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The Glulam Handbook

Project design of glulam structures

Volume 2

SWEDISH WOOD

The Glulam Handbook Volumes 1–3 are the result of a collaboration between glulam manufacturers and the industry organisations in Finland, Norway and Sweden. *The Glulam Handbook Volumes 1–3* are available in three languages – English, Finnish, Norwegian and Swedish. The content of these versions is adapted to meet Eurocode 5 and the associated national annexes, NA.

The Glulam Handbook Volume 4 is available in Swedish and English. It was produced by Swedish Wood and funded by the Swedish glulam manufacturers.

This publication is the second of the four-part Glulam Handbook.

- Volume 1 contains facts about glulam and planning guidance.
- Volume 2 provides calculations for the structural dimensioning of glulam.
- Volume 3 gives a number of example calculations for the most common glulam structures.
- Volume 4 provides knowledge on the planning and assembly of glulam structures.

Further knowledge, information and practical instructions on wood, glulam, CLT and wood construction are available on Wood Campus, **woodcampus.co.uk**, which is constantly updated with new knowledge and practical experiences. Wood Campus is an extensive resource with tables, drawings and illustrations.

Welcome to **woodcampus.co.uk**.

Information on wood, glulam, CLT and wood construction can also be found at **www.swedishwood.com**.

Stockholm, March 2024

Johan Fröbel
Swedish Wood

Cover: Göranssons Arena, Sandviken, Sweden.

Preface

The purpose of *The Glulam Handbook Volume 2* is to assist the reader in the design of glulam structures. Particular emphasis has been given to the understanding of:

- The background for the design of structural systems made of glulam, especially large-span structures.
- The background for the design of connections in structural glulam systems.

The present Volume is mainly related to the European standard EN 1995-1-1:2004 (Eurocode 5 — Design of timber structures — Part 1-1: General Common rules and rules for buildings). Some basic rules for the design of steel members and connections, according to the European standard EN 1993-1-1:2005 (Eurocode 3 — Design of steel structures — Part 1-1: General rules and rules for buildings). In addition, the rules given here are based on the Swedish application rules connected to EN 1995-1-1, described in the document EKS 10 (BFS 2015:6). However, in case of lack of rules or questionable design methods present in Eurocode 5, other design approaches are proposed. For example, for the design of beams with holes, loads attached close to the tension side of a beam, and reinforcements to prevent cracking due to stresses perpendicular to the grain, etc. the German code DIN EN 1995-1-1/NA is used (Nationaler Anhang — National festgelegte Parameter — Eurocode 5: Bemessung und Konstruktion von Holzbauten, Teil 1-1: Allgemeines — Allgemeine Regeln und Regeln für den Hochbau). The Swiss SIA 265:2003 (Holzbau) as well as approaches based on research results and practical experience are also adopted in this document.

This Volume is primarily intended for structural engineers and engineering students.

The interpretations of the building codes, research reports, industry literature, etc. are those of the authors and are intended to reflect current structural design practice. The material presented is suggested as a guide only; final design responsibility lies with the structural engineer.

Lund, March 2016

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Contents

Malmö Central Railway Station, Sweden.

Glulam as a structural material

PATENT Nº 28715.

BESKRIFNING

OFFENTLIGGJORD AF

KUNGL. PATENT- OCH REGISTRERINGSVERKET.

O. HETZER,

WEIMAR (TYSKLAND).

Böjd träkonstruktionsdel för byggnadsändamål.

Klass 27: c.

Patent i Sverige från den 15 juni 1907.

Prioritet från den 21 juni 1906.

Cover of the Swedish patent for the Hetzer Binder.

Glued laminated timber, more commonly known as glulam is a highly engineered wood product. Its finger-jointed strength-graded timber lamellas can be used to create beams of almost any size and shape. And being based on the use of the only truly renewable building material — wood — glulam has obvious advantages from an environmental point of view.

Glulam possesses excellent strength and stiffness properties compared with sawn timber of a corresponding size. As it is stronger in relation to its self-weight than steel, glulam can be used to create large structures, with spans of up to or over 150 m. The combination of being able to freely determine the cross-sectional shape, with the availability of tapered or curved beam shapes and the capability for long span structures has made glulam the preferred choice in many projects of high architectural value.

Today, glulam is used in virtually all types of construction works, including small houses, multi-storey dwellings, halls, sport arenas and road bridges. Through appropriate design, detailing and climatic protection (possibly using surface treatment), glulam can be used in a wide range of applications.

Otto Hetzer (1846–1911), a carpenter, engineer, sawmill owner and inventor from Weimar was the holder of five different German patents issued between 1892 and 1907, all relating to different building components that make use of timber as an engineering material. In the patent DRP No. 197773 from 1906, Hetzer describes how to build up laminated beams with a curved shape. The beams were called “Hetzer binder”. The adhesive used was a casein adhesive (milk protein based). An important early structure was the “Reichseisenbahnhalle” for the world exhibition in Brussels, 1910. The arched glulam frames with tension rods had a free span of 43 m. At that time Hetzer had already realised around 50 projects with relatively large spans.

The introduction of glulam technology in the Nordic countries took place during the second decade of the 20th century.
The Norwegian engineer Guttorm Brekke (1885–1980) had spent some time in Weimar at the Otto Hetzer AG company, and after acquiring the rights for Norway, Sweden and Finland, set up a production plant for the Hetzer binder in Mysen, Norway. A company called Trekonstruktioner A/S was founded in Kristiania (Oslo), Norway, in 1918. In 1919 a Swedish subsidiary company was established in Töreboda, where production of glulam continues to this day. Some of the first glulam structures in Sweden are the concourses of the central railway stations in Stockholm, Gothenburg and Malmö. They were all supplied and built in the 1920s.

There are about ten established glulam factories in the Nordic countries today. Thanks to the harmonisation of standards in Europe, the certification processes in the different countries are identical. The harmonised standard EN 14080 lays down the general requirements to be met by glulam producers in order to be able to CE-mark their products.

1.1 Introduction

Up until the beginning of the 1960s, Scandinavian glulam production was fairly small, but today total production in the Nordic countries is well above 200,000 m^3, of which roughly half is exported.

Most of the glulam used in the Nordic countries is used for industrial buildings, schools, day nurseries and housing, including multi-storey apartment buildings. Together, these account for about 60 percent of the consumption. Glulam is, however, a versatile material and has been used over the years in a wide range of applications, from shuttering, scaffolding and playground equipment to bridges, multi-storey car parks, ski slopes and electricity pylons.

Modern adhesive technology, in combination with the excellent strength properties of timber, makes glulam a highly effective structural material with a unique set of characteristics:

- An appealing appearance that adds value to interiors and exteriors.
- A high strength-to-weight ratio, enabling wide spans.
- Small manufacturing tolerances and good shape stability within normal temperatures and moisture conditions.
- High resistance to fire — often a requirement in public buildings.
- Good heat insulation characteristics, reducing the effect of thermal bridges and reducing the risk of condensation.
- Low weight, resulting in low transport and erection costs and reduced foundations cost and complexity.
- A 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 can be assembled by the use of simple and traditional methods such as nailing, screwing or bolting, unaffected by the time of year or the weather. In addition to these traditional means, more sophisticated methods are available for more demanding situations, for example slotted-in steel plates and dowels or bonded-in rods. A glulam frame can carry its full load immediately after erection and, being a wood-based material, any adjustments necessary on site can be made with simple hand tools. Timber construction is a dry building method that requires appropriate weather protection, which in turn also leads to benefits in terms of the working environment on site.

Timber has been used in construction for centuries and, when used correctly has extremely good durability. In the Nordic countries there are examples of timber buildings over a thousand years old.

Stockholm Central Railway Station – the existing main hall was built in 1925. Architect: Folke Zettervall.

Malmö Central Railway Station main hall, built in 1923. This is one of the first great uses of glulam in Sweden. The station building is today in working order.

1.2 Glulam in the life cycle

When sourced from sustainably managed forests, such as in the Nordic countries, timber has substantial environmental and climate change benefits, compared with other building materials. Firstly, the production of glulam uses little energy. Secondly, during production, bi-products such as saw dust and wood chips are used to produce energy (for example to heat the kilns), thus replacing fossil fuels. Sustainable forest management policies ensure that the harvest from the forest is not greater than the growth, so that the raw material is constantly renewed without affecting the climate negatively by green house gas emissions.

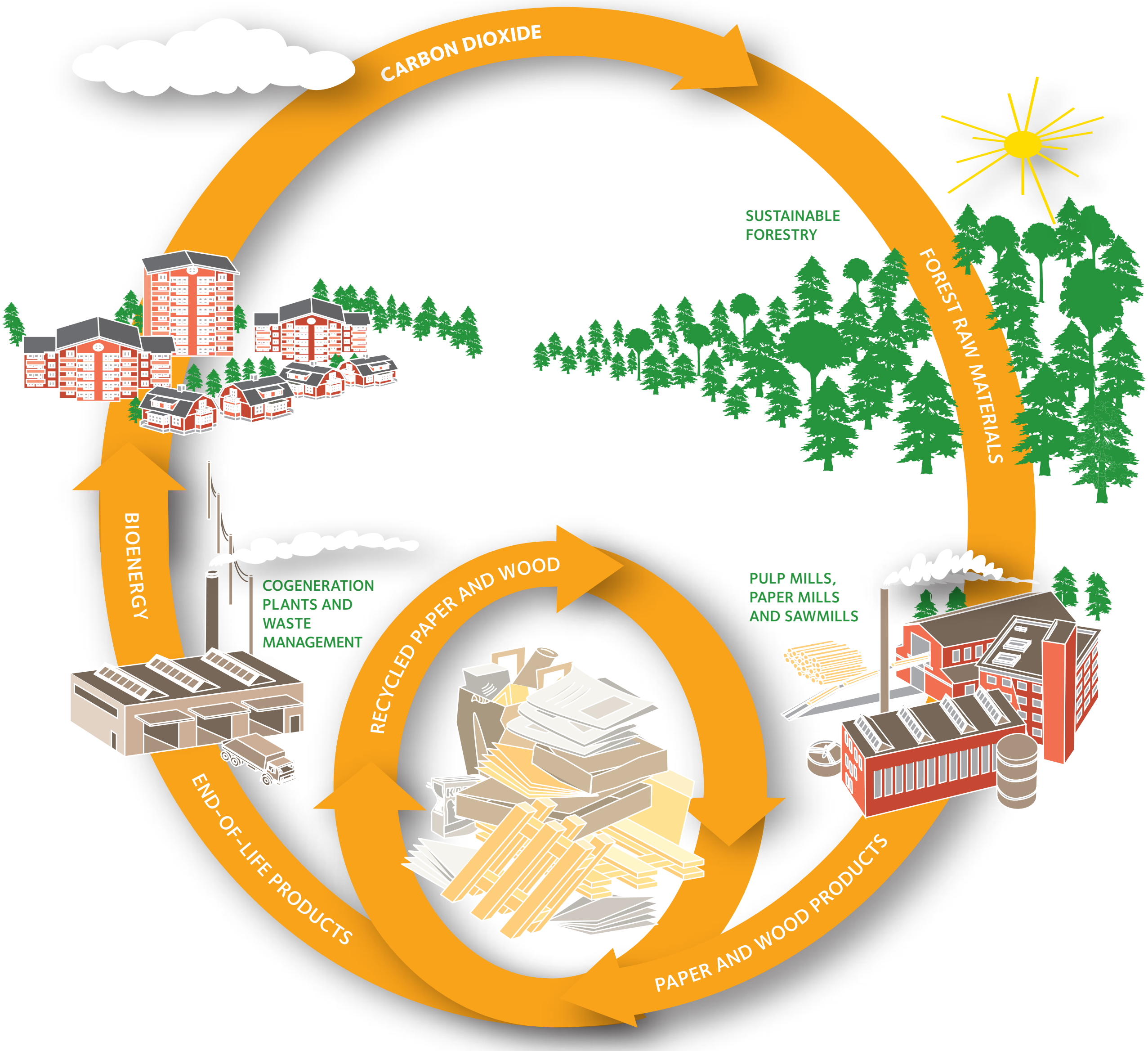


Figure 1.1 **Ecocycle of wood products**

The ecocycle comprises two parts. One relates to the forest and the other to the products. The forest gains its vitality from the sun. Through photosynthesis, solar energy is absorbed and reacts with carbon dioxide (CO_2) to produce nutrients for the growing trees. The forest's products contain carbon (C) that has been absorbed by the trees in the form of carbon dioxide.

The ecocycle of the products includes reuse, repair and recycling. When these products reach the end of their life, the carbon dioxide is released into the atmosphere as the waste decays or is recycled as bioenergy. The carbon dioxide is then captured again by the trees and converted into nutrients and new building blocks for their growth.

During growth trees absorb CO_2 through photosynthesis, storing it in the wood as carbon, which is not released to the atmosphere until the wood is used for e.g. heat production. If the use of timber in construction increases, replacing other less climate efficient materials, a temporary additional carbon sink is established. Although limited in time (50–100 years), it is of great value in temporarily reducing the net emission of green house gases. One cubic metre of glulam stores approximately 750 kg of CO_2. The life cycle of glulam is shown in *figure 1.1*.

Additional environmental benefits can be attributed to the production methods used in glulam production. Glulam is made of timber lamellas bonded to one another with adhesive, which do not affect the environment during their life cycle and can also easily be re-used, reclaimed or used for energy production. The raw material is spruce (picea abies), and a synthetic adhesive. The adhesive is made from non-renewable raw materials, which affects the environmental profile negatively. The amount of adhesive per unit volume is, however, so small (less than 1 percent by 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.

Since glulam is often tailored for specific projects it does not cause significant building waste on the site. Wrappings consist of material that can be recycled.

During its lifetime, glulam has no negative environmental effects of any importance. It can be maintained using traditional methods. It can be repaired easily — parts of a glulam component can, if necessary, be replaced. It can, if needed, be finished in various ways, e.g. by abrasive treatment or rubbing down.

Like all timber, glulam is combustible, with the same energy content as solid softwood. If glulam is unsuitably used or used in poorly detailed construction, it can be subject to biological decay.

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

Spruce forest.

1.3 Glulam production

The term glulam in the text that follows is used with the same definition as that given in the harmonised standard EN 14080, meaning that all production and product requirements stated in that standard are fulfilled. Thus, glulam is a structural component consisting of at least two glued boards or planks from a coniferous timber species. The lamella thickness must be a minimum of 6 mm and a maximum of 45 mm, and the grain direction must coincide with the longitudinal direction of the component. The adhesive bond lines are parallel with the width (normally the smaller face of the beam).

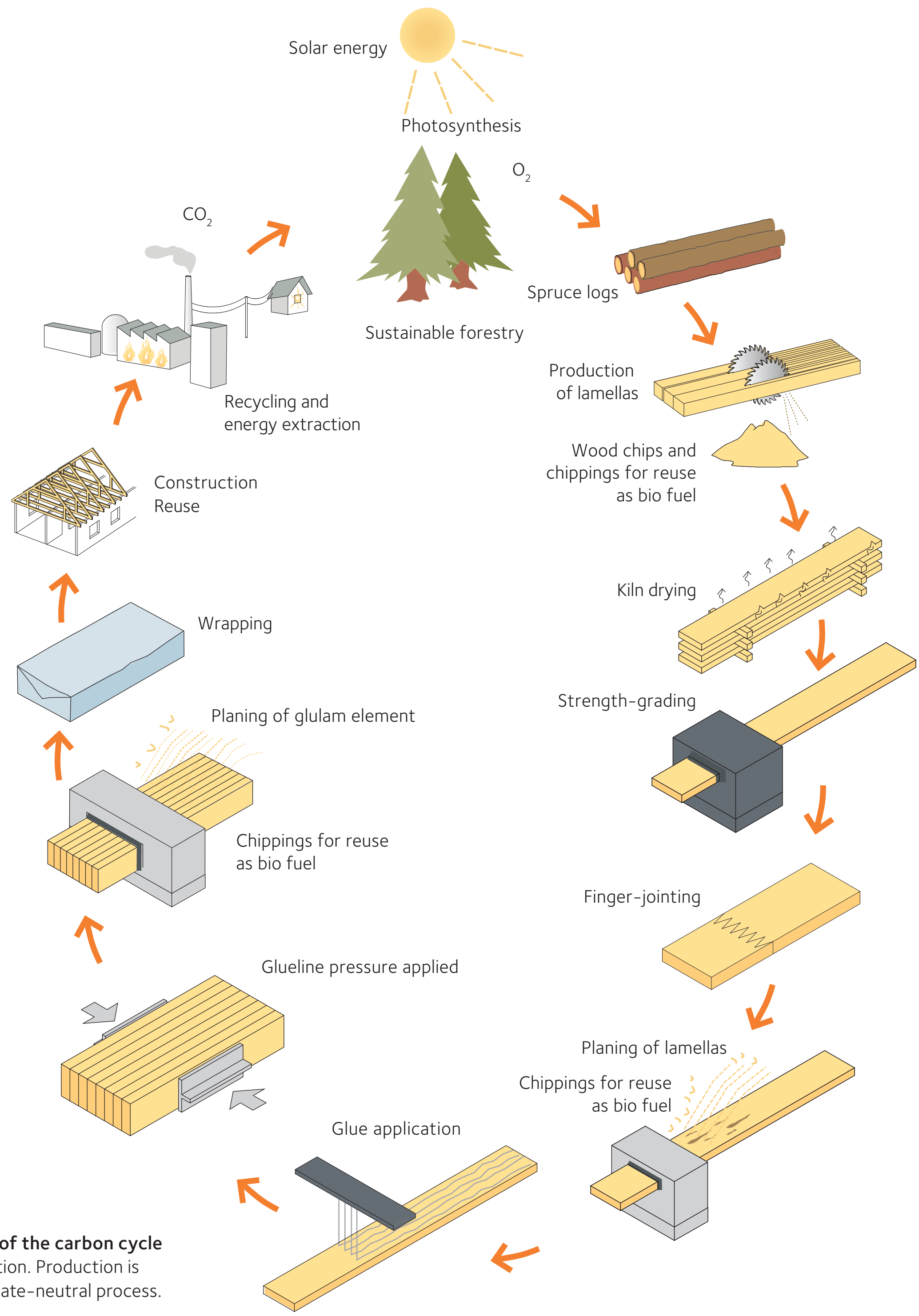


Figure 1.2 Glulam is part of the carbon cycle
Schematic of glulam production. Production is a resource-efficient and climate-neutral process.

1.3.1 Manufacturing process

Glulam production is carried out in much the same way regardless of manufacturer or country. *Figure 1.2*, schematically, a sketch of the manufacturing process.

The raw material is strength-graded timber, usually spruce in the Nordic countries. For construction works likely to be exposed to severe moisture conditions, impregnated (pressure treated) pine is also used. In rare cases, where for aesthetic reasons other timber species are needed, species like birch and larch have been used.

Normally, dried and strength-graded timber is supplied directly from the sawmill. The moisture content in the lamellas must be 6 – 15 % when they are glued together and the difference in moisture content between adjacent lamellas must not exceed 5 %. The strength of the glueline will then be optimal and the moisture content in the finished structure will be balanced, avoiding troublesome splitting and reducing the risk of distortion. Some fissures will always occur in the timber, but this generally has no adverse effects on the load-bearing capacity of the structure.

The cross-section of the glulam can be built up of lamellas of the same strength class, referred to as “homogeneous glulam”. It is, however, customary to use timber of higher quality in the outer laminations of a beam cross-section, where stresses are typically highest; this type of lay-up is referred to as “combined glulam”, *see figure 1.3*. In the factory it is therefore necessary to have space to store at least two strength classes of timber at the same time.

The individual lamellas are finger-jointed to produce lengths of approximately 30 – 40 m. These 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 laminations. To reduce the build-up of stresses during drying and moistening of the cross-section, the lamellas are turned so that the pith sides face the same direction throughout the cross-section. The outermost lamellas are, however, always turned with the pith side outwards.

The adhesive in the finger-joints is allowed to harden, depending on the adhesive system being used, for some hours before the flat sides of the lamellas are planed and then immediately glued.

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

When the adhesive has hardened, the pressure is released and the glulam components are lifted from the benches to a planing 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 arises, hole drilling and pre-drilling for connectors. In some cases the 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.

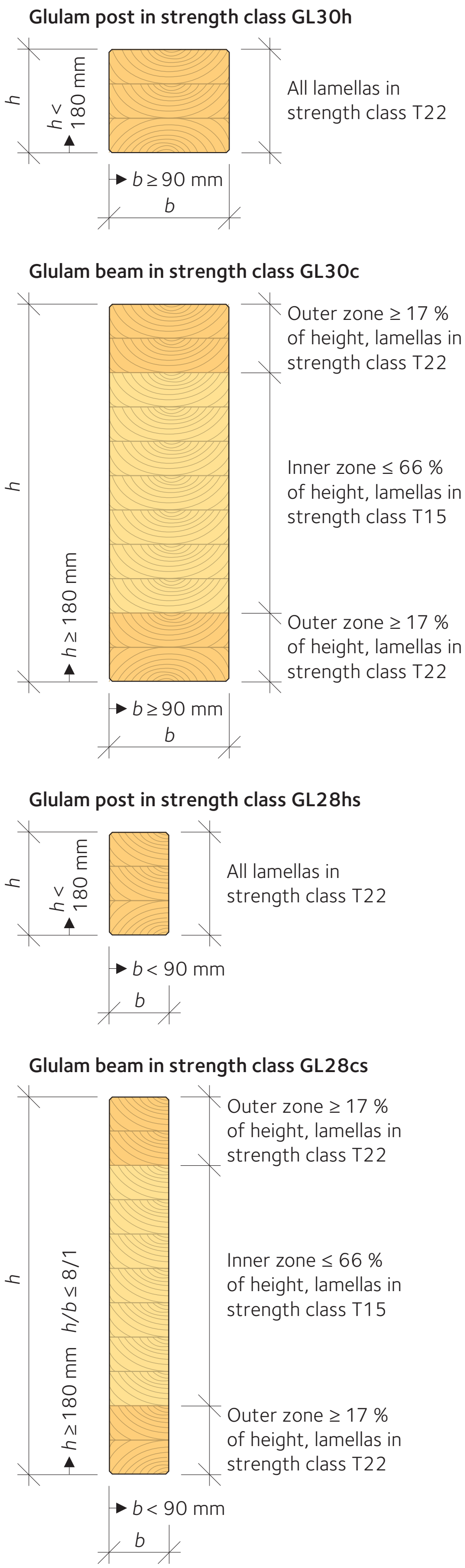


Figure 1.3 Strength classes for glulam
Glulam cross-sections illustrating the different strength classes GL30h (homogeneous glulam), GL30c (combined glulam), GL28hs (homogeneous split glulam) and GL28cs (combined split glulam). Glulam in strength classes GL28hs and GL28cs is made by splitting a wider cross-section into smaller pieces.

