German regulations if the width of the beam is at least 120 mm and as R60 if it is at least 230 mm. The width of the slot is a maximum of 10 mm in each case. For slender beams (*h*/*b* = 4) in class R60 the back of the T-section must be fire insulated (e.g. as in table 14.3) so that charring of the slot does not cause stability problems.

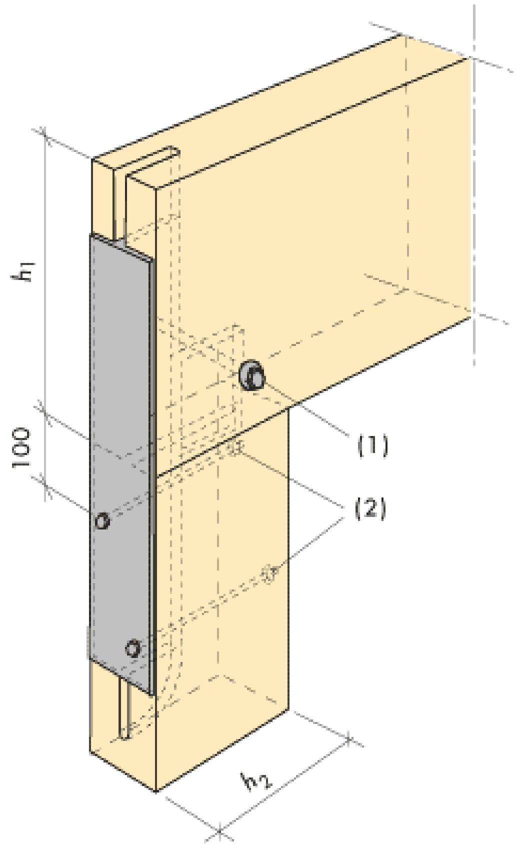


Figure 14.7. Connection column-beam with slotted-in T-section: R30-R60.

15.5.4.14.5.4 Connection with secondary beam

BMF-type beam hangers without fire insulation (figure 14.8) can according to German regulations be classified as R30 if the utilisation factor does not exceed 75%. The minimum sizes in table 14.2 must however be followed and extra long nails are also required: 75 mm instead of the normal 40 mm.

Hangers with bent-in flaps (figure 14.9) are placed in the same fire class if the minimum sizes in table 14.2 are followed.

At joints with columns (figure 14.10) the edge distance e_1 must be respected in order to reach class R30. e_1 is taken from table 14.2.

A beam fixture as in figure 14.11 is classed as R30 if the minimum sizes given in the figure are maintained and the contact pressure during a fire does not exceed 1,25 N/mm^2
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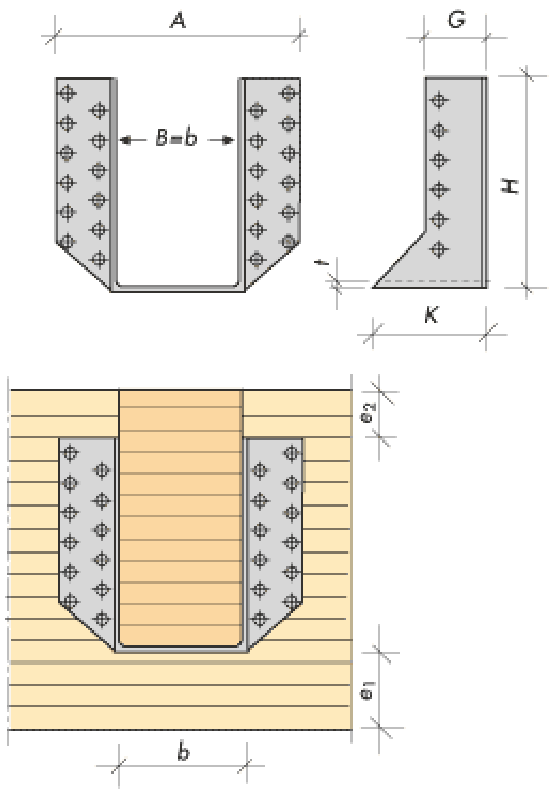

Figure 14.8. Connection of secondary beam using BMF hanger: R30.