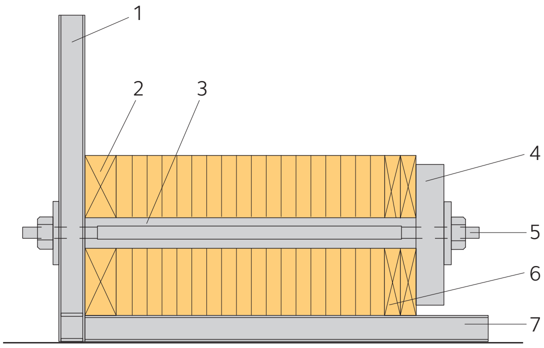


Figure 1.4 **Schematic section through gluing bench**
1. Vertical stop. 2. Pressure distributing base, possible camber pattern. 3. Distance piece. 4. Compression block. 5. Tension rod. 6. Pressure distributing boards. 7. Horizontal stop.

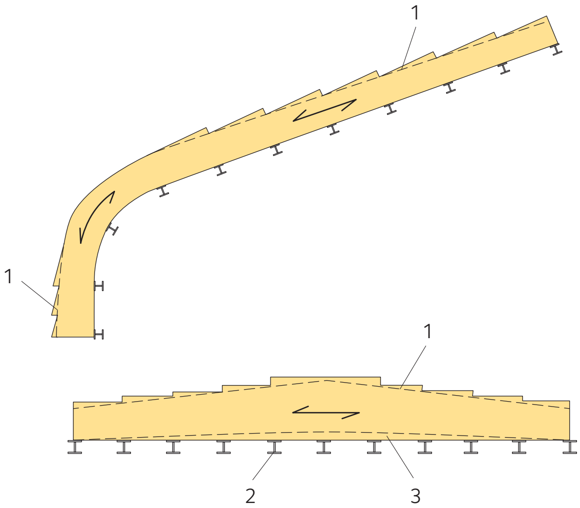
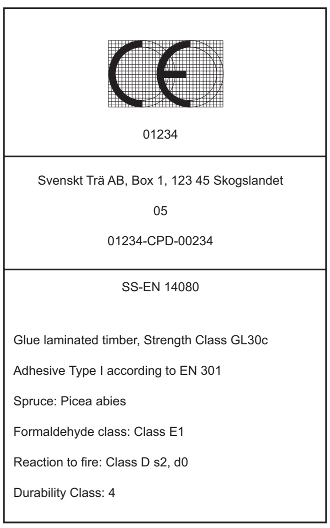


Figure 1.5 **Structural component with varying sectional height**
1. Trimming. 2. Vertical stop. 3. Pressure distributing base, possible camber jig.



The CE mark is a manufacturer's declaration that the product meets all the requirements placed on it by the relevant EU directives.

An example of CE-marking of a glulam element, according to EN 14080.

Figure 1.6 **The CE mark**

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

The most critical parameter in glulam production is the quality of the adhesive bonds. Thus, special emphasis is put on the strength of finger-joints and on the quality of gluelines in between the lamellas in continuous internal factory production control. The internal quality control process is monitored by an external control body accredited by authorities.

1.3.2 Certified glulam – CE-marking

The harmonised European standard EN 14080 and the standards referenced by EN 14080 lay down the requirements of CE-marked glulam. The purpose of CE-marking is to make possible the free trade of products within the European market and to make sure that products from different manufacturers can be compared by means of a number of declared properties (e.g. bending strength, modulus of elasticity, reaction to fire etc). EN 14080 thus defines what properties must be declared by the manufacturer, general principles to be used in the manufacturing, requirements on incoming materials and equipment, and requirements on the finished product. In order to be able to follow the provisions of EN 14080 the manufacturer must establish, document and maintain a factory production control system (FPC system) to ensure that the products placed on the market comply with the declared performance of the characteristics.

The FPC system consists of procedures, inspections and tests and/or assessments and the use of the results to control incoming materials, components, equipment, the production process and the product itself. The FPC adopted by the manufacturer shall be documented in a systematic manner in the form of written policies and procedures.

Before placing a product on the market, initial type testing (ITT), initial inspection of the factory and inspection of the FPC, is undertaken by an external auditing body. The ITT aims at assessing the product in order to ensure its compliance with the properties to be declared by the manufacturer and to the general requirements of EN 14080. The initial inspection of the factory and of the FPC is performed in order to ensure that the FPC includes all relevant elements, and that these are implemented in practice.

If all requirements are met after the ITT and the initial inspections, the manufacturer can CE-mark the products. However, continuous surveillance of the FPC is undertaken twice per year, usually unannounced. The purpose of this continuous surveillance is to make sure that the FPC is followed and relevant for the current production. For example any possible changes in production shall be evaluated, inspection regarding maintenance of equipment and tools shall be performed and records of tests and measurements made by the manufacturer shall be reviewed to ensure that the values obtained still correspond with the values for the samples submitted to Initial Type Testing.

1.3.3 Strength and stiffness

Glulam behaves in the same manner as ordinary structural timber in terms of various strength characteristics:

- 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.

In comparison with a corresponding component made out of structural timber, glulam components have, however, higher average strength and a smaller variability of strength. This so-called laminating effect is usually explained as follows:

The strength of the weakest cross-section — usually at a knot, finger joint or similar is critical to the strength of structural timber. The difference between boards is therefore considerable. In a glulam beam, however, lamellas with differing strengths are mixed, so that the risk of several lamellas with major flaws occuring in the same beam cross-section is minimal.

For glulam beams that are tested under laboratory conditions, i.e. short-term loading and a moisture content of about 12 %, the failures are very brittle and are 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 final failure, though without changing its brittle character. A brittle failure means, amongst other things, that a redistribution of stress does not occur during the failure process and the load-bearing capacity is reached, 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 increasing volume, the strength of large beams tends to be lower than that of small beams. This "volume effect" (size effect or Weibull 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.

The basis for design in e.g. Eurocode 5 is the characteristic strength and stiffness of the glulam. Such characteristic values represent formal values that are fulfilled by a certain percentage of a large population of e.g. glulam beams. As an example, the estimate of the characteristic strength is based on the use of a frequency diagram for the ultimate strength, *see figure 1.9*. Based on such test data it is then possible, with an acceptable level of accuracy, to adapt a statistical distribution — a normal or a lognormal distribution — to the resulting frequency diagram, or at least to its central part.

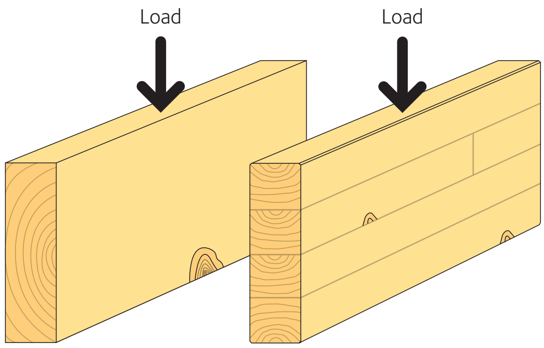


Figure 1.7 The lamination effect
The effect of any timber defects is evened out in glulam. There is very little risk that defects, such as large knots across multiple layers, will end up in the same cross-section. In a single plank, just one knot can significantly damage its strength.

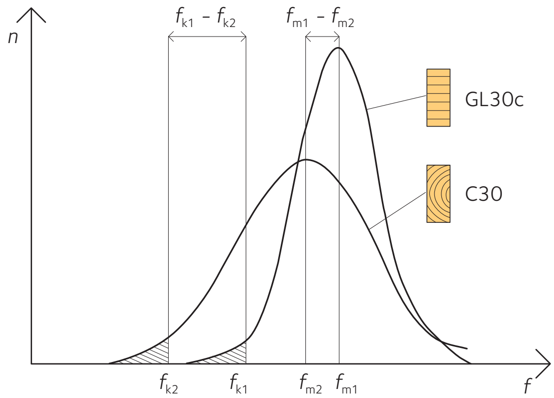


Figure 1.8 Structural members of glulam have a higher average strength and less variation in strength properties than the equivalent section of constructional timber
$f_{k1} - f_{k2}$ = the difference in characteristic strength value.
$f_{m1} - f_{m2}$ = the difference in the strength's mean value.
n = number of samples.
f = strength.
The illustration refers to glulam with a large number of laminations.

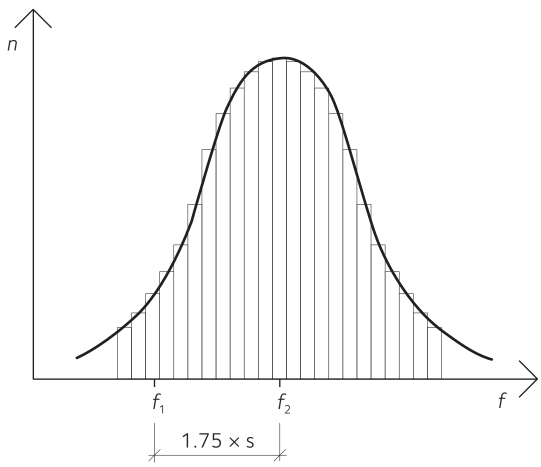


Figure 1.9 Example of a frequency diagram with superimposed normal distribution curve

If, for simplicity, it is assumed that the ultimate strength is normally distributed[1], it is possible to calculate the characteristic value f_k using the formula:

1.1 $$f_{\text{k}} = f_{\text{mean}} - c \times s$$

where f_{mean} is the mean and s is the standard deviation of strength. The parameter c is a coefficient whose value depends on how the characteristic value is defined (i.e. what percentile), on the number of data points used in the estimation of the statistical distribution and on whether the standard deviation used in the calculation is known or an estimate. The standard deviation is a statistical measure of the variability of 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 variability of the material and on the number of tests performed. In structural design situations the strength value to be used is based on the value of the lower 5 % fractile, i.e. the value below which, statistically, five of the actual values out of 100 will fall. If the number of tests is large, $c = 1.75$ in this case.

Characteristic stiffness for deformation analyses in the serviceability limit state (modulus of elasticity, shear modulus) is calculated in the same way, but using the mean instead of the 5 % fractile as the starting point. For stiffness values to be used in strength analyses, e.g. for cases where elastic buckling is of concern, the 5 % fractile value is used.

Ice hockey training rink, Skellefteå, Sweden.

1.3.4 Strength classes

Glulam that has been manufactured in accordance with EN 14080 is assigned a strength class. EN 14080 allows glulam to be assigned to a strength class in several ways:

a) based on calculation (i.e. calculating the glulam properties based on the lamination properties),
b) based on testing of beams or
c) by direct assignment into a strength class as described in EN 14080.

In general terms, glulam strength is determined by the strength of the timber used, its position in the cross-section and the strength of the finger-joints. In EN 14080 a number of pre-defined glulam strength classes are given, but the above-mentioned methods a) and b) can be used by the manufacturer to define a unique strength class in order to e.g. optimise the yield of the raw material locally available. *Table 1.1* and *table 1.2* give the strength and stiffness data for the glulam strength classes pre-defined in EN 14080[2]. *Table 1.3* defines the equations to be used in order to define a unique strength profile.

[1] For glulam, lognormal distribution should generally be used according to EN 14358. This does not change the basic discussion of characteristic values given here.

[2] The values follow EN 14080:2013, a harmonised standard for glulam.

Table 1.1 Glulam strength classes defined in EN 14080. Combined glulam.
Characteristic strength and stiffness values in MPa and densities in kg/m³ for combined glulam.
The manufacturer can also define a unique strength class, so as to maximise the yield of the available material.

Property	Symbol	Glulam strength class						
		GL20c	GL22c	GL24c	GL26c	GL28c	GL30c	GL32c
Bending strength	$f_{m,g,k}$	20	22	24	26	28	30	32
Tensile strength	$f_{t,0,g,k}$	15.0	16.0	17.0	19.0	19.5	19.5	19.5
	$f_{t,90,g,k}$	0.5						
Compression strength	$f_{c,0,g,k}$	18.5	20.0	21.5	23.5	24.0	24.5	24.5
	$f_{c,90,g,k}$	2.5						
Shear strength (shear and torsion)	$f_{v,g,k}$	3.5						
Rolling shear strength	$f_{r,g,k}$	1.2						
Modulus of elasticity	$E_{0,g,mean}$	10,400	10,400	11,000	12,000	12,500	13,000	13,500
	$E_{0,g,05}$	8,600	8,600	9,100	10,000	10,400	10,800	11,200
	$E_{90,g,mean}$	300						
	$E_{90,g,05}$	250						
Shear modulus	$G_{g,mean}$	650						
	$G_{g,05}$	540						
Rolling shear modulus	$G_{r,g,mean}$	65						
	$G_{r,g,05}$	54						
Density	$\rho_{g,k}$	355	355	365	385	390	390	400
	$\rho_{g,mean}$	390	390	400	420	420	430	440

Table 1.2 Glulam strength classes defined in EN 14080. Homogeneous glulam.
Characteristic strength and stiffness values in MPa and densities in kg/m³ for homogeneous glulam.
The manufacturer can also define a unique strength class, so as to maximise the yield of the available raw material.

Property	Symbol	Glulam strength class						
		GL20h	GL22h	GL24h	GL26h	GL28h	GL30h	GL32h
Bending strength	$f_{m,g,k}$	20	22	24	26	28	30	32
Tensile strength	$f_{t,0,g,k}$	16.0	17.6	19.2	20.8	22.4	24.0	25.6
	$f_{t,90,g,k}$	0.5						
Compression strength	$f_{c,0,g,k}$	20	22	24	26	28	30	32
	$f_{c,90,g,k}$	2.5						
Shear strength (shear and torsion)	$f_{v,g,k}$	3.5						
Rolling shear strength	$f_{r,g,k}$	1.2						
Modulus of elasticity	$E_{0,g,mean}$	8,400	10,500	11,500	12,100	12,600	13,600	14,200
	$E_{0,g,05}$	7,000	8,800	9,600	10,100	10,500	11,300	11,800
	$E_{90,g,mean}$	300						
	$E_{90,g,05}$	250						
Shear modulus	$G_{g,mean}$	650						
	$G_{g,05}$	540						
Rolling shear modulus	$G_{r,g,mean}$	65						
	$G_{r,g,05}$	54						
Density	$\rho_{g,k}$	340	370	385	405	425	430	440
	$\rho_{g,mean}$	370	410	420	445	460	480	490

Table 1.3 Characteristic strength and stiffness properties in MPa and densities in kg/m³ of homogeneous glued laminated timber

Property	Symbol	Characteristic values
Bending strength	$f_{m,g,k}$	The characteristic bending strength shall be calculated using the following expression: $f_{m,g,k} = -2.2 + 2.5 f_{t,0,1,k}^{0.75} + 1.5\left(f_{m,j,k}/1.4 - f_{t,0,1,k} + 6\right)^{0.65}$ The expression shall only be used for a characteristic flatwise bending strength of the finger joint in the range: $1.4 f_{t,0,1,k} \leq f_{m,j,k} \leq 1.4 f_{t,0,1,k} + 12$ The formula is also applicable to glulam without finger joints provided $f_{m,j,k}$ is taken as: $f_{m,j,k} = 1.4 f_{t,0,1,k} + 12$
Tensile strength	$f_{t,0,g,k}$	The characteristic tensile strength shall be taken as 80 percent of the characteristic values of the bending strength $f_{m,g,k}$
	$f_{t,90,g,k}$	0.5
Compression strength	$f_{c,0,g,k}$	The characteristic compression strength shall be taken as $f_{m,g,k}$ in N/mm² where $f_{m,g,k}$ is the characteristic bending strength of the glued laminated timber
	$f_{c,90,g,k}$	2.5
Shear strength	$f_{v,g,k}$	3.5
	$f_{r,g,k}$	1.2
Modulus of elasticity	$E_{0,g,mean}$	The mean modulus of elasticity shall be taken as $E_{0,g,mean} = 1.05\ E_{t,0,l,mean}$
	$E_{90,g,mean}$	300
Shear modulus	$G_{g,mean}$	650
	$G_{r,g,mean}$	65
Density	$\rho_{g,k}$	1.1 $\rho_{l,k}$
	$\rho_{g,mean}$	$\rho_{l,mean}$

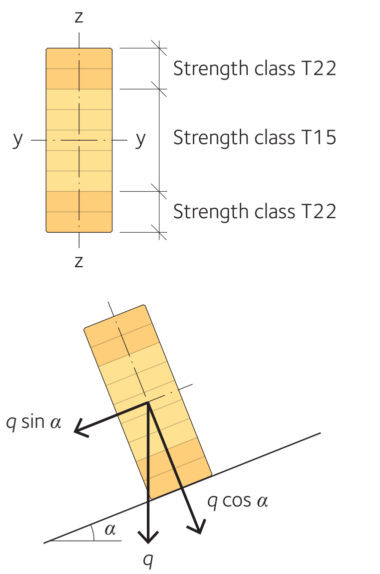


Figure 1.10 Skew bending of a glulam beam

Note that neither in EC5 nor in EN 14080 are there any strength or stiffness values given for members in bending about the weak axis, i.e. bending about an axis perpendicular to the gluelines of the cross section. This is problematic in cases were skew bending is involved. Consider the situation in *figure 1.10*.

According to EC5, the design criterion is given by:

$$\begin{aligned} \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \\ k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \end{aligned} \tag{1.2}$$

where $\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are the design bending stress values with respect to the major axes and $f_{m,y,d}$ and $f_{m,z,d}$ are the corresponding strength values. The factor k_m for glulam is 0.7.

In Eurocode 5 and EN 14080, only the value for $f_{m,y,d}$ is specified. This is not a problem if homogeneous glulam of the same strength class is used for all constituent lamellas. However, there is a debate to be had about how this should be approached in practice for combined cross-sections with different strength classes for the constituent lamellas, *see also figure 1.3, page 13*. One approach can be to proportion the share of lamellas in each strength class to obtain a weighted value for $f_{m,z,d}$. Another way may be to use the system factor k_{sys} according to *Eurocode 5, section 6.6*. However, the arguments below support the conclusion that it is appropriate to apply the same strength value in both directions, i.e. $f_{m,z,d} = f_{m,y,d}$, without compromising safety.

In the case of skew bending as shown in *figure 1.10*, the outer lamellas will be under the most stress and therefore the strength value $f_{m,z,d} = f_{m,y,d}$ can be applied for both the force component $q \sin \alpha$ and $q \cos \alpha$, i.e. the cross-section is considered as a homogeneous cross-section in terms of stress distribution.

In the case of plane bending, or bowing, the following argument can prove that $f_{m,z,d} = f_{m,y,d}$. A significant system effect according to *Eurocode 5, section 6.6* is relevant due to parallel connected lamellas. The finger-joints, which are the weak link in many bending failures around the stiff axis, are also in this case scattered in different positions along the length of the beam and not placed across the entire underside, where a failure usually occurs. In sum, this actually gives an increase in strength, even though the inner lamellas have a lower initial strength value of $f_{m,z,d}$.

Mountain station, Idre, Sweden.

1.3.5 Re-sawn glulam

Glulam may be sawn perpendicular to the glue lines into 2 or 3 parts (re-sawn glulam).

It should be noted re-sawn glulam is treated in a special manner in the standard EN 14080. Narrow glulam beams, i.e. beams with a width less than 90 mm, are typically made by resawing wider cross-sections using a band saw. By doing so, the standard EN 14080, specifies rules for possible downgrading of the glulam as a result of this splitting procedure. According to EN 14080, each part shall have a minimum width b = 38 mm and a maximum depth to width ratio of $h/b \leq 8$. Depending on the grading procedure and the lay-up, the characteristic strength properties of the re-sawn glulam shall be determined by either method a) or b), as follows:

a) If the grading procedures reliably ensure that all lamellas of the re-sawn glulam meet the declared properties, the strength, stiffness and density properties of the re-sawn glulam shall be determined from these declared properties of the lamellas.

b) If the following two requirements are fulfilled, then the characteristic bending strength $f_{m,s,k}$ of the re-sawn glulam in bending shall be determined from the characteristic bending strength $f_{m,g,k}$ of the full-size glulam by *equation 1.3* or *1.4*:
   - The characteristic tensile strength of the lamella is at least 18 N/mm² and maximum 30 N/mm²

and
   - The characteristic tensile strength of the inner lamellas is not more than 8 N/mm² smaller than the characteristic tensile strength of the outer lamellas.

$$f_{m,s,k} = f_{m,g,k} - \frac{96}{f_{t,0,1,k} - 6} + 4 \quad [\text{MPa}] \qquad \text{for one cut} \qquad (1.3)$$

$$f_{m,s,k} = f_{m,g,k} - \frac{96}{f_{t,0,1,k} - 6} \quad [\text{MPa}] \qquad \text{for two cuts} \qquad (1.4)$$

where:

$f_{m,s,k}$ is the characteristic bending strength of the re-sawn glulam
$f_{m,g,k}$ is the characteristic bending strength of glulam before it has been re-sawn
$f_{t,0,l,k}$ is the characteristic tensile strength of the outer lamellas.

Table 1.4 **Characteristic strength and stiffness values in MPa and densities in kg/m³ for re-sawn glulam** (based on strength classes GL30c and GL30h).

Strength values in MPa		GL28cs	GL28hs
Bending parallel to grain	$f_{m,g,k}$	28.0	28.0
Tension parallel to grain	$f_{t,0,g,k}$	19.5	22.4
Tension perpendicular to grain	$f_{t,90,g,k}$	0.5	0.5
Compression parallel to grain	$f_{c,0,g,k}$	24.0	28.0
Compression perpendicular to grain	$f_{c,90,g,k}$	2.5	2.5
Shear (shear and torsion)	$f_{v,g,k}$	3.5	3.5
Rolling shear	$f_{r,g,k}$	1.2	1.2
Stiffness values in MPa		**GL28cs**	**GL28hs**
Modulus of elasticity parallel to grain	$E_{0,g,mean}$	12,500	13,100
Modulus of elasticity, characteristic	$E_{0,g,05}$	10,400	10,500
Modulus of elasticity perpendicular to grain	$E_{90,g,mean}$	300	300
Shear modulus	$G_{g,mean}$	650	650
Density in kg/m³		**GL28cs**	**GL28hs**
Density, characteristic	$\rho_{g,k}$	390	430
Density	$\rho_{g,mean}$	430	480

Manufacture of curved glulam elements.

The mean modulus of elasticity parallel to the grain $E_{0,s,mean}$ of the re-sawn glulam shall be determined from the mean modulus of elasticity $E_{0,g,mean}$ of the full-size glulam from *equation 1.5*:

1.5 $$E_{0,s,mean} = E_{0,g,mean} - 500 \quad [\text{N/mm}^2]$$

The other mechanical properties shall be determined from *table 1.3, page 18*. For combined split glulam, the table gives unreasonable values. Instead, choose values for the corresponding combined unsplit strength class and $f_{m,s,k}$ and $E_{0,s,mean}$ in line with *equations 1.3 – 1.5*. The density values are equal to those of the full-size glulam. If, however, the grading procedure used in grading the lamellas is such that it assures also the properties of the split laminations, no downgrading needs to be adopted. *Table 1.4, page 19*, presents the key material properties for the most commonly used split strength classes in Sweden.

1.3.6 Adhesives

The adhesives used have well documented properties both in terms of strength and durability under long-term loading. Only adhesives with a long-term history of practical experience are used. The formal requirements are given in EN 14080 and sub-standards EN 301 and EN 302.

Earlier, mostly synthetic two-component adhesives of PRF type (phenol-resorcinol-formaldehyde) were used in glulam production. All PRFs used for glulam production belong to so-called type I adhesives, and are approved for use in any service class, i.e. both in- and outdoors. PRF adhesive gives dark red-brown joints.

MUF adhesives (melamine-urea-formaldehyde) are the most common adhesives nowadays. This adhesive type gives strong and durable bonds, appropriate for any service class. Fresh melamine-glued joints are light but will darken with time.

For finger-jointing the lamellas, the lighter melamine is typically used. The marking on the glulam must show which type of glue has been used (I or II in accordance with EN 14080).

Continuous development is taking place in the field of adhesives and new types are being introduced. Thus, one-component polyurethanes can also be used (service class 1–2).

In addition to the adhesives used in the manufacturing of the glulam itself, adhesives are also used in the production of joints (bonded-in steel plates and rods) or in the case of repairs of old structures, on site. In these applications polyurethanes or epoxies are commonly used.

1.4 Glulam components – sizes and shapes

Glulam technology makes it possible to vary the cross-sectional shape, 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 requirements for transportation, etc. Some of these limiting factors are commented on below.

Rectangular cross-sections are the most common, but other cross-sections can be manufactured, e.g. I-, T- and L-sections, hollow sections, or combinations of rectangular sections block-glued, *see figure 1.11*.

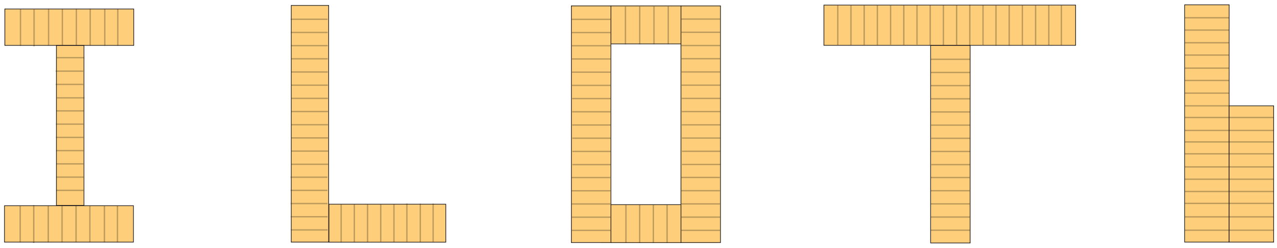


Figure 1.11 The most common cross-section for glulam is rectangular. Examples of other cross-sectional shapes are shown here.

1.4.1 Production standard – stock sizes

Straight glulam components of rectangular cross-section are normally made of 45 mm or 33 mm (treated pine) thick lamellas, in widths corresponding to the sawmills' standard range. After planing of the sides the finished standard widths become 90, 115, 140, 165, 190 and 215 mm. The exact sizes used depend on whether the sides are to be planed and sanded or only planed, in which case occasional patches of unplaned sides are accepted.

Table 1.5 shows sizes in accordance with the established industry standard, with cross-sectional dimensions $b \times h$ and lengths up to 12 m. The sizes apply to components with appearance class Clean planed surfaces (*see section 1.5.3, page 23*) at a reference moisture content of 12 %.

Note that the dimensions given are nominal and apply at a reference moisture content of 12 % with appearance grade Clean planed, no surface repairs.

Other measurements and dimensions may occur. Other dimensions can be ordered (for straight glulam, the height measurement is a multiple of 45 mm).

Table 1.5 Glulam cross-sections for straight members
Nominal sizes corresponding to appearance class. Clean planed surfaces. Stock sizes. Adhesive type I. Four planed sides.

GL28c $b \times h$ (mm)	GL30c $b \times h$ (mm)	GL30h $b \times h$ (mm)
42 × 180	**90** × 180	**90** × 90
× 225	× 225	**115** × 115
× 270	× 270	**140** × 135
56 × 225	× 315	× 140
× 270	× 360	**160** × 160
66 × 270	× 405	**165** × 165
× 315	× 450	
	115 × 180	
	× 225	
	× 270	
	× 315	
	× 360	
	× 405	
	× 450	
	× 495	
	× 630	
	140 × 225	
	× 270	
	× 315	
	× 360	
	× 405	

1.4.2 Maximum cross-sectional sizes

Maximum width is restricted by the simple fact that it is difficult to obtain sawn timber wider than 225 mm, but in some cases it is possible to obtain timber with a width of up to 250 mm. After planing, this corresponds to a nominal width of 215 mm and 240 mm respectively. Components with up to 700 – 800 mm width (e.g. 190 + 190 + 190 + 190 = 760 mm) may be made by block-gluing together components. Another way of obtaining large width beams is by edge-gluing the lamellas prior to forming the cross-section.

Maximum depth is limited to about 2 m by the availability of planing equipment. Larger depths can be achieved by various methods, 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.

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.

1.4.3 Maximum length

In the Nordic countries there are glulam factories that produce up to 40 m long components. In practice, however, length is restricted by transport considerations, *see section 1.6.1, page 25*.

Storage of glulam products.

Table 1.6 Size tolerances for glulam according to EN 14080

Size tolerances for glulam		
Size of *b*		± 2 mm
Size of *h*	≤ 400 mm > 400 mm	+ 4 mm to – 2 mm +1 % to – 0.5 %
Size of *L*	≤ 2.0 m > 2.0 ≤ 20 m > 20 m	± 2 mm ± 0.1 % ± 20 mm
Angles	Cross-sectional angles can deviate at most 1:50 (circa 1°) from the right angles.	
Straightness (for straight elements)	From two arbitrarily chosen points 2 m apart, on any edge of the glulam element, the deviation must not exceed 4 mm. Cambered beams are excluded.	
Maximum deviation from the nominal arc (over 1 m along the curved shape)	± 4 mm (≤ 6 laminations) ± 2 mm (> 6 laminations)	

Bridge, Ringsjöstrand, Sweden.

1.4.4 Camber

Simply supported glulam beams with longer spans (exceeding 10 – 12 m) may need to be cambered to reduce any problems caused by deflections. A moderate camber of up to 200 mm can easily be arranged during production. Some recommendations on the size of the camber are given in *section 3.3, page 42*.

1.4.5 Permitted deviations

Glulam components are manufactured with the same accuracy as rolled steel sections or concrete components. Permitted deviations are given in EN 14080. *Table 1.6* gives some of the basic permitted deviations.

1.5 Appearance and surface finish

Glulam is first and foremost a structural material whose strength, stiffness and durability are, in general, its most important properties.

Glulam components do not usually have the timber quality and surface finish 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 site.

1.5.1 Timber

Glulam is manufactured from strength-graded timber, mainly spruce, which means a reduction in knot sizes, but not 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 lamella.

1.5.2 Glue joints

As mentioned above, phenol-resorcinol adhesives give dark joints and melamine adhesives give light joints. Melamine joints can, however, darken with time. Finger-joints in the lamellas therefore appear as dark patches or thin zig-zag lines on the sides of the components, especially if phenol-resorcinol glue has been used.

Glulam components narrower than approx. 90 mm are normally sawn from wider components. The saw cut 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. Re-sawn components should therefore be avoided if appearance requirements are high.

1.5.3 Surfaces

When glulam components are lifted from the gluing tables the sides are uneven and marred by excess glue that 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 structural engineer should agree how much working is suitable in each case.

1.5.4 Surface treatment

The purpose of surface treatment is to give the timber a certain desired appearance and to protect the material from sudden moisture changes, thus avoiding splitting. Film-type finishes such as paint or lacquer also make it easier to keep the timber clean and give some protection against mechanical damage. Surface finishes can also be used to avoid flame spread and smoke in fires.

Glulam can be surface-treated with the same products and methods as ordinary timber, e.g. it can be stained, painted or lacquered. The technical, economical and aesthetic demands determine the choice in each particular case. 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 construction period so that harmful moistening and staining are avoided. In out-door sheltered conditions it is often sufficient to use a primer, e.g. 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 coatings may be required.

Outdoors, glulam is subjected 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 placed under a roof or by being covered by ventilated cladding, *see figure 1.13*.

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 that preserves the "white" appearance of fresh timber externally over the long term and with a reasonable amount of maintenance. If the natural ageing of timber is unacceptable in an outdoor 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 easier to maintain. In order that the surface does not become darker each time it is treated, maintenance of stained surface can be carried out using colourless or diluted stain.

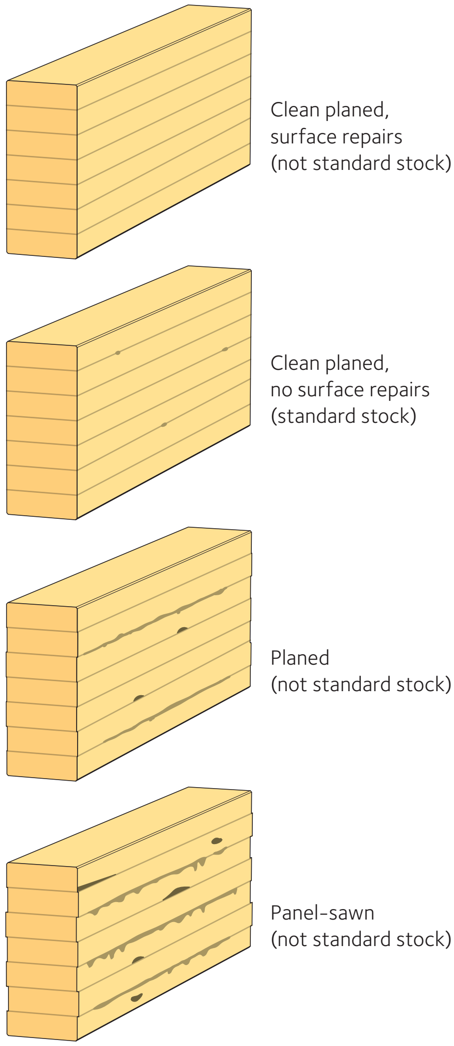


Figure 1.12 Appearance grades, examples. From the bottom up: Panel-sawn, Planed (industrial appearance grade) and Clean planed, no surface repairs or Clean planed, surface repairs (architectural appearance grade), *see also page 22-23 in The Glulam Handbook Volume 1.*

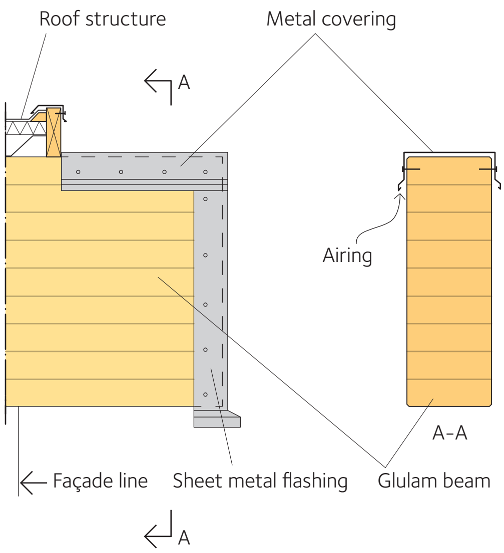


Figure 1.13 An example of protection of an external cantilevering beam

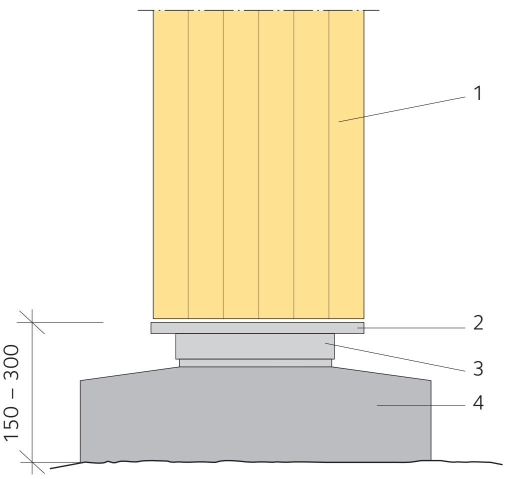


Figure 1.14 **Design for timber protection of a glulam column base**
1. Glulam column.
2. Moisture protection of e.g. 5 mm hard-board.
3. Spacing plate (steel, treated timber or concrete).
4. Concrete foundation.

Railway bridge, Munkedal, Sweden.

1.5.5 Constructional timber protection

Timber is an organic material, which can be attacked by fungi or destructive insects under certain circumstances. In each particular case this can seem to be a disadvantage, but is one of the major advantages of the material from an ecological point of view. However, the structure must be protected against such attacks during the lifetime of the building. This is done primarily by thorough detailing, using designs that ensure the conditions that produce rot do not arise.

This "constructional timber protection" is based on keeping the timber as dry as possible (moisture content less than 20 %). If this is not possible, the structure must be designed so that the timber can dry out after wetting. Dry timber does not rot. The use of properly designed details to prevent the uptake of moisture is of the utmost importance, and special care should be taken in designing joints and fastener details so that free water does not enter the timber (e.g. by limiting the amount of exposed end grain surfaces, avoiding small clearances which will give rise to capillary suction etc.). For some cases, e.g. in bridge construction, the easiest protection is obtained by covering the glulam completely with wooden panels or boards. These can then be easily replaced. In bridge construction it is also common to use covered pedestrian bridges, not only for increased comfort for the end-users, but also as a means of improving its environmental protection.

Another efficient method of protecting timber against rot is to pressure-treat the timber with suitable protective fluid. Pressure treatment can, however, never replace correct detailing 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 the law and in various standards.

Glulam can be manufactured from lamellas that have been 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 treated after gluing, but the component size is then limited by the size of the treatment equipment. Due to of the risk of splitting, treatment should be carried out using oil-based products. The use of creosote products should, for reasons of workers' safety, be used with great care.

1.5.6 Protection during transport, storage and erection

Glulam products are usually packaged individually by the glulam manufacturers using a recyclable material. The packaging is intended to protect against moisture, precipitation, solar radiation, dirt and some mechanical damage during transportation, storage and even during assembly.

The wrapping is not a reliable protection against moisture. Indeed, under adverse circumstances moisture can condense on the inside of the wrapping and must then be drained by opening the wrapping. It is thus advantageous in many cases to unwrap the glulam as soon as possible after erection of the building (provided of course that the roofing is in place and that a reasonable climate in the building is guaranteed). This will minimise the risk of moisture staining of the glulam surface, which of course should be avoided if naked wooden surfaces will appear in the finished building. Find out more in *The Glulam Handbook Volume 1, Handling glulam correctly*.

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 300 mm deep that 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 outdoors, 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 reach the ground.
- Avoid long-term storage on the site, especially outdoors.

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, wide straps should be used and the edges of the glulam should be protected with e.g. metal angles to avoid lifting marks. Working gloves, straps and other lifting equipment must be free from loose dirt. Do not walk on surfaces, which will be visible after erection.

1.6 Transportation and erection

Transportation and erection are the last operations in the building of a glulam structure and 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.

1.6.1 Transportation

Transportation 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 into shorter sections transportable on a normal lorry.

Long vehicles require special permission from the authorities. Components up to 25 – 30 m are usually no problem. The greatest length without special permission vary between different countries, but is normally around 24 m. Special transport is usually required if the width exceeds 2.6 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.

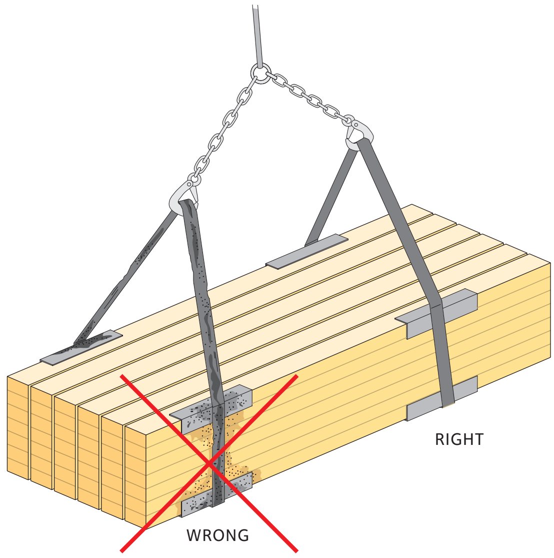


Figure 1.15 Make sure that building gloves, straps and other lift equipment are free from dirt

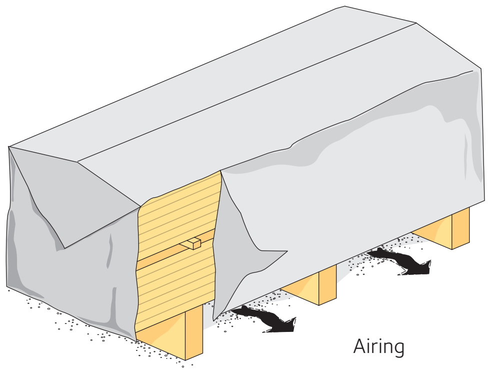


Figure 1.16 Storage of glulam
Example of how to give glulam short-term protection on the construction site.

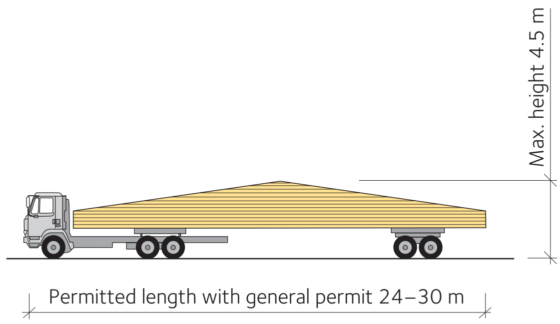


Figure 1.17 Transporting glulam
The maximum permitted length and height of vehicles without a special permit is the same across Europe. Transport rules may vary in detail from one country to another.

Farm building, Kålaboda, Sweden.

1.6.2 Erection

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

The best alternative is to lift the glulam component directly from the lorry to its place in the building. This is, however, seldom possible and it is generally necessary to plan temporary storage for some time on the site. The instructions in *section 1.5.6, page 24*, should then be followed.

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

It is important to have planned erection before the components are unloaded to avoid time-consuming re-loading. 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 structure or similar is erected.

Plastic wrappings should be cut open underneath the beams to avoid moisture forming inside the film. The wrapping can be removed completely, but the risk that visible elements become dirty during erection must always be borne in mind. Roof structures consisting of corrugated steel sheets laid directly on the beams are especially vulnerable, as water leaking from joints in the sheeting may discolour the sides of the beams before insulation and roofing felt are in place.

Three-pin frames and arches consist of two parts connected to concrete foundations or columns, 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 so that a half-frame or half-arch is lifted into place by the mobile crane. The foot or lower 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 that has been erected in the same way. As soon as bracing is completed the tower is moved to the next module and the process repeated.

1.7 Ordering and delivery

1.7.1 Specification

On drawings and in the specification, the following should be stated:

- Identification code (e.g. B1, P3 etc).
- Type of component (e.g. by referring to drawing).
- Nominal size (width × depth at left support/maximum depth/depth at right support × 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.
- Adhesive type (I or II in accordance with EN 14080).
- Surface finish (state if necessary which sides are visible).
- Surface treatment (if desired).
- Camber (if desired).
- Timber species (other than spruce, e.g. pressure treated pine).
- Permitted deviations, if different from the requirements of EN 14080.