Table 14.2

Minimum sizes for class R30 at connection of secondary beam using BMF-type hanger. Symbols, see figure 14.8.

	Utilisation factor[1)]33%	Utilisation factor[1)] 75%
B (mm)	100	120
A (mm)	170	200
G (mm)	40	44
K (mm)	75	85
t (mm)	2	2
e_1 (mm)	50	100
e_2 (mm)	20	30
Nail length (mm)	75	75
Number of nails		
in primary beam	2×6	2×7
in secondary beam	2×12	2×13

1)Load effect under the action of fire as a percentage of the design load bearing capacity when designing without regard to fire.

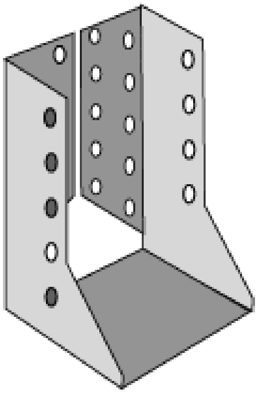

Figure 14.9 Hanger with bent-in flaps.

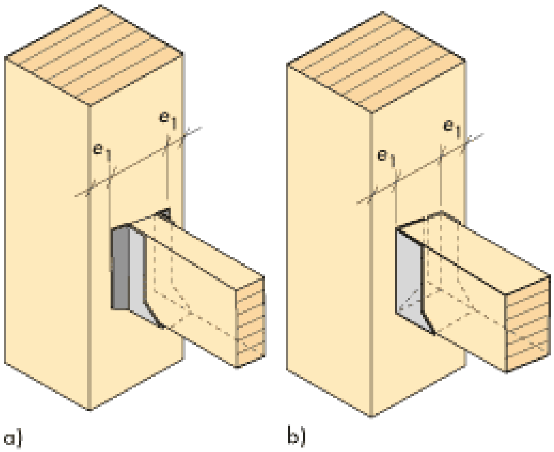

Figure 14.10. Edge distance at beam-column connection with BMF-type hanger: R30 under certain conditions (see table 14.2). a) Bent-out flaps. b)Bent-in flaps.

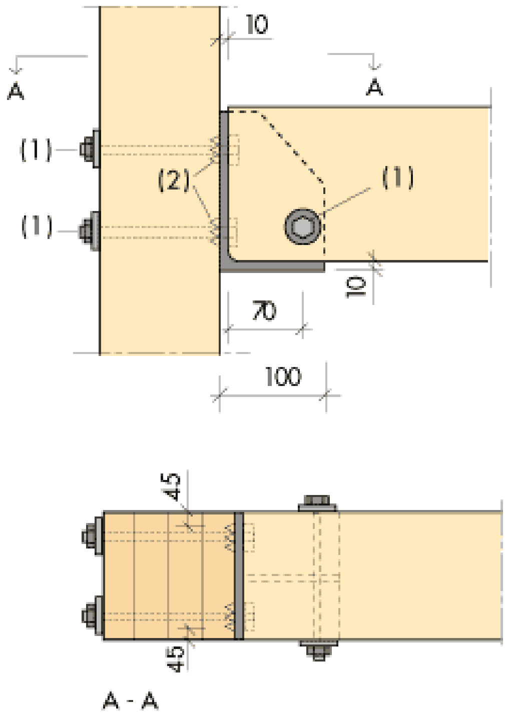

Figure 14.11. Beam connection with steel bracket and screws/bolts with connectosr: R30.

15.5.5.14.5.5 Beam joints

Bolted beam joints (see figure 14.12) meet the requirements of R30 without fire insulation if the load effect in a fire is a maximum of 65% of the design load bearing capacity without regard to fire.

The requirements of R30 are met by fire insulation as in table 14.3, even if the utilisation factor in fire is 100%.