Example of a specification:
4 no. double pitched beams 165 × 680 / 1370 / 680 × 22,000 mm, GL30c/type I, Clean 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 consists of two components).
- Possible reference to a 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.

1.7.2 Dimensioning

Clear, correct and unambiguous dimensioning of a glulam element promotes a well-made glulam construction and lessens the risk of errors and delays when completing an item. An example of a correct dimensioning is given in *figure 91 below*.

All required views must be drawn up and figured. Cut arrows ease comprehension. Meanwhile it is often enough to draw up the beam or column viewed from the side. The glulam element is best oriented horizontally or vertically on the finishing work drawing, in order to save on drawing space and simplify the figuring.

For measuring, start from the unfinished glulam element and include all the sizes in the x- and y-axis required to be able to finish the beam. It is best to start from the same point on the figuring of, for example, a notch, even if it takes up more drawing space. If holes are made at the site, hole diameters and possible notches plus reinforcings are to be set out. If there are slits or glued-on wooden parts these are dimensioned in a suitable way. Explanatory text can complement the figuring.

Modern 3D- drawing software normally generates 2D-drawings, which will need the figuring to be checked to ensure it is complete. If the 3D-models of glulam elements can be inserted in the finishing work drawing this simplifies the understanding with complicated processing.

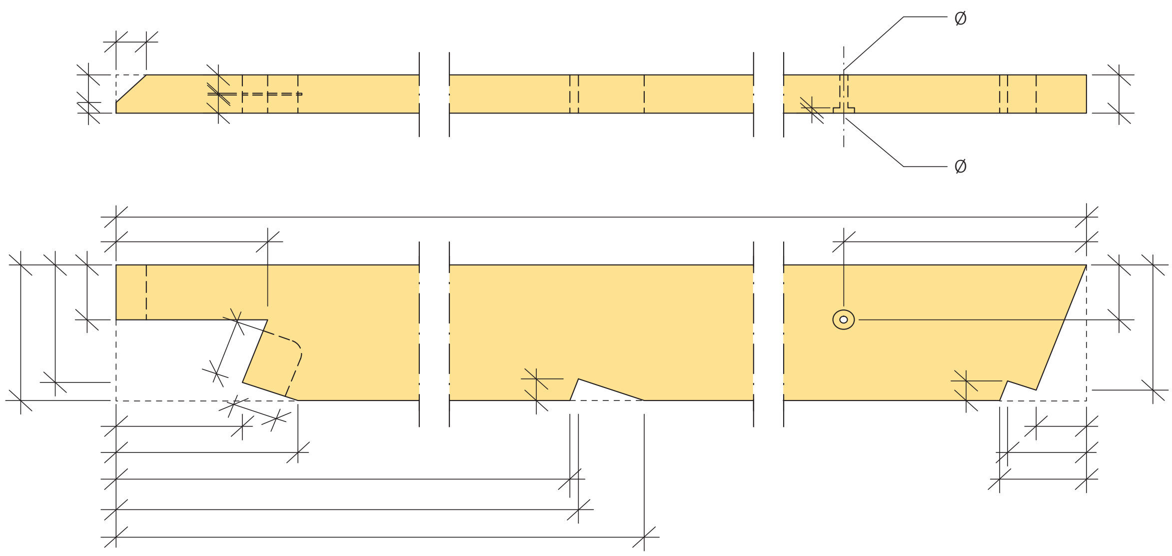


Figure 1.18 Examples of clear figuring for manufacture finishing work

Design methods for timber and glulam structures

2.1 General introduction to structural design

Structural design involves choosing the dimensions of load-bearing members with the primary aims of minimising the risk of a failure in the structure leading to serious personal injuries and ensuring the building functions satisfactorily in normal use. The load-bearing structure must be designed with regard to the requirements set for material resistance, performance and durability during the service life of the structure.

Structural design is based on verification. The aim of this verification is to show that the actual requirements are fulfilled for the chosen material, dimensions and structural system. This verification can be achieved by calculation or testing, or by a combination of the two.

The design working life is the period for which it is assumed a structure is going to be used for its intended purpose, with anticipated maintenance but without major repair being necessary. A design working life of 50 years for buildings and 100 years for bridges is often recommended.

2.1.1 Conceptual process and preliminary design

Conceptual design is the first stage of a building project. During this stage, needs are identified and examined, requirements for potential solutions are defined, potential solutions are evaluated and approved, a suitable structural concept for further design is developed. Once the concept has been chosen, well defined and approved, the second stage of the design process is reached by verification according to a required design code. The intuitive phase, which is part of the first stage, includes the definition of specific goals for the project, understanding the client's wishes and transforming them into a list of requirements, understanding the design criteria as specified by society via current design codes, generating simple concept solutions for various ideas and the preliminary evaluation of each idea.

The design requirements should be as clear as possible and should include issues such as performance, quality parameters, reliability, safety, product life span, aesthetics, ergonomics, economy and maintenance. The main aim of the systematic search is to generate a wide range of possible design concepts. A combination of intuitive and systematic methods is probably the best way to generate new concepts. Various ideas are created by the systematic search and are produced within the intuitive thinking process.

The main concern for a preliminary evaluation is to reduce the number of concepts. This can be achieved by discussions or by using preliminary evaluation matrices. It is important to document the reasons behind the choice of a solution so as to be able to backtrack to this step in the following process. The evaluation phase includes ranking different concepts, making a simplified risk analysis of each concept and making a final evaluation of concepts. Ranking can be done by weighing up objectives and various requirements in order to make some kind of systematic comparison of different concepts. A matrix showing the grades of various requirements in relation to different concepts is produced. The concept with the highest grade will obtain the highest ranking. A risk analysis is an important part of the conceptual design. All possible risks should be listed for two or three concepts with the highest ranking. These risks may include construction, transportation and production risks, economic risks, accidental risks or weather risks, for example.

The aim of preliminary sizing is to estimate the preliminary dimensions of the load-bearing members in a building, such as trusses, beams, columns, walls and floors.

The preliminary sizing often includes obtaining relevant load combinations in order to estimate sectional forces acting on a structural part that is being analysed. Estimations of section sizes are based on:

- Experience and recommendations.
- Preliminary design forces (including interaction).
- Preliminary design stresses.
- Preliminary deflection.

Bandy hall, Nässjö, Sweden.

2.1.2 Detailed design by verification

Outcomes and process

During the detailed design phase, all the key design decisions are finalised. As a result of the detailed design, the following outcomes are achieved:

- The structure is fully and unambiguously defined and validated.
- All major building systems are defined.
- All parts are fully engineered and co-ordinated.
- Costs and construction methods are established to a high level of precision.
- Agreement is reached on tolerances between the companies involved to ensure constructability, prefabrication and transportation.
- Quality levels are established.
- Prescriptive specifications are completed based on prescribed and completed systems.

The structural design process is regulated by the relevant design code, which applies to the geographical location where a structure is going to be built. In different countries, the design codes may be based on various principles, such as limit state design, reliability design or allowable stress design, for example. In most European countries, limit state design applies and all stages of this design process are defined in a set of Eurocodes. The same design code must be used when a structural design is produced for an entire structure.

Verification

Verification is the main part of structural design, and from about 2010 a common set of codes — Eurocodes — are used for design of construction work throughout the EU. The verification of the load-bearing capacity of a structure or part of it is conducted in the ultimate limit state, while the verification of whether it functions is conducted in the serviceability limit state. To verify a structure in

Nordens Ark zoo, Hunnebostrand, Sweden.

the ultimate limit state, a designer needs to create structural and load models by using design values for actions, material or product properties and geometrical data. Load cases should be selected by identifying load arrangements and possible deviations from assumed directions and positions of actions and sets of deformations and imperfections that should be considered simultaneously.

2.2 Applying the Eurocodes

2.2.1 Set of Eurocodes and general requirements

The Eurocodes were established by the European Commission to create a set of harmonised technical rules for the design of structures which should replace the national rules in the European member states. Once the Eurocodes apply, they have to be used together. The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990	Eurocode 0:	Basis of structural design
EN 1991	Eurocode 1:	Actions on structures
EN 1992	Eurocode 2:	Design of concrete structures
EN 1993	Eurocode 3:	Design of steel structures
EN 1994	Eurocode 4:	Design of composite steel and concrete structures
EN 1995	Eurocode 5:	Design of timber structures
EN 1996	Eurocode 6:	Design of masonry structures
EN 1997	Eurocode 7:	Geotechnical design
EN 1998	Eurocode 8:	Design of structures for earthquake resistance
EN 1999	Eurocode 9:	Design of aluminium structures.

All Eurocodes have two main notations throughout all standards. One notation has the suffix “P” indicating “Principle requirements”. This means that one must comply with the requirements following the suffix “P”. They are usually general statements and definitions for which there is no alternative. The second notation, where there is no “P” means “Application Rules”. These are generally recognised rules that comply with the “Principle requirements” where there can be other alternatives or solutions. Eurocode 0 describes the principles for limit state design and the requirements for safety, serviceability and durability of structures. The following general requirements with respect to the design process are specified in Eurocode 0:

- The choice of the structural system and the design of the structure is made by appropriately qualified and experienced personnel.
- Execution is carried out by personnel having the appropriate skill and experience.
- Adequate supervision and quality control is provided during execution of the work, i.e. in design offices, factories, plants, and on site.
- The construction materials and products are used as specified in the Eurocodes or in the relevant execution standards, or reference material or product specifications.
- The structure will be adequately maintained.
- The structure will be used in accordance with the design assumptions.

2.2.2 Design situations and verifications

To verify a structure or part of it, a designer has to distinguish between design situations, *see table 2.1*, agents, actions, combination of actions and their effects, *cf. figure 2.1*. Examples of agents are gravity, wind, snow, solar radiation, earthquakes and so on. Examples of actions are load, pressure, temperature, ground acceleration and so on. Combinations of actions are actions likely to occur simultaneously. Finally, effects on the structure from combinations of actions may be stress, internal force and moment, rotation, displacement and so on.

Eurocodes are based on limit state design. A limit state defines conditions beyond which the structure no longer satisfies the relevant performance requirements. These conditions are classified into ultimate and serviceability limit states. Ultimate limit states (ULS) relate to safety; states associated with collapse or with other forms of structural failure. Serviceability limit states (SLS) relate to those states in which the structure, although standing, behaves in an unsatisfactory fashion due to excessive deformation or vibration, for example. The verification or design procedure is illustrated in *figure 2.1*.

Design and detailing of glulam and timber structures are performed in accordance with Eurocode 5. However, it only applies together with the main Eurocode 0 and Eurocode 1, which deal with all types of load. Eurocode 5 consists of three parts:

EN 1995-1-1	Design of timber structures — Part 1-1: General — Common rules and rules for buildings
EN 1995-1-2	Design of timber structures — Part 1-2: General — Structural fire design
EN 1995-2	Design of timber structures — Part 2: Bridges.

The Eurocodes are open for national choice of certain design rules or values of functions. These Nationally Determined Parameters (NDPs) are given in the National Annex to the relevant Eurocode.

2.2.3 Principles for limit state design

Eurocode 0 describes two methods for limit state design:
- Reliability methods.
- The partial factor method.

The limit state design is based on a statistical approach, with an assessment of the probability of reaching a given limit state, and on establishing an acceptable maximum level of that probability for design purposes. Consider the special case when the limit state of failure can be described in terms of load effect E and resistance R so that failure will occur if $E > R$ (the so-called fundamental case). The load effect E usually corresponds to a maximum value during a reference period T (often taken as one year for time variable loads). Both E and R are random variables. The safety margin defined as $Z = R - E$ is also a random variable, so that the structure is regarded as safe if $Z > 0$.

Table 2.1 Design situations and their need for verification

Design situations		Verifications
Persistent	Normal use	ULS, SLS
Transient	Execution, temporary conditions applicable to the structure, e.g. maintenance or repair	ULS, SLS
Accidental	Normal use	ULS
	During execution	ULS
Seismic	Normal use	ULS, SLS
	During execution	ULS, SLS

Requirements

Serviceability Safety

Durability

Design Situations

Agents

Actions

Combinations of Actions

Effects

Ultimate Limit States Serviceability Limit States

Figure 2.1 Requirements and terminology used in the design procedure

If it is assumed that both R and E are normally distributed with mean values μ_R and μ_E, as well as standard deviations σ_R and σ_E, the probability of failure P_f can be expressed as in *equation 2.1*.

2.1 $$P_f = P(Z \leq 0) = \Phi\left(\frac{0-\mu_Z}{\sigma_Z}\right) = \Phi(-\beta)$$

where Φ is the standardised normal distribution function, $\mu_Z = \mu_R - \mu_E$ and $\sigma^2_Z = \sigma^2_R + \sigma^2_E$ and $\beta = \mu_Z/\sigma_Z$ is the so-called reliability index. The reliability index β is a measure of the safety level and gives an expression for how many standard deviations the failure zone ($Z \leq 0$) is away from the mean value of Z. The relation between β and P_f is given in *table 2.2*.

Swimming facility, Torsby, Sweden.

Table 2.2 Relationship between reliability index β and probability of failure P_f

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1.28	2.32	3.09	3.72	4.27	4.75	5.20

Structural codes based on limit state design such as Eurocode 0 usually define a formal safety level in terms of a minimum target reliability index β_{target} (or maximum permissible probability of failure). For Eurocode 0, the general target safety index is set to 4.7 for a reference period of one year and 3.8 if a reference period of 50 years is used. In national applications, many countries allow β_{target} to be a function of the expected consequences of failure, defined by reliability classes. This means, for example, that lower values for β_{target} may be used in cases where the consequences of a failure with respect to human life are regarded as small and vice versa. Reliability methods are in general not suitable for ordinary design of structures but may be used for code calibration purposes and comparisons of reliability levels of structures. Ordinary design is preferably carried out using the partial factor method.

2.2.4 Verification based on partial factor method

In normal engineering design, an evaluation based on a reliability index is not practical. Instead the so-called partial safety factor method is used. The method is also called the partial coefficient method. 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:

- Uncertainty in representative values of actions (γ_f).
- Uncertainty in material properties (γ_m).
- Consequences of failure (γ_d).

An important aspect of the partial safety factor method is the concept of characteristic value, which should be based on a clear statistical definition. The characteristic actions (commonly known as loads) are obtained from the set of Eurocodes 1. The design actions are obtained by multiplying the characteristic actions by partial safety factors γ. The value of a partial safety factor for both loads and material parameters depends on the definition of the characteristic value used. For time-variable actions (e.g. snow and wind), a typical definition of characteristic value Q_k is that the probability of exceeding Q_k should be 2 percent per year. This is the same as stating that Q_k should have a return period of 50 years, meaning that the load level Q_k is exceeded only once on average during a 50-year period.

Referring to the fundamental case in ultimate limit state introduced in *section 2.2.1, page 31*, it has to be verified that the design resistance R_d is larger than the design load effect E_d for the structural element considered, *see equation 2.2*.

$$R_d > E_d \tag{2.2}$$

This requirement has to be verified for each failure mode and a number of different load combinations depending on the design situation. The design value E_d of the load effect is determined on the basis of permanent actions G, time-variable actions Q and accidental actions. The design value G_d of permanent actions is determined by *equation 2.3*.

$$G_d = \gamma_G \times G_k \tag{2.3}$$

where γ_G is a partial safety factor for permanent load G and G_k is a characteristic value for permanent actions, typically defined as the 50 % quantile or the mean value.

Design values for variable actions are defined by *equation 2.4*.

$$Q_d = \gamma_Q \times Q_k \tag{2.4}$$

where γ_Q is a partial safety factor of variable action Q and Q_k is characteristic value of variable action Q, typically defined as the 98 % quantile of the distribution for annual maxima of Q.

The design value R_d for resistance is mainly determined on the basis of material parameters and dimensions. The design value f_d of a strength parameter f is determined from *equation 2.5*.

$$f_d = \eta \frac{f_k}{\gamma_m} \tag{2.5}$$

where γ_m is a partial safety factor for the material, f_k is a characteristic value for the material property, typically the 5 % quantile and η is a factor accounting for differences between in-situ conditions in the structure and the conditions in tests used to determine the property.

In the serviceability limit state the critical condition is usually formulated in such a way that the calculated deflection, vibration, slip in a joint and so on, 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 section 2.2.6, page 34*, a concept that does not exist in the Eurocodes.

Swimming facility, Lerum, Sweden.


Shopping center, Sälen, Sweden.

2.2.5 Load effects and load combinations

The term “load effects” covers, for example, internal forces or moments and also deflections caused by loads. As a basic rule, the structure is not designed for a single load but for a combination of loads – self-weight and snow load, for example. To obtain a design combination value for each load situation, each action should be considered in turn as a leading action (with its full value) and combined with the other actions considered with their combination values, described by a reduction of the characteristic value Q_k by factors ψ_0, ψ_1 and ψ_2 described as:

- **The combination value** ($\psi_0 Q_k$), used for the verification of ULS and for the characteristic combinations of irreversible SLS (consequence of actions exceeding the specified service requirements will remain when actions are removed).
- **The frequent value** ($\psi_1 Q_k$), used for the verification of ULS related to accidental actions and for the verification of reversible SLS. The frequent value is exceeded approximately 1 percent of the time.
- **The quasi-permanent value** ($\psi_2 Q_k$), used for the assessment of long-term effects of SLS, such as deflections or cracks, and for the representation of variable action in accidental combinations of ULS. Corresponds to the time average of the variable action Q.

The factor ψ_2 can also be regarded as a factor that converts loads with short-term duration to an equivalent permanent action in order to calculate the long-term deflection effected by creep.

EN 1990 defines load combination rules for the different design situations, shown in *table 2.1*. These rules define how permanent loads and variable loads should be combined to determine the load effect. The following general format is, for example, valid for design in persistent or transient design situations in the ultimate state, *equation 2.6*.

2.6
$$E_d = \sum_{j \geq 1} \gamma_{G,j} \times G_{k,j} + \gamma_{Q,1} \times Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \times \psi_{0,i} \times Q_{k,i}$$

where indices j and i denote the j^{th} permanent load component and the i^{th} variable load component respectively, and

- $Q_{k,1}$ is the characteristic value for the leading variable load.
- $\gamma_{Q,1}$ is the partial safety factor associated with $Q_{k,1}$.

2.2.6 Safety classes

The risk that failure in a structure will involve serious personal injuries differs for different types of building, depending on their use and for building elements depending on their function. As a result, the risk of personal injury is greater in the failure of a roof beam than in the failure of a stud in a partition wall and greater if the beam supports the roof of a sports hall than if it is in a porch.

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_d in the ultimate limit state. When the safety class is included in the general format of *equation 2.6*, it will influence the total design load effect E_d, *see equation 2.7*.

There are three safety classes: safety class 1 ($\gamma_d = 0.83$) with little risk to serious personal injuries; safety class 2 ($\gamma_d = 0.91$) with some risk to serious personal injuries; and safety class 3 ($\gamma_d = 1$) with large risk to serious personal injuries. In the serviceability limit state however no distinction is made between the safety classes. Assignment to safety classes is not practiced at present in the Eurocodes.

$$E_d = \gamma_d \sum_{j\geq 1} \gamma_{G,j} \times G_{k,j} + \gamma_d \times \gamma_{Q,1} \times Q_{k,1} + \gamma_d \sum_{i>1} \gamma_{Q,i} \times \psi_{0,i} \times Q_{k,i} \quad 2.7$$

2.3 Concepts used for limit state design of glulam and timber

As a building material, glulam differs from steel, reinforced concrete or other composites in a number of ways. Wood is a biological and natural material with highly variable properties. Wood is orthotropic, i.e. it has different properties in different directions and is hygroscopic, which means that it has a moisture content that constantly changes with changes in the relative humidity of the surrounding environment. Some of the characteristics that are specific when it comes to glulam design are:

- The moisture content in timber and service classes.
- The duration of load for various types of load.
- Partial factors for material properties and adjustments using various modification factors.
- Possible increase of the design values for bending and tension due to size effect.
- The difference in material response when the loads are applied in various directions in relation to the grain orientation of timber.

Swimming facility, Gislaved, Sweden.

2.3.1 Effect of moisture content and service classes

Moisture content and variations in moisture content play a significant role for all properties of glulam and all wood-based products. Moisture content affects both strength and stiffness. To incorporate this effect in design, three service classes have been defined in Eurocode 5. They are:

- **Service class 1** is characterised by a moisture content in the materials corresponding to a temperature of 20 °C and the relative humidity of the surrounding air only exceeding 65 % for a few weeks per year. The average moisture content in most softwood does not exceed 12 %.

  This includes for example:
  - Floors and trusses in unheated but ventilated attics above permanently heated rooms.
  - Columns and studs in external walls of 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 riding schools are generally included in this class.

- **Service class 2** is characterised by a moisture content in the materials corresponding to a temperature of 20 °C and the relative humidity of the surrounding air only exceeding 85 % for a few weeks per year. The average moisture content in most softwoods does not exceed 20 %.

  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 riding school and farm buildings.
  - Glulam structures covering poorly ventilated swimming pools.

- **Service class 3** is characterised by climatic conditions leading to higher moisture content than in service class 2. The average moisture content in most softwood exceeds 20 %.

  This includes for example:
  - Glulam construction in rooms or buildings where moisture is generated by processes or storage.
  - Structures that are not protected from damp or are in direct contact with the ground.

2.3.2 Load duration classes

Timber experiences a significant loss of strength over a period of time. In order to take account of the loss of strength, load duration classes were established to facilitate the design procedure. The duration classes cover a range of durations that may occur in practice and associated actions, *see table 2.3*. The influence of load duration on the strength capacity of glulam is taken into account by assigning a factor k_{mod} as a function of service class and one of the five load-duration classes in *table 2.4*. The load-duration factor k_{mod} is a reduction factor for the characteristic strength of glulam, timber and wood-based products, varying between 0.2 and 1.1. Only for an instantane-

Table 2.3 Definition of load duration classes

Load duration class	Period of time	Load examples
Permanent (P)	> 10 years	Self-weight
Long term (L)	6 months – 10 years	Storage goods
Medium term (M)	1 week – 6 months	Snow Imposed load residential
Short term (S)	< 1 week	Wind
Instantaneous (I)		Wind Impact, explosion Single concentrated load on roof

Source: Table according to EN 1995-1-1:2004, 2.3.1.2.