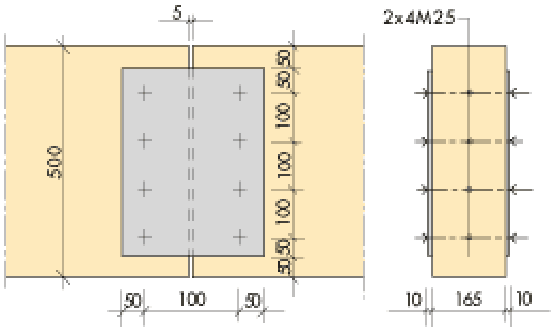

Figure 14.12 Bolted beam joint: R30.

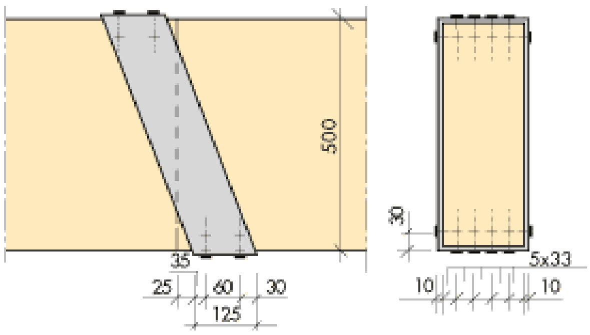

Figure 14.13. Beam joint with Gerber fixture: must be protected.

The Gerber fixture illustrated in figure 14.13 needs fire insulation if it is to receive a fire class. With insulation as in table 14.13 the requirements of R30 are met if the utilisation factor in fire is a maximum of 65%.

15.5.6.14.5.6 Ridge joints

Ridge joints with nailplates as in figure 14.14a require fire protection as in table 14.3 if fire class R30 or R60 is to be achieved.

If aesthetic requirements are high, BMF-type ridge fixtures (figure 14.14b) can be a suitable alternative. Without fire protection this design is judged as meeting the requirements of R30. If the space between the beams is filled with mineral wool ($\rho \geq 140$ kg/m^3) and the fixture is placed within the effective residual section, the fire class is assessed as R60.

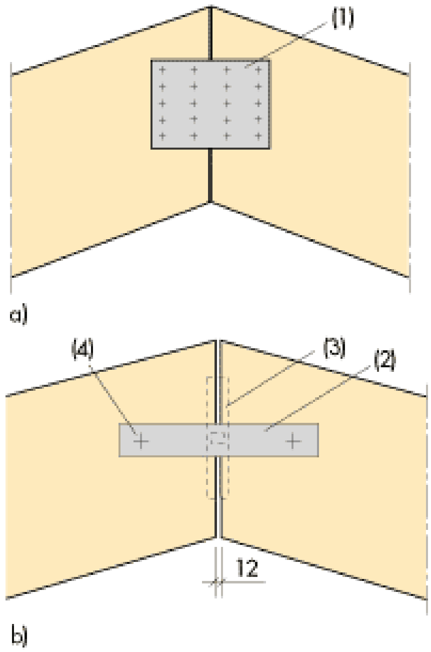

Figure 14.14 Ridge joint a) Nailplates (1): fire protection must be applied. b) BMF-type ridge fixture (3) steel flat (2) bolt (4) R30

15.5.7.14.5.7 Steel ties

An unprotected steel tie does not as a rule meet the requirements of R30. Protection is most easily provided by applying preformed mineral fibre sections. If visual standards are high these can be enclosed in plastic tubes.

Note that the increase in length due to the rise in temperature is considerable despite the insulation, and that the supports must be designed with regard to this.

Ties fixed as in figure 14.15 must have fire protection in order to reach a fire class. The material and thickness of insulation can be chosen with the aid of table 14.3.

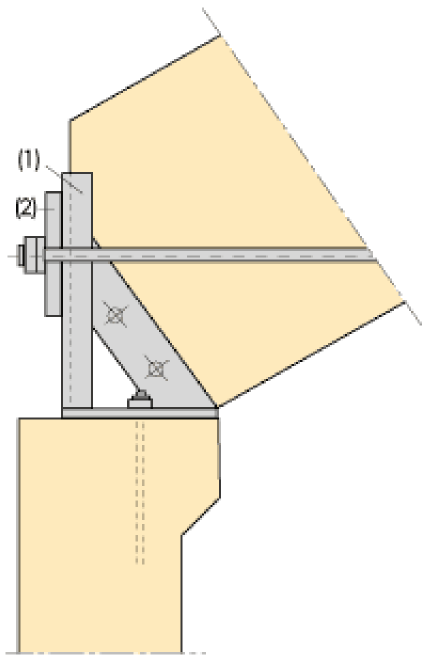

Figure 14.15 Steel tie fixture: fire protection must be applied. U-channel section (1), steel flat (2).

15.5.8.14.5.8 Fire protection

Previous experience points to the view that connections and joints with exposed steel parts can only exceptionally reach a higher fire resistance than 15 20 minutes, i.e. as a rule less than that of the connected timber components. Steel parts must therefore often be protected from the direct action of fire in order to delay a rise in temperature. The insulation methods that are used are in part those used for larger steel structures, e.g. painting with fire retardant paint or cladding with various types of incombustible sheet material and in part cladding with timber or wood-based sheet materials. As far as nails and screws are concerned, countersinking and plugging is a method which is often used.

In table 14.3 examples of required insulation thicknesses are given for various fire classes and various insulating materials.

Mineral fibre sheets are held to be the cheapest alternative. Even if the detail to be protected has an uneven surface the mineral fibre sheet can be made to cover the detail and at the same time provide a tight joint with the glulam. Mineral fibre can be fixed with nails if these are provided with a large washer under the head. The diameter of the nail must be at least 3 mm and the penetration in the timber at least 25 mm. The washer must have a contact area against the insulation of at least 6 cm2.

The distance from a nail to the edge of the insulation or to a joint in the insulation material may not exceed 100 mm. The distance between nails along an edge or a joint is chosen so as to give a tight joint between timber and mineral fibre. The distance between nails must however not exceed 400 mm.

Fire retardant paint is considerably more expensive than mineral fibre. If aesthetic requirements are high it can nevertheless be an interesting alternative. The fixing details can usually be pre-painted. If the details are nailed on, the heads of the nails can be left unpainted as they will be protected by the carbon froth formed by the paint when hot. Whether this can also be assumed for screw or bolt heads and nuts should be discussed with the paint manufacturer.

Table 14.3. Examples of fire protection.

Material	Thickness (mm) R30	R60
Mineral fibre sheets (ρ = 140 kg/m^3)	30	70
Fire retardant paint	1)	1)
Plasterboard, normal quality	13	2×13
Fibre silicate sheets (ρ = 450 kg/m^3)	10	20
Plywood or glulam sheets	20	50

1) According to manufacturer's instructions.

Plasterboard and fibre silicate sheets can be used to good effect where the steel details can be recessed or only project a few mm from the glulam. Plasterboard is the cheaper of these sheets. Both can be painted. Plasterboard can be of normal quality. Sheets can be fixed direct to the glulam with Gyproc nails or screws. Timber penetration shall be at least 25 mm.

The distance from nail or screw to edge of sheet shall be approx 15 mm. Distance between fasteners along the edge shall not exceed 150 mm for nails or 200 mm for screws. Nail or screw spacing shall in other respects correspond to an average distance of approx. 125 mm in both directions.

Glulam or plywood sheets can be used to protect nails or screws against fire, or to build in projecting details. Countersunk screws can be protected with timber plugs glued into the countersinking.

The sheets are nailed, screwed or (even better) nailed and glued to the face of the glulam, in which case the minimum values of edge distance and spacing given in current regulations are also the maximum values along the edges of the sheet. Nail or screw spacing shall in other respects correspond to an average spacing of approx. 125 mm in both directions.