Table 2.4 Values of k_{mod} for glulam, laminated veneer lumber (LVL) and solid timber depend on service and load-duration classes

Material	Service class	Load duration class				
		Permanent	Long term	Medium term	Short term	Instantaneous
Glulam, LVL and timber	1	0.6	0.7	0.8	0.9	1.1
	2	0.6	0.7	0.8	0.9	1.1
	3	0.5	0.55	0.65	0.7	0.9

ous load and dry glulam (service class 1 and 2 in accordance with EN 14080), this factor is 1.1 and results in an increase in the characteristic strength.

The values of the modification factor k_{mod} are given in *table 2.4*. If the load combination consists of actions belonging to different load-duration classes, a value of k_{mod} should be chosen which belongs to the action with the shortest duration.

In a connection between wood materials with different values of k_{mod}, the strength modification factor can be determined as in *equation 2.8*.

$$k_{mod} = \sqrt{k_{mod,1} k_{mod,2}} \quad (2.8)$$

where $k_{mod,i}$ is the strength modification factor of the materials 1 and 2.

2.3.3 Design of glulam 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 for glulam. First, the characteristic value f_k is taken from one of the *tables 1.1 – 1.3* with current characteristic strength and stiffness properties in MPa and densities in kg/m^3 are published. Then the design value f_d for the relevant property is calculated in accordance with *equation 2.5*. For glulam, *equation 2.9* applies. The characteristic value f_k is adjusted with k_{mod} due to the type of load and service class, *cf. table 2.4* and by the partial coefficient γ_M, which is 1.25 for glulam due to uncertainty in the material. As strength classes for glulam are certified and the manufacturing process of glulam undergoes a specific control, a lower γ_M applies than for structural sawn timber.

Machine shed, Söderköping, Sweden.

$$f_d = k_{mod} k_h \frac{f_k}{\gamma_M} \quad (2.9)$$

For rectangular cross-sections with a depth smaller than 600 mm, values for the characteristic bending strength $f_{m,k}$ and the characteristic strength in tension parallel to the grain $f_{t,0,k}$ may be increased by factor k_h (size effect factor), *see equation 2.10*. For other characteristic strength properties, factor k_h is equal 1.

$$k_h = \min \begin{cases} \left(\frac{600}{h}\right)^{0.1} \\ 1.1 \end{cases} \quad (2.10)$$

where h is the depth (in mm) of the cross-section.

According to rules in Eurocode 5, the bending and tension strength shall be corrected with regards to size effects. In present rules, the shear capacity of glulam is reduced by k_{cr}.

2.3.4 Design of glulam in the serviceability limit state

In the serviceability limit state, the structure shall be sufficiently stiff to eliminate any unpleasant vibration or deformation that might impair the function of the building element, e.g. roof drainage. The stiffness of a glulam component is affected by several other factors, apart from its geometry. These factors are the duration of the load, the moisture content and temperature of the material.

Train and bus station, Umeå, Sweden.

Above all, variations in load and moisture content (which includes changes in moisture content) are of great importance. As a rule, $\gamma_M = 1.0$ is used when designing in the serviceability limit state.

When calculating the deformation, one has to define the relevant loads to be used. Three different load combinations can be applied in order to define relevant design situations. The load combinations for serviceability limit states are defined in Eurocode 0. Further information on how to calculate initial and final deformations glulam members and connections, can be found in *chapter 6, page 82*.

2.4 Recommendations on camber and limitation of deflections

It makes sense from a visual point of view to limit the deflection of roof beams, for example. Deflection, as small as 1/300 of the span, is visible, especially if horizontal reference lines exist. However, such visual requirements should be formulated differently for different types of building — for example lower in a storage building than in an exhibition hall.

2.4.1 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 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 generally 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.

2.4.2 Deflections

In *table 6.1, chapter 6, page 89*, acceptable deflections for straight members in relation to the span for some types of component are proposed. The information can also be used for arches, frames and other structures.

Connection to non-loadbearing internal walls must 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.

Structural systems in glulam

Glulam provides a wide variation of structural systems. In this chapter a number of basic designs of glulam structures for hall-type buildings and timber bridges 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 (e.g. free height, constraints in roof slope, natural light, etc., *see figure 3.1*) as well as by the budget. Production and transportation constraints can in some cases be critical, *see section 1.6 in Chapter 1, page 25*.

Efficient material utilization can be found by considering the following recommendations (rules):

- Elements should preferably be loaded with axial forces (compression, tension).
- High bending moments and high risk for lateral buckling (slender beams) should be avoided.
- Shear forces should be critically considered.
- Eccentricities and torsion should be avoided.
- Tension perpendicular to the grain from loads or shrinkage should be avoided.
- Space systems are often optimal with regard to stabilization and robustness.

Structural systems often may be optimized by:

- adapting the dimensions to the sectional forces and, where applicable, the relevant sectional force distribution (e.g. double tapered beam, *see figure 3.5, page 40*)
- choosing suitable locations of supports (intermediate supports, continuous beams)
- choosing locations of hinges (e.g. Gerber systems)
- use of struts to decrease the span, *see figure 3.2*
- trussed beams (creating intermediate elastic supports), *see figures 3.11 to 3.13, page 44*.

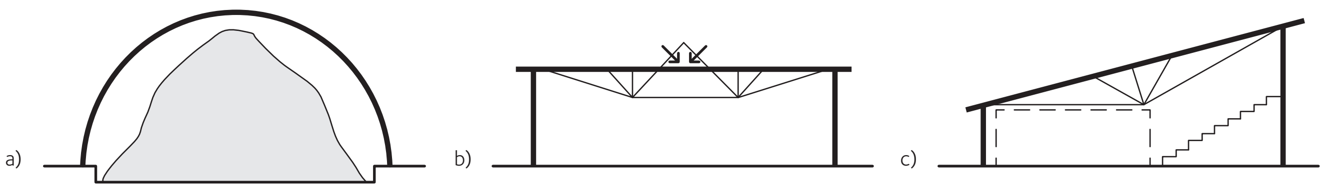


Figure 3.1 Shaping a building due to different constraints. Use of the building as storage for: a) bulk material, b) natural light via the roof and 3) prescribed free height.

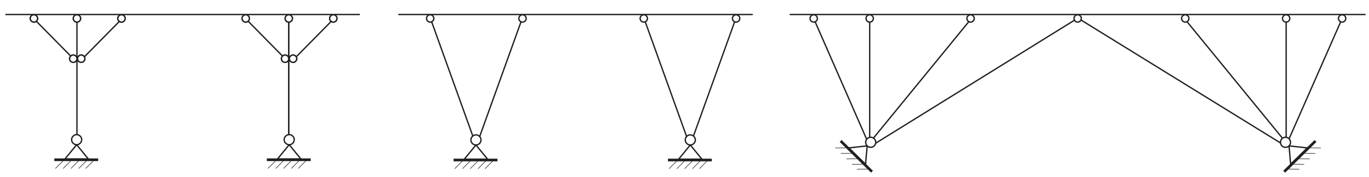


Figure 3.2 Struts reducing the free span of the beam

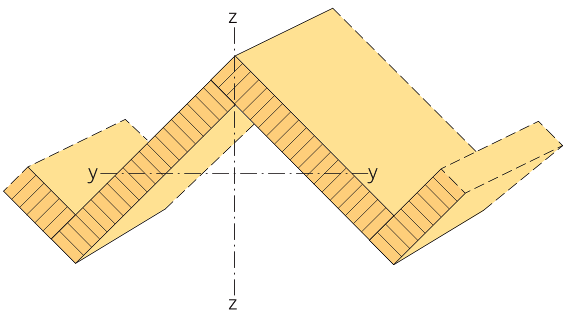


Figure 3.3 **Surface system (folded structure) fulfilling several functions: space enclosing, load-bearing, stabilizing**

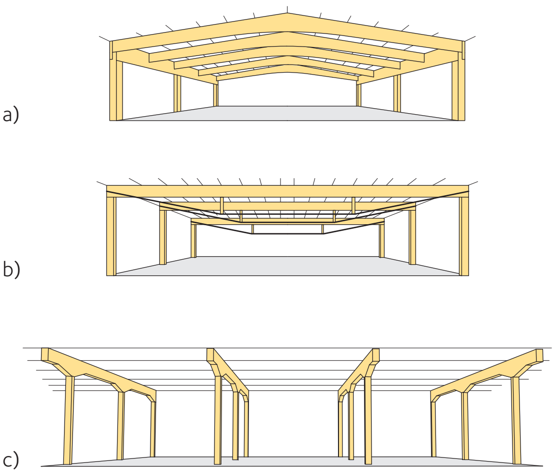


Figure 3.4 **Beam and column system**
a) Pitched cambered beam with curved underside, b) trussed beam, c) beam with increased sectional depth at the supports.

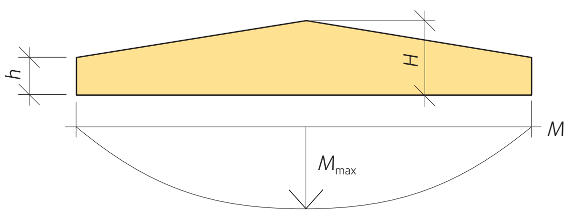


Figure 3.5 **The shape of a symmetrical double tapered beam approximates to the moment diagram of a beam simply supported and with uniform loading**. It is therefore more economic in material than a constant-depth beam.

Elements creating a surface are in general "self-optimized" as they fulfil several functions: load-bearing, stabilizing and space enclosing, *see figure 3.3*.

Table 3.1 at the end of this chapter is an overview of the most common types of glulam systems for buildings. To make the choice easier, the table gives recommended span ranges and approximate member depths for the various types of structure. They correspond to average values under normal conditions. Small loads or closely-spaced members reduce the member depth h somewhat. The opposite also applies.

3.1 Beam and column systems

In its simplest and most common form the glulam structure consists of beams simply 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 depth to vary with the forces in the beam. The connection between beam and column should be designed to prevent lateral buckling of the beam ("fork support", *see Chapter 14, page 198*).

An example of this is the symmetrical double-tapered beam, where the depth is at a maximum at mid-span, where the bending moment is largest, *see figure 3.5*. However, due to the inclination of the timber surface, compression stresses perpendicular to the grain develop at the tapered edge. This is not so critical for the beam in *figure 3.5*. If the beam is turned upside down, however, the tapered side will be stressed in tension, and tension stresses perpendicular to the grain will develop at the edge, which can lead to severe cracking of the beam. Therefore, double-tapered beams should only be used where the inclined side is the compressed side in bending (upper side). More on this can be found in *Chapter 7, page 102*.

It is often deformation, i.e. the largest 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 edge curve. A popular form is a symmetrical double pitched beam with a curved underside, *see figure 3.4 a)*. However, in curved beams, stresses perpendicular to the grain develop and need to be considered in the design. Very often, those beams are reinforced for tension perpendicular to grain stresses with glued-in rods, self-tapping screws or plywood sheets, *see Chapter 7, page 102*.

An important part of a building's function is to provide services. It affects the architectural impact to a high degree. A question that often arises, is therefore if it is possible to make holes and notches in glulam components. In a normal beam the whole cross-section is used to take shear forces. These are largest at the supports and it is therefore usually not suitable to make holes or notches near the supports. Further, holes should be placed in the centre of the cross-section, where the bending stresses are small. *Figure 3.6* illustrates the region where any hole should preferably be placed in a simply supported beam. The above argument deals with principles. Detailed information on design aspects of beams with holes and notches is given in *Chapter 5, page 70*. Holes should never be made in beam regions that experience stresses perpendicular to grain, e.g. parts of tapered beams, curved beams and pitched cambered beams.

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. However, it is almost impossible to clamp a column perfectly (slip and gaps in joints with mechanical fasteners), so for such a column the buckling length is, in practice, a little more than twice the column length *L*, *see Chapter 4, page 53*. In a typical column, hinged at the top and the base, the buckling length is equal to *L*.

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 structure. 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 13, page 170*.

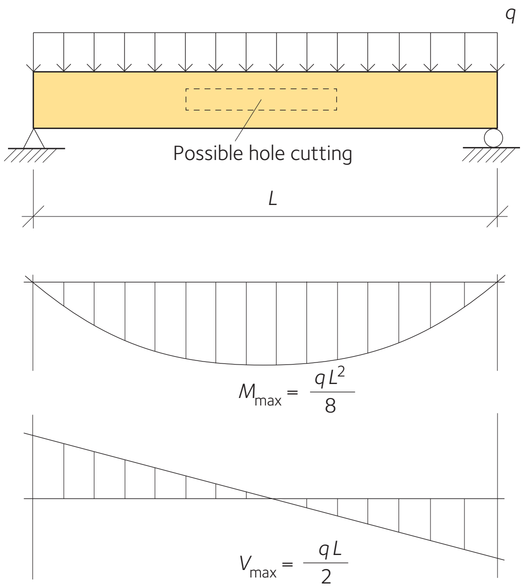


Figure 3.6 External moments and shear forces in a simply supported beam, with uniformly distributed load. Holes could be made in the central part of the beam.

3.2 Continuous beams

Beams with several supports or beams with a cantilever can make better use of material than that achieved with simply supported beams, because the bending moment doesn't vary so much along the beam length. The efficiency can be further increased by increasing the depth of the beam at the inner supports, thus increasing the bending moment at these supports and decreasing the bending moment in the bay.

Continuous beams can usefully be designed as what is known as a Gerber System. Here the joints are designed as hinges, and placed so as to achieve a favourable distribution of bending moments and suitable lengths for transportation. Suitable locations for Gerber hinges in continuous beams are given in *Chapter 12, page 166*, for beams with uniform loading.

Continuous beam systems are particularly suitable for primary load-bearing structures in roofs. For secondary structures, however, simply supported beams should be preferred in order to prevent progressive failure, *see more in Chapter 12, page 166*.

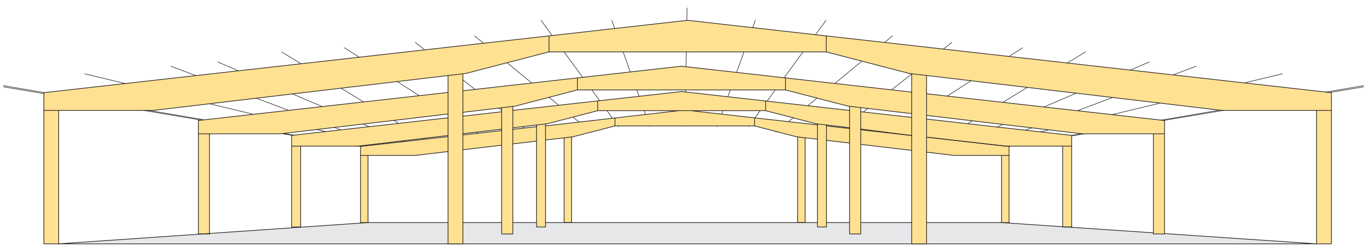
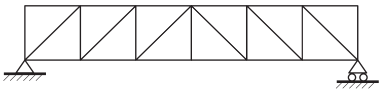


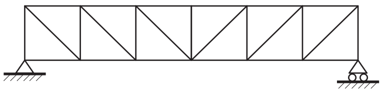
Figure 3.7 Continuous beam using the Gerber system. Here as optimised beams with overhang and inset pitched beams.

Parallel trusses

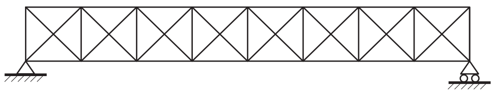
a)



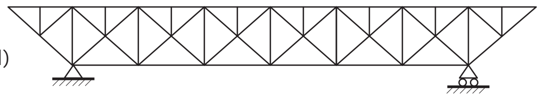
b)



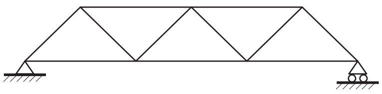
c)



d)

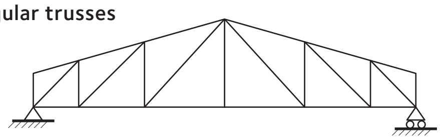


e)

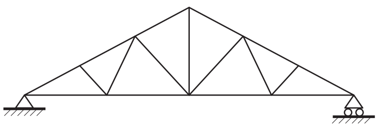


Triangular trusses

f)

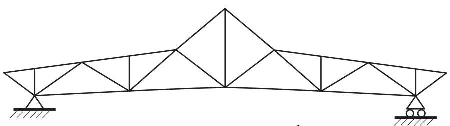


g)

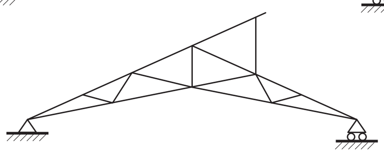


Special truss design allowing for natural light

h)

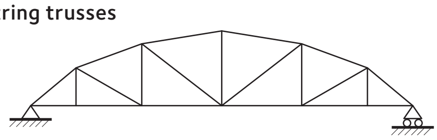


i)

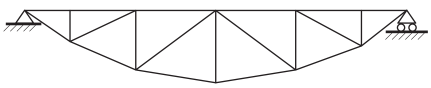


Bowstring trusses

j)



k)



3.3 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, *see figure 3.8*. This applies particularly where a low roof slope is required and where the construction height is fairly generous.

Among the advantages is the fact that trusses can be made in the factory in suitable parts, tailored for transportation needs. These parts are then assembled on the building site. Among the disadvantages are a large number of, sometimes complicated, nodes, which implies high costs. It is desirable that the architect takes part in the design of the truss, especially the nodes and other details. Truss design should include a check of different locations of diagonals, in order to optimize the structure, *see figure 3.8*. At all nodes, the centre lines of the meeting elements should intersect at one point in order to avoid eccentricities, which induce (secondary) bending moments.

Pipes etc. for service functions can in many cases be placed near the top member of a truss or above the bottom member. Compression members are made in glulam, while tension members can be in steel with a cross-section that mainly absorbs tensile forces. In this case the structural height is the distance between the centre lines of the top and bottom members. It can be either constant along the truss or follow the bending moment distribution, *see figure 3.8 f)–k)*. Trusses are a good choice at spans between about 30 to 85 m.

Figure 3.8 Trusses, assuming vertical loading (dead weight, snow) on the upper chord

Parallel trusses.
a) Compressed diagonals,
b) tension diagonals,
c) double diagonals, or
d) with extra supports of the compressed chord,
e) parallel truss with compression and tension diagonals, optimized for small number of joints.

Triangular trusses.
f) Trimmed,
g) normal.

Special design to allow for natural light.
h), i).

Bowstring trusses.
j) Verticals and diagonals subject to very small compressive or tensile forces. Curved upper frame for compressive forces,
k) verticals and diagonals subject to very small compressive or tensile forces. Curved subframe for tensile forces.

3.4 Three-pin trusses

Three-pin trusses and other truss systems can provide a good solution (with respect to both load-bearing and economical aspects), where demands on span exclude solid beams and where arches or 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 hinged 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, *see figure 3.9*. In the latter case the truss is normally supported by columns, but the tension member can also be inserted in a tube cast into the floor slab. The beams are usually straight and of constant depth, but variations in type or shape can also occur, e.g. trussed beams.

The trussed beam can be regarded as a transitional structural form between solid beams and trusses, *see figures 3.10 and 3.11*. The joints are, however, fewer and simpler in design than for a pure truss. Today, there are companies that produce purpose-made steel parts for this type, such as tension members and joints. By pre-stressing, the tension rods may induce a positive effect on the buckling behaviour of the compressed struts, *see figure 3.12*. When the strut starts to deflect out of the plane, the concentrated load on the strut (from the tension rods) changes its direction from vertical to inclined and still points towards the joint between strut and beam. The resultant at A is a horizontal force P_1, which makes the system self-stabilizing, thus decreasing the induced displacement at A, *see figure 3.12 a)*. The self-stabilizing effect occurs as long as the connection point between beam and strut is on the same level or above the connection points between beam and tension rods, *see figure 3.12*. To be sure that this system is self-stabilizing, the trussed beam should have a sufficient pre-camber, so that even under full load the camber should be at least $L/200$. In beams without pre-camber, other stabilizing methods may be used, e.g. stabilizing the lower end of the strut against out-of-plane movement or by rigid connections between beam and strut, *see figure 3.13*. In *figure 3.13 b)* a knee brace with members that can take compression is not recommended if non-symmetrical loading (i.e. loading on only one side of the main member) can occur, as the stabilizing member could push the main member sideways and thus destabilize the main member. In those cases, stabilization with tension rods, *see figure 3.13 c)*, is a much safer method.

Three-pin trusses can be designed as space frames. The roof beams are then arranged to radiate from a summit and the tension members are arranged as a polygonal tension ring, linking the bottom ends of the beams around the edge of the roof, *see figure 3.9 f)*.

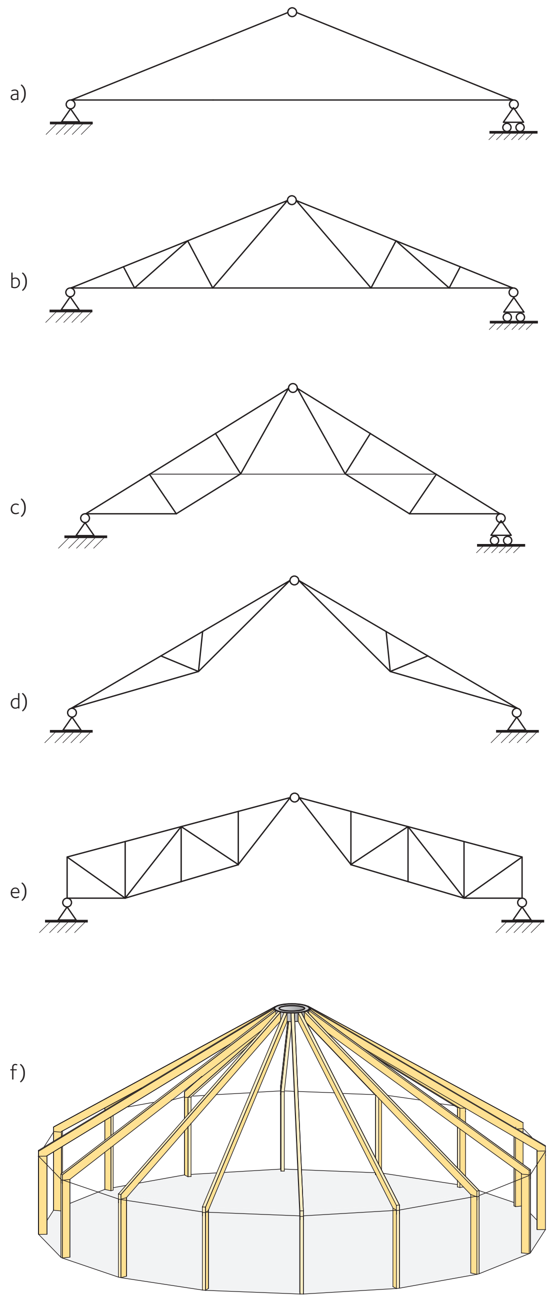


Figure 3.9 Three-pin trusses
a) Three-pin truss with tension rod,
b) three-pin truss with trussed beams and tension rod,
c) three-pin truss with tension rod in the middle part and trussed beams,
d), e) three-pin truss with trussed beams, without tension rod (two fixed supports),
f) three-pin trusses radially arranged with polygonal tension ring around the roof edge.

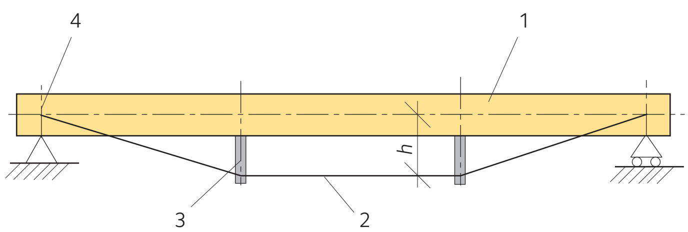


Figure 3.10 Example of trussed beam with a structural depth of *h*
1. Beam
2. Tension member (steel)
3. Compression member (strut)
4. Recessed or external steel fixtures.

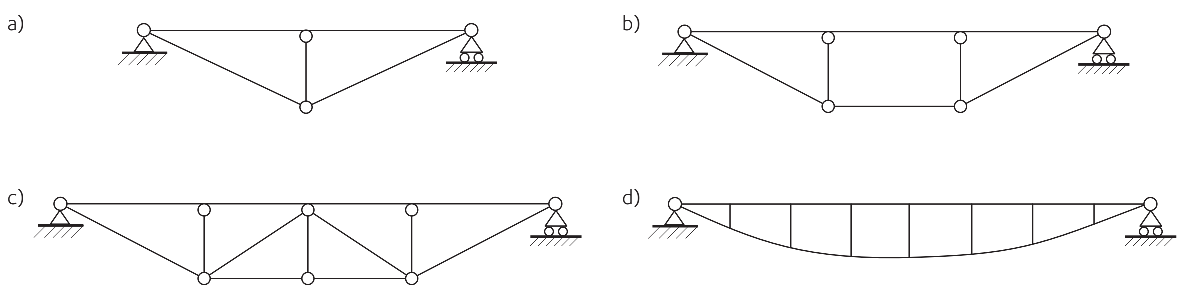


Figure 3.11 **a) – c) Trussed beams with different number of struts, d) fish belly beam.**

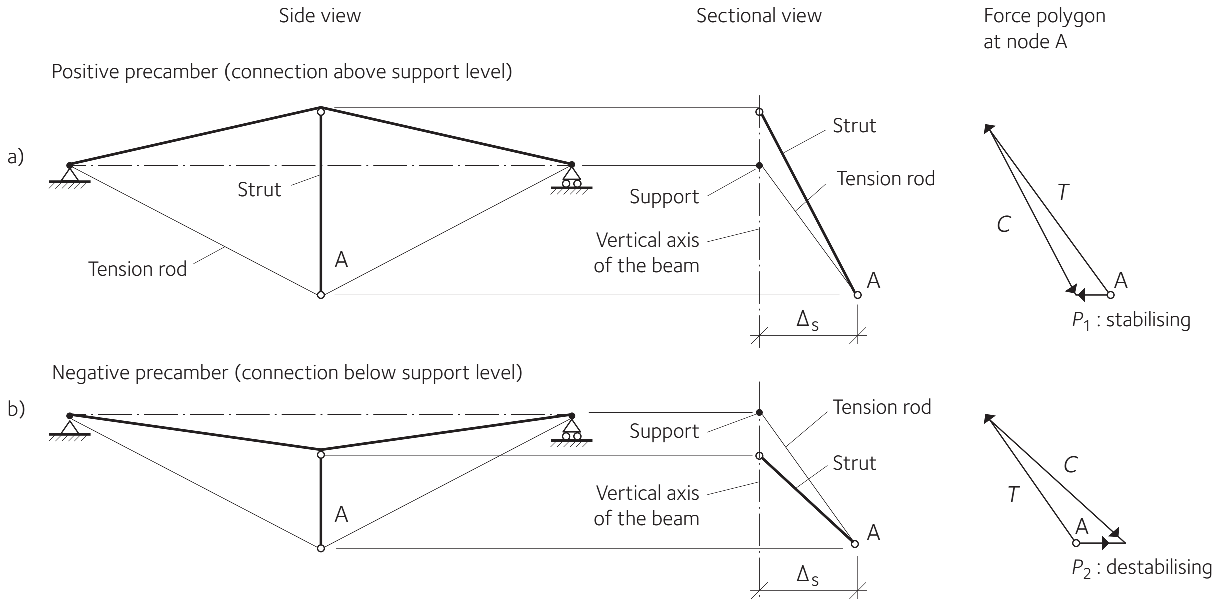


Figure 3.12 **a) Stabilizing trussed beams with pre-camber, creating a self-stabilizing system. b) Destabilizing situation in an initially deformed beam with 'negative' pre-camber.**

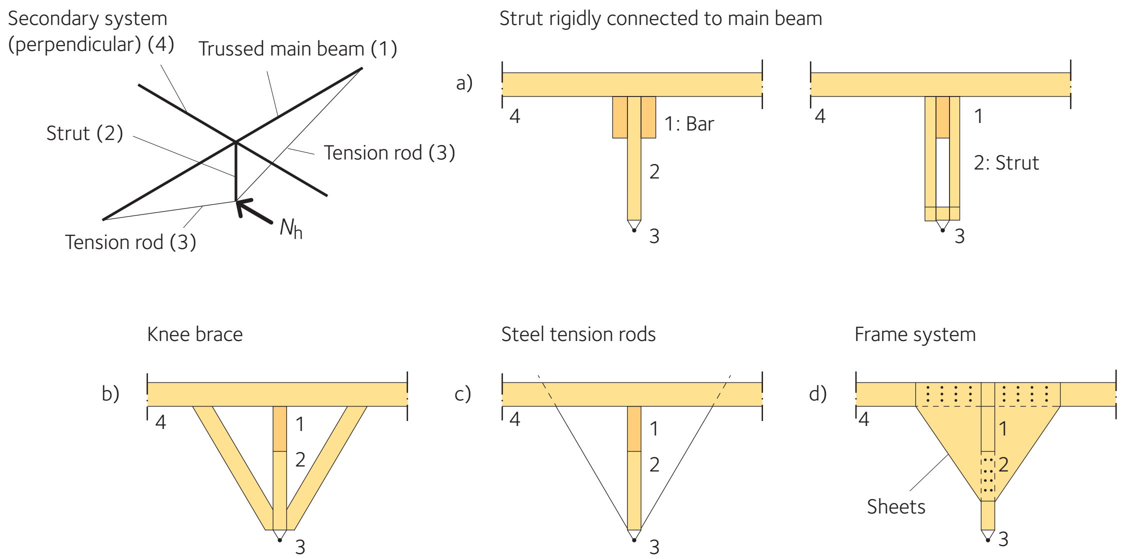


Figure 3.13 **Stabilizing trussed beams by either different means of rigid connections between beam and strut a), d) or bracing the lower end of the strut against out-of-plane movement b), c).**

3.5 Arches

Glulam is a very versatile structural 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 "line of thrust" can be chosen. An arch that follows the line of thrust and is subjected only to vertical loads will be loaded in pure compression throughout its whole length. If the load is uniformly distributed horizontally the line of thrust will be a parabola; if, the load is concentrated in a few distributed places, it will be a polygon. However, as most structures are loaded with different loads and load combinations (self-weight, snow, wind, non-symmetrical or eccentric snow and wind etc.), it is difficult to find a suitable single compression line for those load cases and load combinations. The arch geometry should thus be chosen for the largest loads or permanent loads, and the arch will in course of time be loaded in various combinations of compression and bending.

Thanks to the fact that the material is more efficiently utilised in an arch, the cross-sectional depth will be much smaller than that of a beam with the same span and loading. The difference in the way a beam functions, compared to an arch, is illustrated in *figure 3.15*.

The design options, together with the high strength, mean that glulam structures are particularly viable over large spans. Glulam arches have been built with free spans of over 100 m.

In practice, circular arches are probably the most usual form for short spans. For large spans, however, parabolic arches can be more economical. In order to increase the headroom near the supports, an elliptical or other arch form can be chosen. Another method of increasing the headroom is to place the arch on columns, *see figure 3.14 b)*.

An arch requires 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 placed under the floor slab, *see figure 3.14*.

Arches are normally built with hinged fixtures at the supports and (usually) a hinged joint at the ridge. In larger spans, additional joints can be arranged for transportation reasons. Such joints are made rigid and placed in areas where the bending moments are small.

The three-pin arch is statically determinate, which means simple calculations and insensitivity to subsidence of the supports. This arch-type is also stable in its own plane and does not transmit any bending moments to the foundations. However, as arches are not stable in the out-of plane direction, but are subjected to column buckling and lateral buckling, they should not be too slender (not too small cross-section width) and have to be braced accordingly, *see also Chapter 13, page 170*.

Arches radially arranged will shape a dome-like form. A genuine dome also utilises the shell effect. If spans are large, a dome can be an interesting solution.

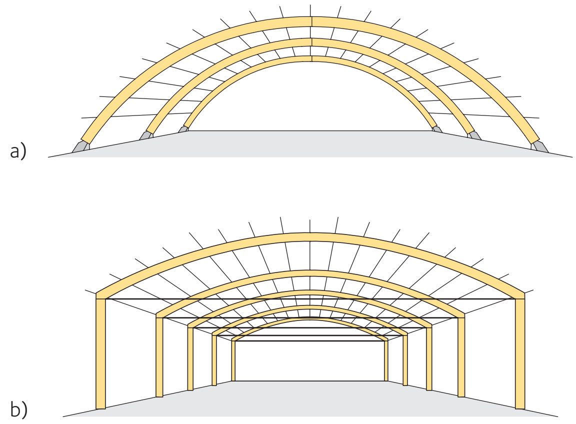


Figure 3.14 Arches
a) Three-pin arches on foundations,
b) arches with ties on columns.

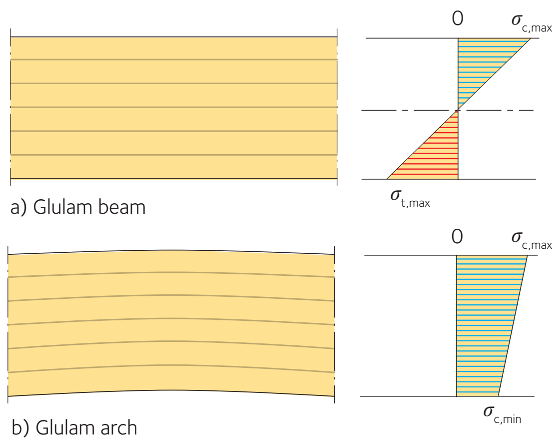


Figure 3.15 Bending stresses in a loaded beam a) vary through the depth. Compression at the top and tension at the bottom. At the centre of the cross section the stress is zero. An arch b) on the other hand has compression throughout its cross-section.

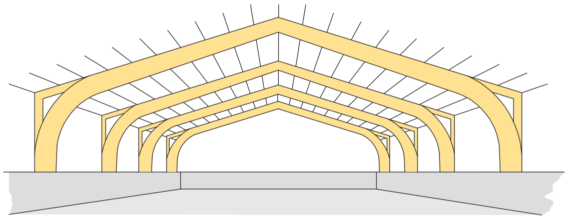


Figure 3.16 **Portal frame with curved haunches**

Riding school, Bökeberg, Sweden.

3.6 Portal frames

For functional, aesthetic or economic reasons it may be preferable to use a different type of arch than the material-saving parabola or the circular arch. Headroom requirements throughout the whole area of the building often lead to the characteristic glulam form of a three-hinged frame with curved haunches, *see figure 3.16*, or, if demands on utilisation of the whole area are extreme, sharp-cornered haunches.

The function of the building is improved in these two cases – at the cost of a somewhat lower degree of material utilisation. In other respects the three-pin frame has the same advantages as the three-pin arch – simple design and simple foundations. This type is especially suitable on poor sub-soils, as it does not transfer any bending moment to the foundations.

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

3.7 Cantilevers

In many situations it is a requirement that one or both of the long sides of a building are open and free from columns. Examples are open-air stages, train platform canopies and grandstands, *see figure 3.17*.

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

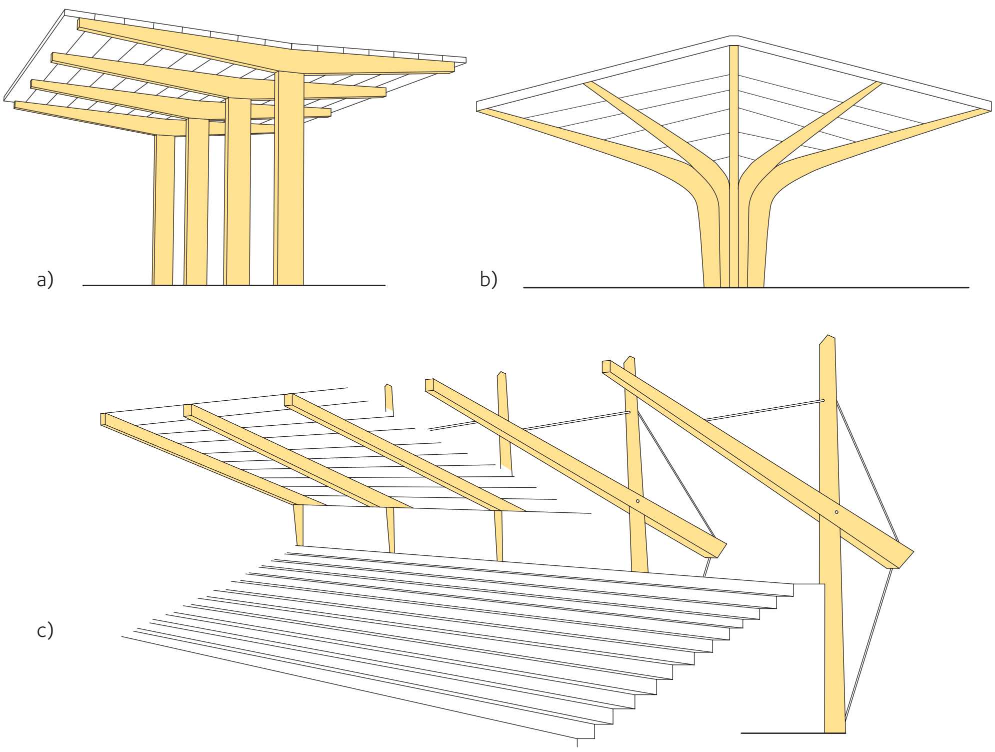


Figure 3.17 **a) Cantilevers, b) canopy with clamped columns, c) grandstand with cantilevers at rear**

3.8 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 "hypar" shells (hyperbolic paraboloid) may be mentioned as common types, *see figure 3.18*. A valuable characteristic of the two last named types is that their surface can be generated by straight lines and can thus easily be built up of one or more intersecting layers of timber boards or corrugated metal sheets.

3.9 Combination systems

Combinations of various structural systems often provide elegant solutions.

The desire for plenty of daylight in a building can be satisfied with a saw-tooth structure consisting of three-pin trusses placed on continuous beams, *see figure 3.19 a)*.

Difficult sub-soil conditions can be mastered by concentrating the load reaction forces to a few support points, each with a reinforced foundation slab. In the composite arch-beam systems in *figures 3.19 b) and c)*, the main part of the roof load is carried by the arches.

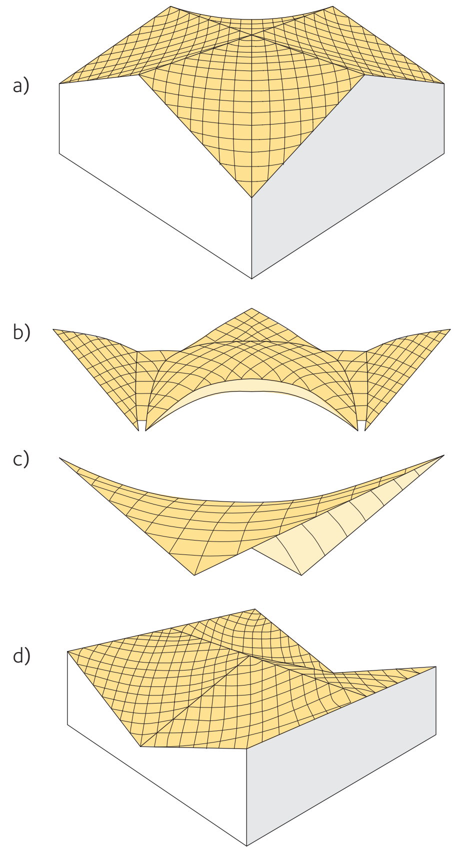


Figure 3.18 Shells
a), c), d) Hyperbolic paraboloid,
b) intersecting shells.

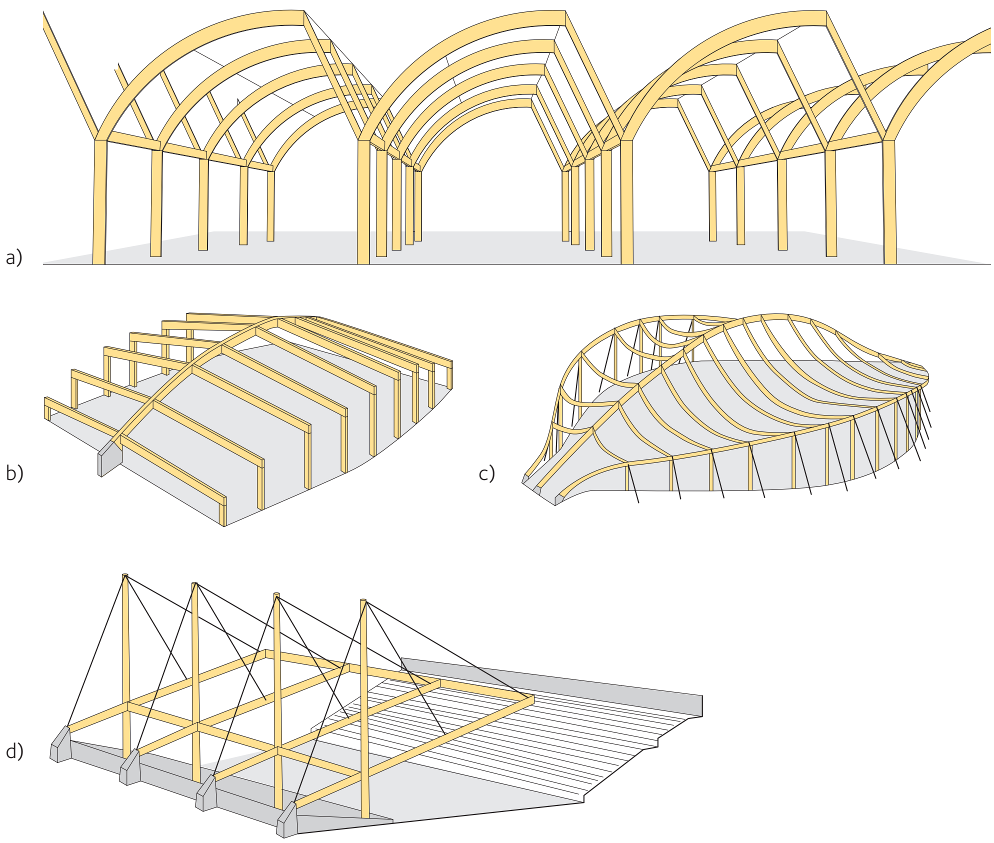


Figure 3.19 Combination systems. a) Saw-tooth roof, b), c) arch and beam combinations, d) cable-stayed roof.