17.2.1.16.2.1 Slab bridges

The simplest type of timber bridge is the plank, which combines both roadway and structural system in the same structure. A modern development of this principle is the transversely pre-stressed timber slab bridge (see figure 16.4).

The transverse stressing technique was developed in Canada and has in recent years been used both in the US and in Switzerland. In the Nordic countries, a large number of transversely pre-stressed bridges have been built in recent years.

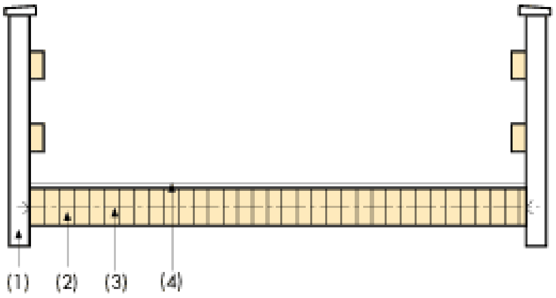

Figure 16.4 Slab bridge with transversely pre-stressed slab of glulam. (1) Handrail. (2) Upright glulam beams. (3) Steel tension rod. (4) Road surface.

The road slab is made of glulam beams   for smaller spans planks   pressed together by stressed steel rods. The structure is easy to erect and the slab has good load distribution characteristics. It is laterally stiff and extra wind bracing is therefore unnecessary. The slab is normally provided with damp proofing and a top surface that protects the timber from moisture from above. The exposed timber surface is thus small and the variations in moisture content (moisture movement) also small.

The roadway can with advantage be made continuous in several bays. It can also be used as the top or bottom member in a truss, or form a part of a hanging structure, strut frame or similar (see below).

17.2.2.16.2.2 Girder bridges

In girder bridges the load-bearing construction usually consists of two or more longitudinal glulam beams; for small spans and small loads the beams can be of sawn timber. The beams can span one or more bays. When the distance between beams is small, the roadway consisting of planks can rest directly on the beams (figure 16.5). In small bridges the roadway can act as a diaphragm and take up horizontal loads, e.g. wind, while preventing the beams from buckling. Larger spans demand wind bracing, usually in the form of horizontal trusses between the main beams, level with their tops or undersides.

If the main beams are more widely spaced, the roadway will instead rest on transverse secondary beams which transfer the load to the main beams. The roadway of planks resting on the secondary beams has usually a top surface of asphalt (figure 16.6).

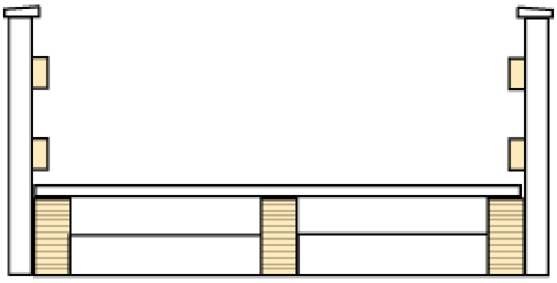

Figure 16.5 Girder bridge with planks laid direct on the glulam beams.

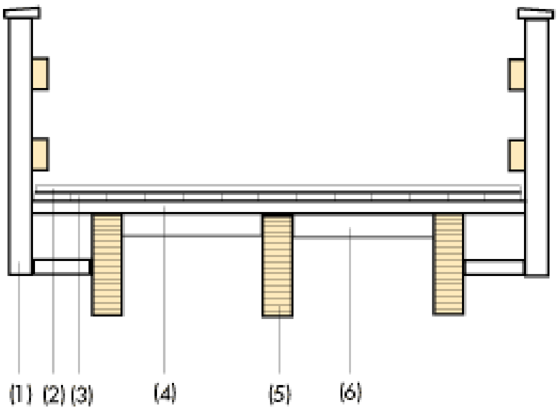

Figure 16.6 Girder bridge with transverse secondary beams and planks. (1) Handrail. (2) Road surface. (3) Planks. (4) Transverse secondary beam. (5) Glulam beams. (6) Wind bracing.

For long spans and heavy traffic the roadway is often designed as a pre-stressed slab, structurally co-acting with the main beams in a T or box section (see figures 16.7 8). The roadway then acts as a diaphragm for horizontal loads and no special measures to counteract wind forces are therefore necessary.

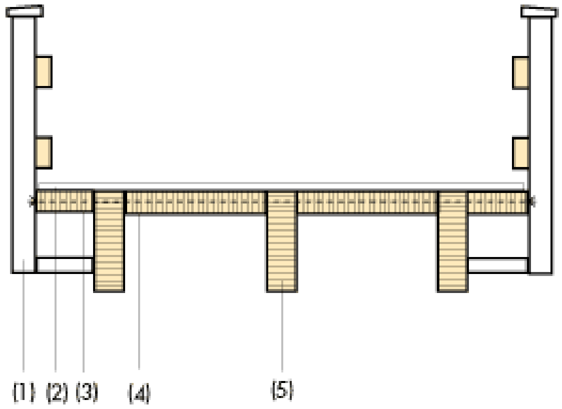

Figure 16.7 T-beam bridge with pre-stressed slab. (1) Handrail. (2) Road surface. (3) Pre-stressed roadway. (4) Steel tension rods. (5) Glulam beams.

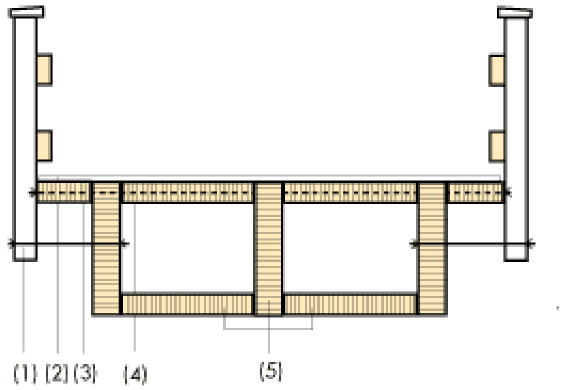

Figure 16.8 Pre-stressed box beam bridge. (1) Handrail. (2) Road surface. (3) Pre-stressed roadway. (4) Steel tension rods.

Beams designed as king post or strut frame structures were formerly used for spans too large to be bridged with ordinary girders (see figures 16.9 10). The main beams were thus provided with one or more intermediate springing points, which meant that the material was used more effectively. The first type can be seen as a simple form of truss.

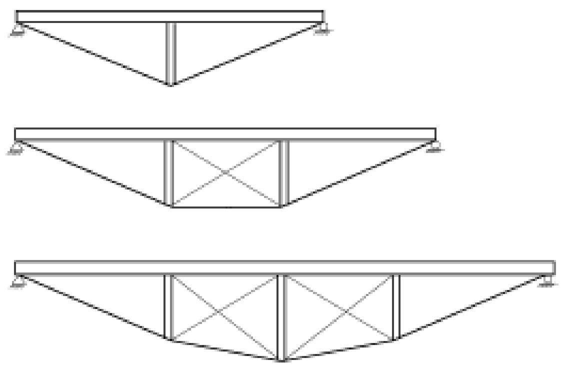

Figure 16.9 Examples of trussed systems with steel tension chord and timber posts acting as intermediate elastic supports.

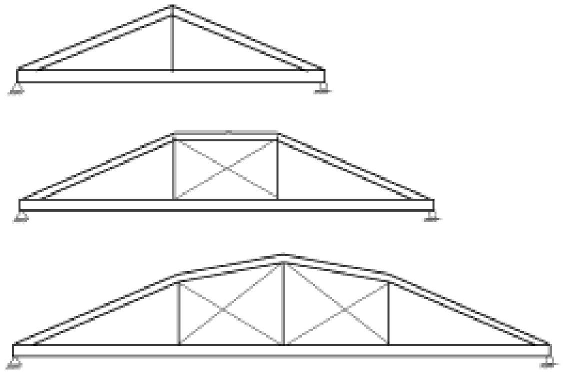

Figure 16.10 Examples of trussed systems with glulam compression chords and timber posts acting as intermediate elastic supports.