3.10 Glulam bridges

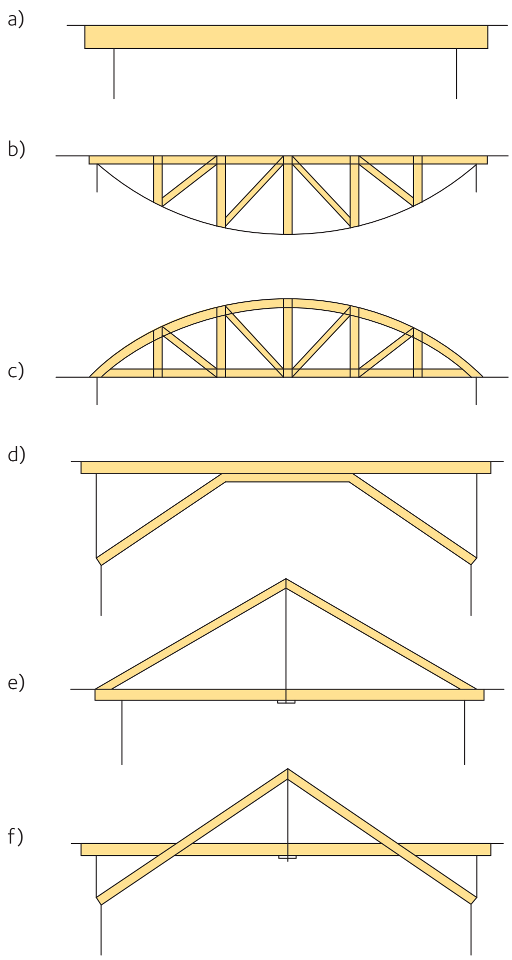


Figure 3.20 Girder bridge types
a) Simple girder, b), c) truss, d) strut frame, e) king post, f) combined king post-strut frame.

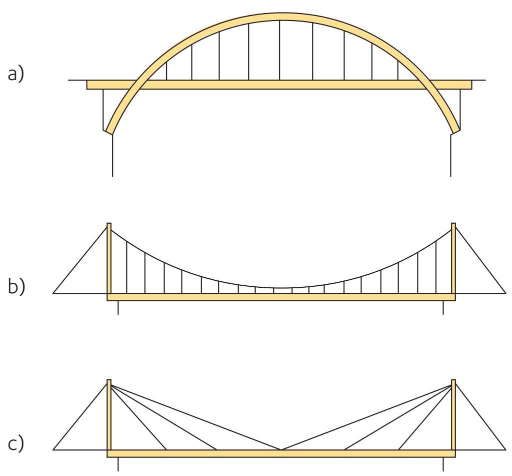


Figure 3.21 a) Arch bridge, b) suspension bridge, c) cable-stayed bridge.

Today in the Nordic countries timber is mainly used for footbridges (for pedestrians, bicycles etc.). In recent years, however, the interest in timber road bridges has shown a marked increase and a considerable number of such bridges have been built.

From a 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 part 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 that carry the loads (self-weight, traffic load, snow and wind) 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 mostly made of reinforced concrete, sometimes of steel, but historically stone and brick were common. Timber poles can be used if they are protected when in possible contact with water.

Girder bridges, arches and cable-supported bridges (suspension and cable-stayed bridges) are the three principal types of bridge superstructure, *see figures 3.20 and 3.21*. Girder bridge types often include slab bridges, trusses and other structural types consisting of assemblies of bars, *see figure 3.20*. Combinations of various types also occur. Since the types of structural systems used for roof structures (described earlier) and for bridges do not differ much, only aspects particular to timber bridges will be mentioned here.

The type of structure that is most suitable in each case depends on the specific conditions, e.g. what free span and what unobstructed height is demanded, the room available for the structure and the type of traffic. Appearance is often of great importance, since large bridges are as a rule a dominating feature in the landscape. Other factors influencing the choice are the soil conditions and any requirement concerning the building material to be used.

Stress-laminated deck bridges

One of the simplest types of timber bridge is the nail-laminated timber deck, which consists of a series of side-by-side boards or planks nailed to each other; it combines both roadway and structural system in the same structure. A modern development of this principle is the transversely pre-stressed timber slab bridge, typically named stress-laminated deck bridge, *see figure 3.22*.

The road slab is made of glulam beams – for smaller spans, planks – pressed together by pre-stressing steel rods. The structure is easy to erect and the slab has good load distribution characteristics, and as it is laterally stiff, extra wind bracing is 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 be made continuous in several bays.

Girder bridges

In girder bridges the load-bearing structure usually consists of two or more longitudinal glulam beams; for small spans and small loads these beams can be of sawn timber. The beams can span one or more bays. When the distance between the beams is small, the road-deck consisting of planks can rest directly on the beams, *see figure 3.23*.

In small bridges the bridge deck can act as a diaphragm and take up horizontal loads, e.g. wind, while preventing the beams from buckling. For larger spans, wind-bracing is needed, usually in the form of horizontal trusses between the main beams, in level with their tops or undersides.

If the main beams are more widely spaced, the bridge deck will instead rest on transverse secondary beams, which transfer the load to the main beams. The bridge deck of planks resting on the secondary beams usually has a top surface of asphalt, *see figure 3.24*. For long spans and heavy traffic the deck is often designed as a pre-stressed slab, structurally co-acting with the main beams forming a T-section or box section. The deck then acts as a diaphragm for horizontal loads and no special measures to counteract wind forces are therefore necessary.

King post, strut frame

Girder bridges designed as king post or strut frame structures, *see figure 3.20 d), e) and f), page 46*, were formerly used for spans too large to be bridged with ordinary girders. The main beams were thus provided with one or more intermediate support points, which meant that the material was used more effectively. The tension rods are normally steel. It is necessary to take the differences in heat expansion and stiffness properties of the materials into account in the design process.

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.

Arch bridges

These structures are normally designed with double arches and with the roadway placed below, between or on top of the arches, *see figure 3.25*. The arches are laterally restrained by trusses or frames. Together with the bridge deck, these components are also used to take wind loads and other horizontal loads on the structure.

For transportation and manufacturing reasons the arches are often designed as 3-pin arches, especially 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 possible to make rigid joints on the building site.

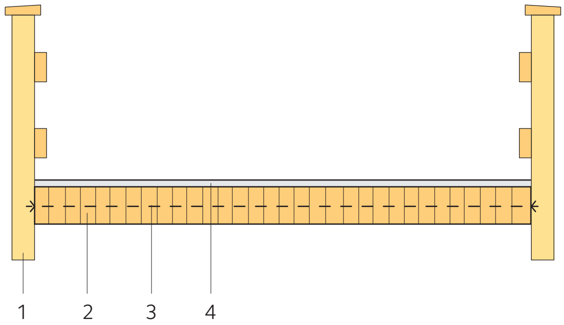


Figure 3.22 **Slab bridge with transversely pre-stressed slab of glulam.** 1) Handrail, 2) upright glulam beams, 3) steel tension rod, 4) road surface.

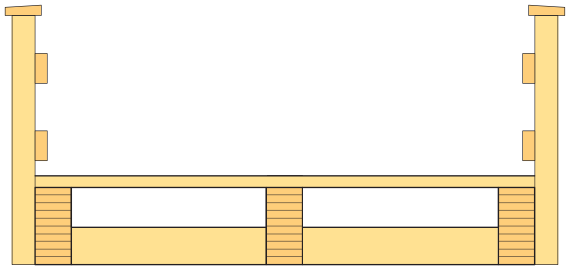


Figure 3.23 **Girder bridge with planks laid directly on the glulam beams**

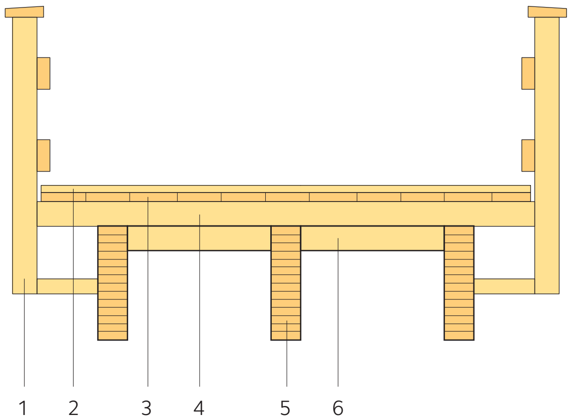


Figure 3.24 **Girder bridge with transverse secondary beams and planks.** 1) Handrail, 2) road surface, 3) planks, 4) transverse secondary beam, 5) glulam beams, 6) wind bracing.

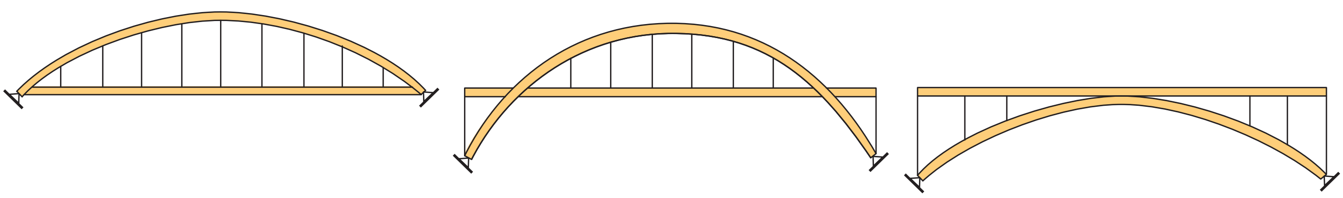


Figure 3.25 **Arch bridges with through, half-through and top roadways**

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, *see figure 3.21 b), page 48*.

A cable-stayed bridge consists of a girder bridge supported on two or more rigid supports. Between these supports, the beams are elastically supported by inclined cables from one or more towers, *see figure 3.21 c), page 48*. The oblique support reactions from the cables induce axial forces in the bridge girders. The difference in stiffness and heat expansion characteristics of the materials (i.e. steel and timber) must be taken into account in the design.

Figure 3.26 Stress-ribbon bridge

Stress-ribbon bridges
A stress-ribbon bridge may be regarded as a kind of suspended bridge, where the bridge deck has two functions: (1) being a (curved) roadway surface for pedestrians and light vehicles and (2) acting as a primary, pre-stressed, tensile bridge component, *see figure 3.26*. The bridge form is partly determined by the catenary-like curve for the load case in question. As the bridge deck has some bending stiffness and due to non-symmetrical loading, the form deviates from the catenary. The large horizontal forces have to be resisted by the substructure. Due to shrinkage and swelling and to a minor extent due to temperature changes, the length of the structure will change, which has to be considered in the design.

3.11 Connection details

Glulam structures are often visible and thus constitute an important part of the architecture of a (timber) building. This is no less true for the connection details, which should receive special attention on the part of the designer.

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 tensile forces. Today, connections and intersections are made with nails, screws and various types of steel fixtures, which can equally well transfer tensile and compressive forces, *see further Chapter 14, page 198*, which gives detailed information on different connection details and design procedures.

Steel fixtures transfer forces in a more concentrated and well-defined manner than the old types of timber joints. For example a hinge, which joins two structural components without the ability to transfer moments, can really be designed as a true hinge.

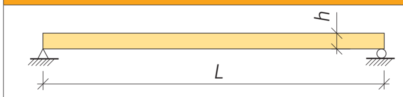
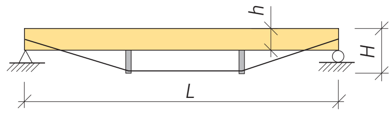
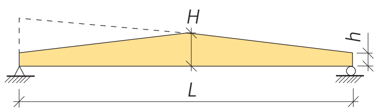
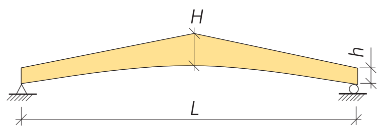
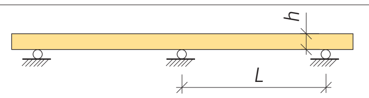
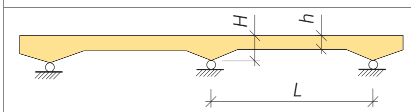
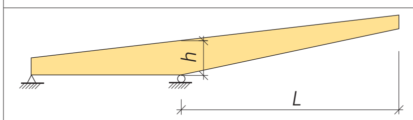
Bandy hall, Nässjö, Sweden.

3.12 Summary table

The summary in *table 3.1* covers only the most common types of timber structure for buildings. In order to help the designer to choose between different structural types and simplify design, recommended spans and approximate structural depths are given for various types of structure. They correspond to average values under normal circumstances. Light loading or dense spacing of units mean somewhat lower structural depths compared to those in the table. The reverse may also be true. A good proportion (starting value) between member depth h and width b is $h/b = 5$. The choice of a structural system is often influenced by various limitations in production and transportation.

The values in *table 3.1* should be seen as recommended starting values for member depths, not as final values, and detailed design calculations must be carried out to find the final member size for both the ultimate limit state and the serviceability limit state. If needed, the system can be given some pre-camber to account for anticipated future deflections. For some structures, e.g. arches, tension stresses perpendicular to grain have to be checked. Also the different shape factors for e.g. snow loads on different roof types have to be considered carefully. All structures have to be braced to prevent lateral buckling and to withstand horizontal loads, *see Chapters 4, page 53 and 13, page 170*.

Table 3.1 Structural systems using glulam. Recommended roof slopes and spans
Approximate sectional depths for members at approximately 6 m centres, variable load approximately 3.0 kN/m², self-weight roof structure 0.5 kN/m². The values in this table differ in some cases from the values in the corresponding table in *The Glulam Handbook Volume 1*, due to different input data. Systems in italics and marked with * are less common.

System sketch	Name	Slope	Span L (m)	Depth h
	Straight beam on 2 supports	≥ 3°	< 24	$h \approx L/14$
	Straight trussed beam on 2 supports	3 – 30°	< 50	$h \approx L/33$ $H \approx L/12$
	Double pitched beam, single pitched beam	1.4 – 6°	10 – 25	$h \approx L/20$ $H \approx L/14$
	Symmetrical double pitched beam with curved underside	3 – 15°	10 – 20	$h \approx L/30$ $H \approx L/16$
	Straight continuous beam on several supports	≥ 3°	< 25	$h \approx L/14$
	** Haunched continuous beam on several supports*	≥ 3°	< 25	$h \approx L/18$ $H \approx L/14$
	Cantilevered beam on two supports	< 10°	< 12	$h \approx L/7$

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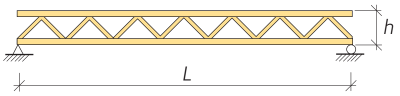
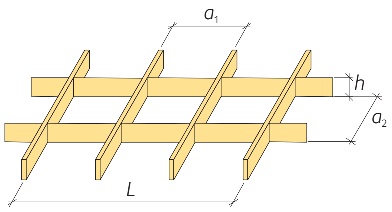
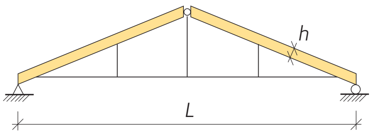
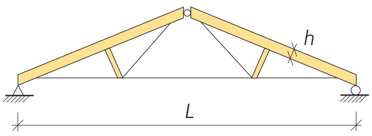
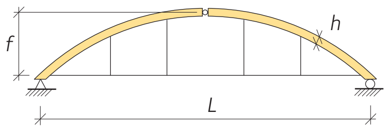
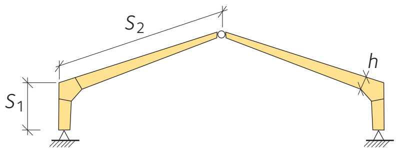
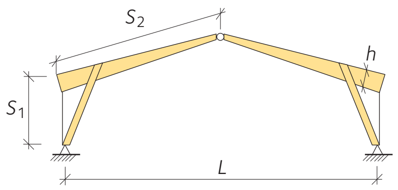
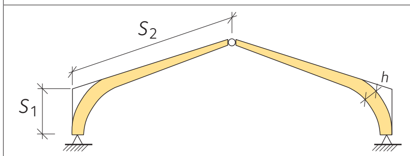
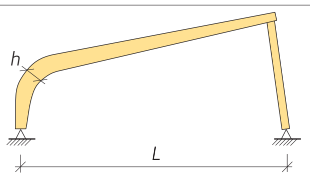
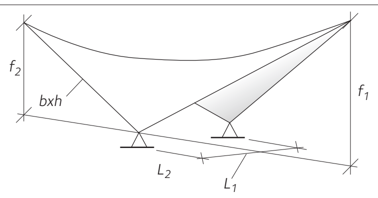
System sketch	Name	Slope	Span L (m)	Depth h
	Straight truss on two supports	≥ 3°	30 – 85	$h \approx L/12$
	* *Orthogonal grid*	≥ 3°	12 – 25	$h \approx L/20$ (a = 2.4 – 7.2 m)
	Three-pin truss with or without tie (affects support type)	≥ 14°	15 – 30	$h \approx L/28$
	Three-pin truss with tie and trussed beams	≥ 14°	25 – 80	$h \approx L/45$
	Three-pin (two-pin) arch with or without tie (affects support type)	$f/L \geq 0.14$	20 – 100	$h \approx L/50$
	Three-pin portal frame with finger-jointed haunches	≥ 14°	15 – 25	$h \approx (S_1 + S_2)/13$
	* *Knee braced portal frame*	≥ 14°	10 – 50	$h \approx L/30$
	Three-pin portal frame with curved haunches	≥ 14°	15 – 50	$h \approx (S_1 + S_2)/15$
	* *Propped half portal frame*	≥ 20°	10 – 25	$h \approx L/25$
	* *Hyperbolic paraboloid shell (HP shell)*	$\frac{(f_1 + f_2)}{(L_1 + L_2)} \geq$	$L_1 \approx L_2$ 15 – 60	$h \approx b \approx L/70$ (edge beams)

Straight beams and columns

4.1 Beams

Beams are normally straight glulam components with a rectangular cross-section, which are subjected to bending. Beams can be used in bridges, floor joists, roof beams, purlins etc. Beams with varying cross-sections along their length or with curvature are treated in *Chapter 7, page 102, and in Chapter 11, page 153*.

For beams, necessary design checks should be made with regard to bending moment capacity, shear capacity, deflections and vibrations. At beam supports, the capacity for compression perpendicular to the grain has to be checked. Depending on the length of the beam, different design criteria can be critical. For medium-span beams, bending is most critical, whereas shear can be critical for heavily loaded short- to medium-span beams. For long-span beams, the serviceability criteria for deflection and vibration are most often decisive, *see section 6.2, page 84 and section 6.3, page 95*. For medium-span, simply supported beams with uniform loading, approximate beam depths are about $L/20$ for bending moment and $L/30$ for shear (of course depending on the loading).

4.1.1 Bending and shear

According to the theory of elastic bending, when a solid rectangular member is subjected to a bending moment M about the strong (y-y) axis, the design stress at any distance z from this axis is:

$$\sigma = \frac{M \times z}{I_y} \qquad (4.1)$$

where I_y is the second moment of area of the member cross-section about the y-y axis. When interested in the maximum stresses at the upper and lower edges of a beam with a rectangular cross-section, the section modulus W_y about the strong axis is used:

$$W_y = \frac{I_y}{\left(\frac{h}{2}\right)} = \frac{b \times h^2}{6} \qquad (4.2)$$

with b being the beam width and h the beam depth respectively. The maximum bending stress in the cross-section can then be obtained by dividing the design bending moment by the section modulus:

$$\sigma_{m,y,d} = \frac{M_d}{W_y} \qquad (4.3)$$

Similar expressions can be derived for bending about the weak axis of the member (z-z).

If a member is subjected to bending about two axes (strong and weak) at the same time, the design conditions to be fulfilled are given by:

4.4
$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$
$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

where $\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are the design bending stresses about the principal axes and $f_{m,y,d}$ and $f_{m,z,d}$ are the corresponding design bending strengths.

Note that for glulam, the bending strength for bending in the weak direction is not the same as the bending strength for bending in the strong direction due to system effect and different grades of glulam lamellas, *see section 1.3.4, page 16*. k_m is a modification factor allowing for redistribution of stresses and material inhomogeneity. For rectangular glulam cross-sections, $k_m = 0.7$ is used. For other cross-sections, $k_m = 1.0$.

All elements subjected to bending will also have shear stresses parallel to the beam axis. Shear stresses in the beam are largest at the neutral axis and zero at the edges. For rectangular cross-sections, the maximum shear stress τ (at the neutral axis) is given by:

4.5
$$\tau = \frac{3V}{2 \times b \times h}$$

where V is the shear force, and b and h are the beam width and depth respectively. In design, the following expression shall be satisfied:

4.6
$$\tau_d \leq f_{v,d}$$

where τ_d is the design shear stress and $f_{v,d}$ is the design shear strength. Due to cracking in the wood caused by moisture-induced stresses, Eurocode 5 recommends not to use the whole width b of the glulam element in *equation 4.5*, but an effective width $b_{ef} = k_{cr} \times b$, with $b_{ef} < b$. Different values for k_{cr} basically depending on the timber material, can be found in the national application documents to Eurocode 5. In Sweden, the factor k_{cr} depends upon the exposure conditions and is displayed in *table 4.1*.