The tension rods are normally steel. When designing it is necessary to take the differing (heat) expansion and stiffness properties of the materials into account.

Strut frames can be an effective solution e.g. for a girder bridge spanning a deep ravine where the inclined struts can be supported by the sides of the ravine. The strut frame can be seen as an intermediate form between a girder bridge and an arch (see figure 16.11).

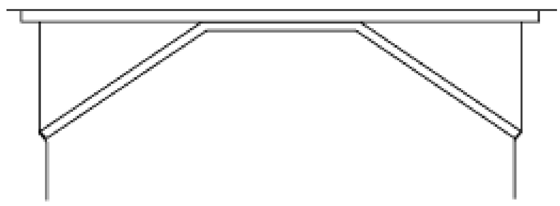

Figure 16.11 Example of a simple strut frame.

17.2.3.16.2.3 Truss bridges

Truss bridges are used for spans where solid beams are no longer competitive. Parallel trusses are most usual, but parabola-shaped trusses are also used, usually with continuous top and bottom members. Parallel trusses can be continuous over several supports. They can also be integral with the roadway, which can then be used for transverse stabilisation or conceal a wind bracing.

Trusses are factory-assembled in suitable lengths for transport and erection.

17.2.4.16.2.4 Arch bridges

The arch form means that the structure   at least for uniformly distributed loads   is mainly subject to compression. It is therefore specially suitable for material with a high compressive strength and where the arch form does not complicate manufacture. Glulam unites these characteristics and is a frequent choice for timber arch bridges.

The structure is normally designed with double arches and the roadway placed either under, between or on top of the arches (figure 16.12). The arches are laterally restrained by trusses or frames. Together with the roadway these are also used to take up wind loads and other horizontal loads affecting the structure.

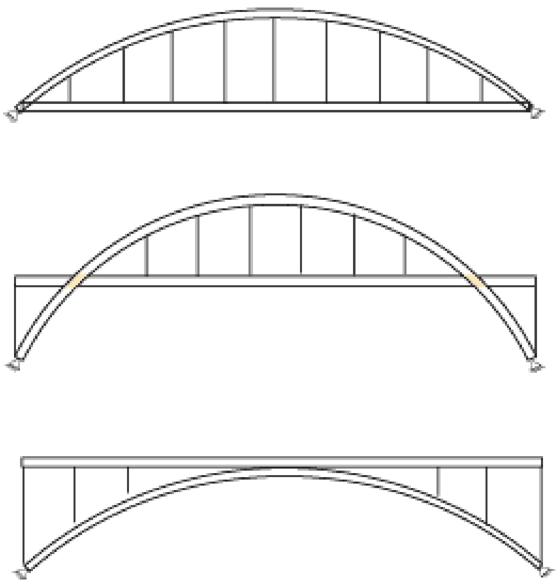

Figure 16.12 Arch bridges with through, half-through and deck roadways.

For transport and manufacturing reasons the arches are often designed as 3-pin arches, specially for spans over 20 metres. The 3-pin arch also has the advantage that the structure is statically determinate and can cope with relatively large displacements in the foundations. If 2-pin arches are more suitable for other reasons, it is however possible to construct rigid joints on the building site.

17.2.5.16.2.5 Suspension and cable stayed bridges

A suspension bridge consists of a stiffened roadway which with the aid of vertical ties hangs from suspension cables spanning between two towers (figure 16.3).

A cable stayed bridge consists of a girder bridge supported on two or more rigid supports. Between the rigid supports, the beams are elastically supported by inclined cables from one or more towers. The oblique support reactions from the cables create compressive forces in the roadway. This is in fact a pre-stressing which can be utilised to increase the moment capacity of the girders. The differing stiffness and (heat) expansion characteristics of the materials must be taken into account in designing.

17.3.1.16.3.1 General

The aim of bridge planning is to find the optimal design of the bridge from given conditions of the location of the bridge, the terrain, geo-technology and hydrology, the space available, road geometry etc. In addition to the technical and economic assessments which must always be carried out there are other interests to be satisfied. Amongst these are aesthetic requirements, which depending on the size and position of the bridge are accorded greater or less importance; and operational and maintenance considerations.

17.3.2.16.3.2 Design

Cross-sectional forces in a timber bridge structure can, in the same way as in building construction, be calculated with the aid of the formulae and instructions in chapters 4 - 10.

The carrying capacity of a timber bridge structure can be calculated in the same way as for buildings, taking into account the special safety requirements and conditions regarding loading combinations and climatic conditions given in the bridge regulations.

17.3.3.16.3.3 Durability

In the Nordic countries load-bearing timber structures are normally of pine or spruce. These species are not attacked by timber fungi as long as the moisture content does not exceed 20% for long periods. With careful design of the superstructure, the greater part of the timber structure can be maintained below this level. For parts in contact with the ground, or where the constructional timber protection is assessed as insufficient to keep the moisture content sufficiently low, the natural resistance of timber can be supplemented by chemical timber protection. Treated timber is however a controversial product from the environmental point of view, and its use is regulated in environmental regulations. In some countries it is thus a criminal offence to use creosote or preservatives containing chromium or arsenic compounds other than when this is necessary for long-term timber protection:

- in contact with the ground
- in marine situations
- in handrails and other forms of accident protection
- in structures in damp-prone environments where the piece is difficult to replace.

Bridge handrails and the primary beams of a bridge structure should be covered by the last two points, and can therefore in such cases be made of treated timber.

In certain cases, however, road authorities no longer accept preservatives containing creosote, chromium or arsenic compounds. At present long-term experience is lacking on the more environment-friendly preservatives which have been developed to replace the CCA-chemicals (copper, chromium and arsenic).

To guarantee a sufficiently long life-span for the structure, steel details (e.g. joint fixtures) must be protected from corrosion or be of corrosion-proof material. Corrosion is sustained primarily by road salt, water, rain and condensation. It should be noted that a timber bridge over a major road will be affected by the salt which swirls up from that road. For specially exposed or vital parts, stainless steel or extra protective treatment against corrosion should be considered.

17.3.4.16.3.4 Surface treatment and maintenance

Untreated timber subjected to the elements ages, partly due to the ultra-violet rays of the sun. The timber surface erodes and changes colour, and cracks of various width and depth appear. Nevertheless the structure can survive for hundreds of years, as many historic buildings show.

It is, however, often desired to protect the timber against surface erosion and cracking and to reduce variations in the moisture content   or else there is a desire for a different colour than that of ageing wood. Either the timber structure can be given a surface cladding, or the timber surface can be treated.

Treatment of exposed timber out of doors has many purposes

- Protection against absorption of water
- Delaying the exchange of moisture with the surroundings
- Protection against UV rays
- Protection against erosion by the elements
- Giving the timber a certain colour

Protection against absorption of water can be achieved with a water-repellent product which reduces the wetting and capillary action, or with a treatment which forms a film preventing water from reaching the surface of the timber.

Exchange of moisture with the surrounding air can be prevented by a more or less vapour-tight surface treatment.

Protection against UV rays can be provided by adding pigments which prevent the passage of light. The pigment also provides the treatment with a certain colour.

It is recommended that glulam surfaces which are not clad and which are exposed to sunshine or precipitation shall be protected with a pigmented surface treatment at least 60 m thick. The treatment, which should be carried out in the factory or immediately after erection, can consist of 1 2 coats of an alkyd-oil based stain with fungicide additive and two coats of alkyd-oil-based opaque paint.

It is normally necessary to renew the treatment at intervals of a few years. For parts which are only indirectly exposed, the maintenance interval can be longer. Surfaces completely protected from precipitation require no surface treatment at all.