Table 4.1 **Values for crack modification factor k_{cr} depending on exposure conditions**

Type of exposure	k_{cr}
Not exposed to rain and solar radiation	0.86
Partially or totally exposed to rain and solar radiation	0.67

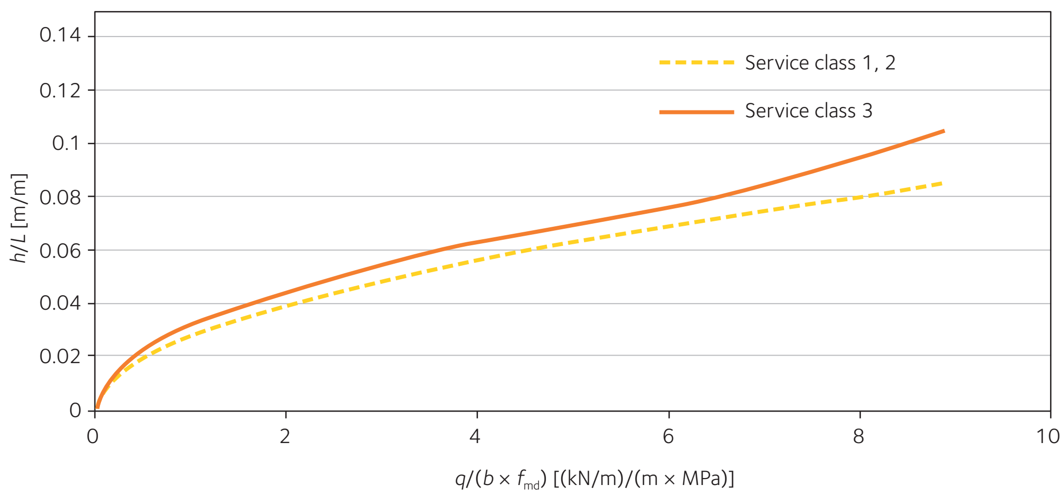


Figure 4.1 **Necessary beam sizes expressed as beam depth/span ratios for varying load levels.** Calculated for service classes 1, 2 and 3 respectively using GL30c. f_{md} calculated according to *equation 2.9*. Design criteria used are related to the bending stress and shear stress (including $k_{cr} = 0.85$). Beams have to be prevented from lateral buckling.

4.1.2 Axial tension and combined tension and bending

Straight glulam members may be subjected to axial forces – tension or compression – or subjected to a combination of axial forces and bending. When loaded in tension, the volume of the member should be considered, as the strength is volume-dependent (Weibull weakest link). When loaded in compression, short members will fail in compression, while slender members will fail in buckling. Members loaded in compression are treated in *section 4.2, page 62*.

For straight members loaded with axial tensile force and/or bending, the following design criteria apply:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$
$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad 4.7$$

where $\sigma_{t,0,d}$ is the design tension stress parallel to the grain, $\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are the design bending stresses about the principal axes. $f_{t,0,d}$ is the design tension strength, and $f_{m,y,d}$ and $f_{m,z,d}$ are the design bending strengths. k_m is a modification factor according to *section 4.1.1*.

4.1.3 Lateral-torsional buckling

Slender straight members are subjected to lateral-torsional buckling if the compressed edge of the member is not restrained from moving out of plane, *see figure 4.2*. In such cases, lateral-torsional buckling occurs at lower stresses than the bending strength. In reality, slender elements are seldom completely straight, which increases the risk of out-of-plane deformation. Other parameters influencing the risk of lateral-torsional buckling are support conditions (free, pinned or fixed), type of loading, position of loading (edge/neutral axis), type of bracing and distance between bracings. Those factors affect the effective length, which is used in the design equations.

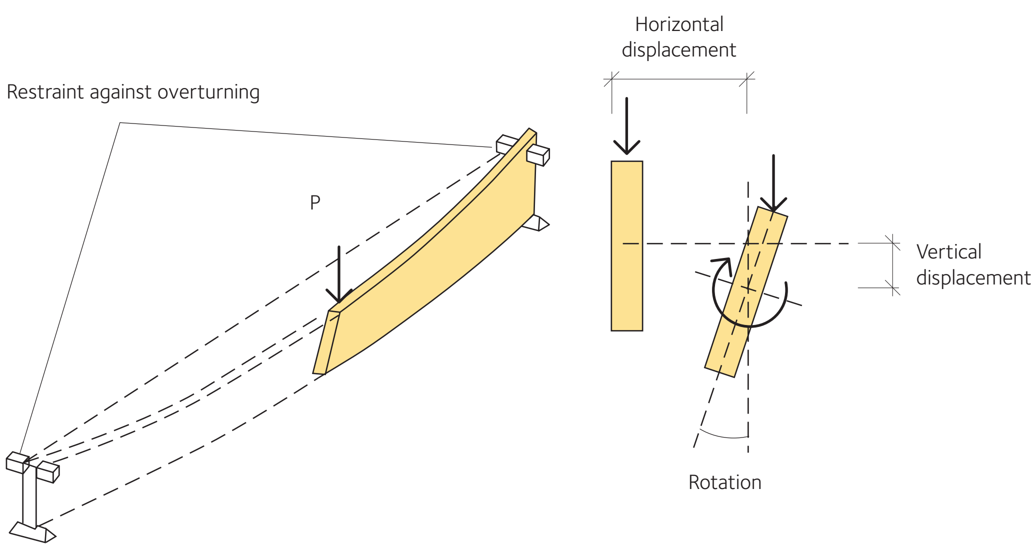


Figure 4.2 Lateral-torsional buckling of a simply supported beam

The critical bending stress $\sigma_{m,crit}$ at which lateral-torsional buckling occurs in a glulam beam with a solid rectangular cross-section can be written as:

$$\sigma_{m,crit} \approx \frac{0.78 \times b^2}{h \times l_{ef}} \times E_{0.05} \tag{4.8}$$

Here b and h are the width and depth of the beam respectively, l_{ef} is the effective length, accounting for loading (bending moment distribution), support and bracing conditions, and $E_{0.05}$ is the 5th percentile value of the modulus of elasticity along the grain direction. In the case of a simply supported beam with uniform moment loading and loading at the neutral axis of the beam, l_{ef} may be substituted with the beam span l in *equation 4.8*. Ratios for l_{ef}/l for different conditions can be found in *table 4.2*. Those ratios are valid for beams with restrained supports and loaded at the centroidal axis. For load application at the compressed beam edge, l_{ef} should be increased by $2h$, for load application at the tension edge of the beam, l_{ef} should be decreased by $0.5h$. Beams that are braced at the compressed edge at certain intervals and with bracing of sufficient stiffness have an effective length of $l_{ef} = a$.

The relative slenderness ratio for bending $\lambda_{rel,m}$ can be calculated from the characteristic bending strength $f_{m,k}$ and the critical bending stress $\sigma_{m,crit}$:

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} \tag{4.9}$$

Table 4.2 **Effective length as a ratio of beam span**
Beam loaded at centroidal axis.

Beam type	Loading type	l_{ef}/l
Simply supported	Constant moment	1.0
	Uniformly distributed load	0.9
	Concentrated force at mid-span	0.8
Cantilever	Uniformly distributed load	0.5
	Concentrated load at the free end	0.8

For load application at the compressed beam edge, l_{ef} should be increased by $2h$, for load application at the tension edge of the beam, l_{ef} should be decreased by $0.5h$.

Table 4.3 **Values of k_{crit} as a function of $\lambda_{rel,m}$**

k_{crit}	Relative slenderness for bending $\lambda_{rel,m}$
1	$\lambda_{rel,m} \leq 0.75$
$1.56-0.75\lambda_{rel,m}$	$0.75 < \lambda_{rel,m} \leq 1.4$
$1/(\lambda_{rel,m})^2$	$\lambda_{rel,m} > 1.4$

When the slenderness ratio $\lambda_{rel,m} < 0.75$, beams are stable and not subjected to lateral instability. For $\lambda_{rel,m} > 1.4$, the failure mode is lateral-torsional buckling. For slenderness ratios in between these values, the failure is in bending after pronounced deformations in vertical and lateral directions. Depending on the slenderness ratio, the factor k_{crit}, accounting for lateral-torsional buckling, can be obtained from *table 4.3* and *figure 4.3*.

Critical buckling stresses for other than straight beam types, such as arches and beams with varying cross-section, are discussed in *Chapter 7, page 102*, and in *Chapter 11, page 153*.

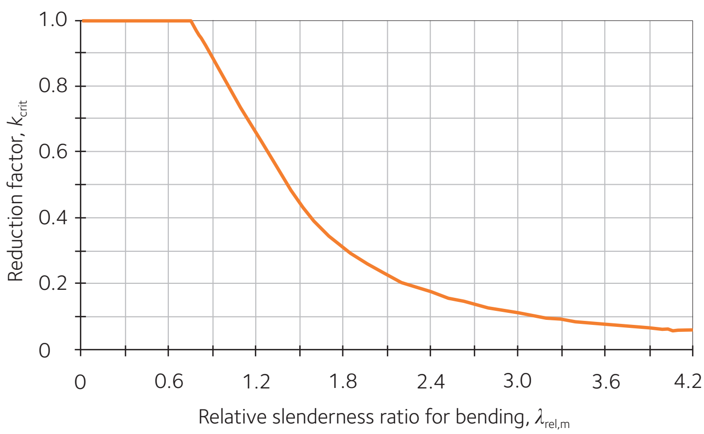


Figure 4.3 **Values of k_{crit} as a function of $\lambda_{rel,m}$; graphical representation**

In members only subjected to bending about the strong axis, the stresses should satisfy the following design expression:

$$\sigma_{m,d} \leq k_{crit} f_{m,d}$$

where $\sigma_{m,d}$ is the design bending stress, $f_{m,d}$ is the design bending strength and k_{crit} is a modification factor taking account of lateral-torsional buckling as defined in *table 4.3* and *figure 4.3*.

Members subjected to combined bending and axial compression force are treated in *section 4.2.5, page 67*, and those with compression perpendicular to grain in *section 4.1.4, page 58*.

Bracing of slender beams should be arranged either continuously or at certain intervals along the compressed edge, *see figure 4.4*. Continuous bracing can be achieved, e.g. by attaching a roof plate to the upper edge of a simply supported beam. Interval bracing can be done by purlins attached to the compressed edge of the beam. For continuous beams, bracing is necessary at the lower edge at intermediate supports, *see figure 4.5*. Bracing should be realized with e.g. wires running from the roof to the lower edge of the beam, thus not taking any compression forces. Otherwise, non-symmetrical loading on the roof and corresponding deflection of the roof could lead to the beam being pushed sideways at the lower edge. When a slender beam is braced against lateral buckling, the engineer should always consider the stiffness of the lateral bracing and the bracing of the whole system, not only the single beam, compare *figure 4.6*, showing different buckling lengths depending on the type of bracing of the entire system. In doubtful situations, it is best to assume no lateral bracing or to evaluate it as elastic restraint. Lateral bracing is very important, and many failures have occurred due to inadequate lateral bracing either during the erection phase (i.e. failure before bracing was put in place) or in the finished structure. Very stiff bracing could lead to progressive collapse (if one roof beam fails, it drags other roof beams with it), whereas bracing with adequate stiffness leads to robust structures. A more thorough discussion of bracing and its stiffness is given in *Chapter 13, page 170*.

4.10

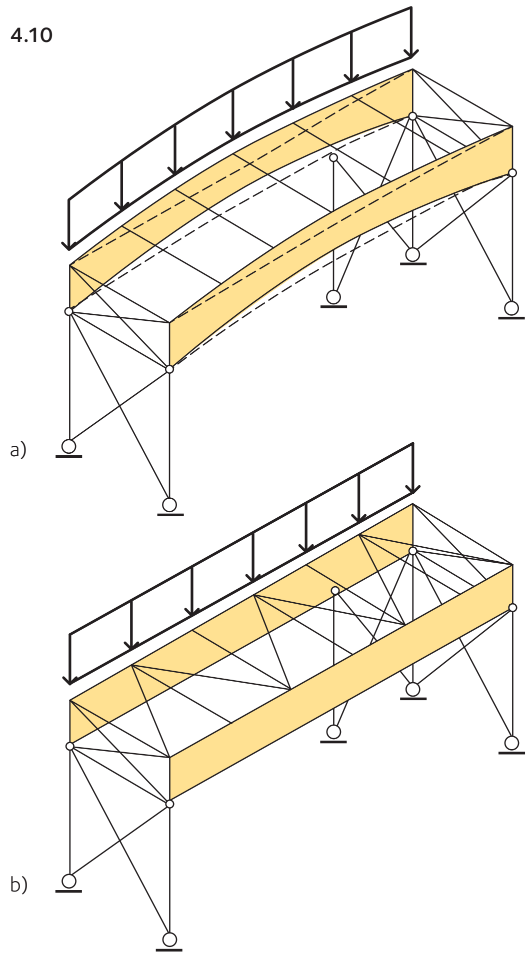


Figure 4.4 Lateral bracing of slender beams
a) Beams on fork supports, unbraced against buckling,
b) beams on fork supports, braced against buckling.

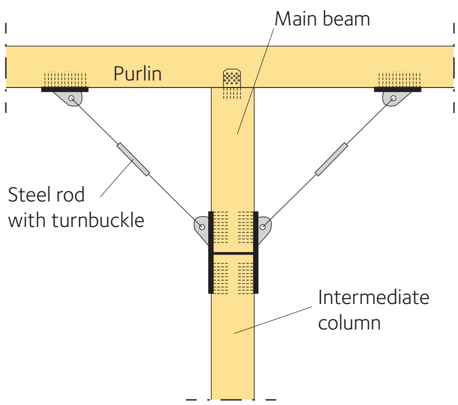


Figure 4.5 Lateral bracing of a slender beam at lower edge – above intermediate support – connected to roof structure. This bracing should not be able to take compressive forces. For non-symmetrical loading on the roof, which induces deflection of the roof on one side of the beam, and if the bracing can take compressive forces, then the bracing could push the beam over and lead to lateral buckling.

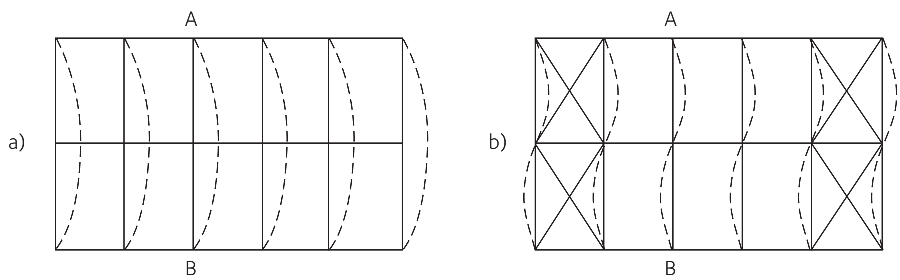


Figure 4.6 Lateral buckling of a roof or floor system
a) Unbraced,
b) braced.

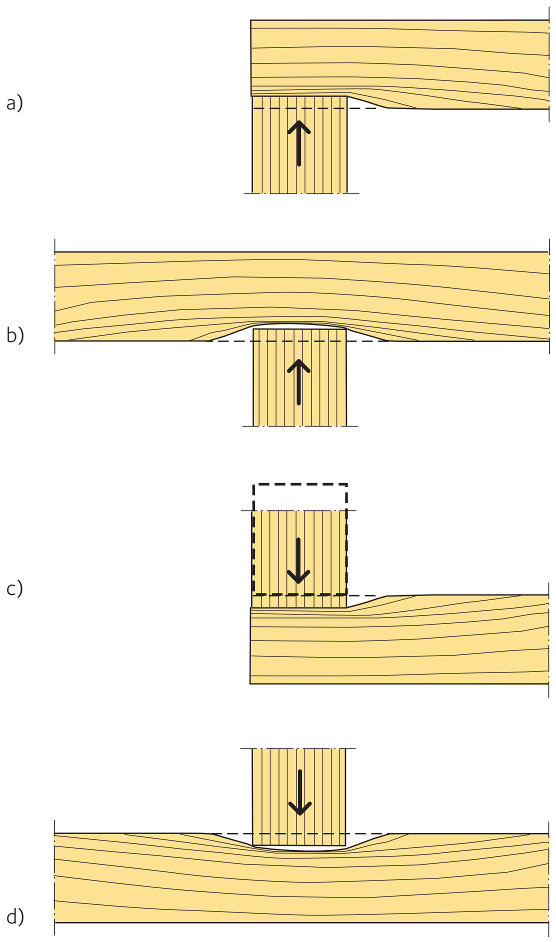


Figure 4.7 Bearing failure perpendicular to the grain
a) and b): Members on discrete supports,
c) and d): members on continuous support.

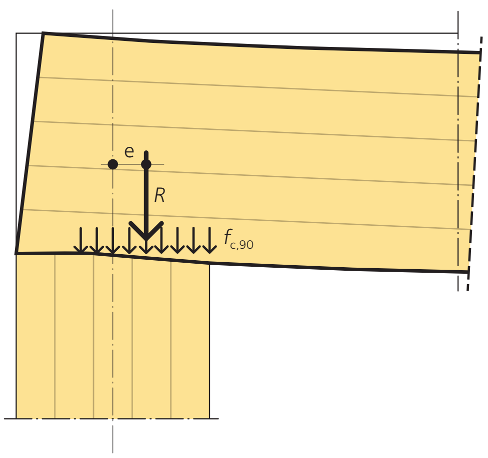


Figure 4.8 Large contact area, leading to non-uniform deformation and eccentric loading at the support

4.1.4 Compression perpendicular to the grain

In many structural applications, e.g. beam supports and multi-storey buildings, wood members are loaded perpendicular to their grain direction. Wood has low stiffness perpendicular to the grain and exhibits large moisture-induced movements (shrinkage and swelling) in this direction, which will lead to large deformations. Those are especially critical for multi-storey buildings, where deformations from several floors add up. Depending on the configuration (loading, supports), different failure types can be identified, *see figure 4.7*. There it is indicated that the load is usually not only transferred via the actual contact area, but via an effective contact area, enlarged by neighbouring "unstressed" wood regions.

Deformation due to compression perpendicular to grain does not lead to ultimate failure, but is more a serviceability limit state problem. However, in the codes, it has been chosen to treat this as an ultimate limit state problem. Thus, the design approach is to check the occurring stresses against the strength perpendicular to grain instead of using the stiffness values. Further, large reductions in strength due to service class (moisture) and load type (load duration) are used in the calculation of design strength (modification factor k_{mod}), *see sections 2.3.1, page 35, and 2.3.2, page 36, equation 2.9 and equation 4.12*. For strength perpendicular to grain we don't consider any size effect, i.e. the factor $k_h = 1$. Further, the stiffness perpendicular to the grain is more or less constant in the moisture range observed in buildings (moisture contents between 10 and 20 %) and the compression strength does only vary a little in this moisture range. However, this is not reflected in the design, which results in very large contact areas needed to fulfil the design equations. Large contact areas could then, due to non-uniform deformation, lead to eccentric loading, which is not considered in the calculations, *see figure 4.8*. An approach to overcome this problem is to use the characteristic value of the compression strength perpendicular to the grain instead of the corresponding design strength in the design equation (*equation 4.11*). This means, in other words, that both factors k_{mod} and γ_M shall be set equal to 1.0. The validity of such an approach should be limited, however, to timber structures with relatively low ratios of dead load to live load, say $g_k/q_k \leq 0.4$. As an example, consider a common timber structure, with $g_k/q_k \leq 0.4$, medium load duration and service class 1 or 2; the approach will lead to a reduction of the necessary contact area by 36 percent ($k_{mod}/\gamma_M = 0.8/1.25 = 0.64$) compared to the contact area which would be necessary if design strength was used. For timber structures subjected to heavier permanent loads ($g_k/q_k > 0.4$), however, the design value of compression strength perpendicular to the grain should be used.

Nowadays connections between beams and columns are reinforced in many applications. Reinforcement can e.g. be done with glued-in rods of steel or wood, self-tapping screws or outer nail plates, *see figure 4.9*. Reinforcement reduces deformation, thereby increasing the bearing capacity compared to a non-reinforced set-up. For self-tapping screws, possible failure modes include screws pushing into the timber, buckling of screws, and failure by reaching the compressive strength perpendicular to grain in the timber in the plane, where the screw tips are located. Eurocode 5 does not provide design equations for reinforced members loaded in compression perpendicular to the grain. However, there are calculation models that describe the capacity of a reinforced member as the sum of the capacity of the timber and the capacity of the reinforcement, such as those used by wood screw manufacturers.

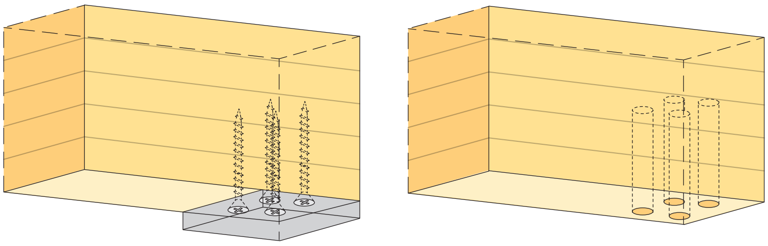


Figure 4.9 Example of a beam support reinforced for compression perpendicular to grain with self-tapping screws (left) and wooden glued-in rods (right), respectively

To calculate the capacity of the reinforcement, all possible failure modes have to be considered, and the different failure modes depend on the geometry of the beam and the number of reinforcing elements. In reinforced beam supports with few short screws, the load-carrying capacity of the reinforced beam support is characterised by pushing the screws into the timber. Simultaneously, the compressive strength perpendicular to the grain at the contact surface is reached. The pushing-in capacity of screws is considered equal to the withdrawal capacity. In beam supports with slender screws, buckling of screws occurs. Simultaneously, as in the first case, the compressive strength perpendicular to the grain at the contact surface is reached. In beam supports with many short screws, the load-carrying capacity is characterised by reaching the compressive strength of timber perpendicular to the grain in a plane formed by the screw tips.

Shopping centre, Hemavan, Sweden.

Design approach

The design approach for compression perpendicular to grain is to fulfil the design equation:

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} \leq k_{c,90} \times k_1 \times f_{c,90,d} \quad (4.11)$$

where:

$\sigma_{c,90,d}$ is the design compressive stress in the effective contact area perpendicular to the grain.

$F_{c,90,d}$ is the design compressive load perpendicular to the grain.

A_{ef} is the effective contact area in compression perpendicular to the grain.

$k_{c,90}$ is a factor taking into account load configuration, possibility of splitting and degree of compressive deformation.

k_1 is a factor that takes into account the ratio between permanent load and live load g_k/q_k.

Note that Eurocode 5 assumes $k_1 = 1$, for all cases.

Values for k_1:

$g_k/q_k \leq 0.4$: $k_1 = \dfrac{f_{c,90,k}}{f_{c,90,d}}$

$g_k/q_k > 0.4$: $k_1 = 1$

$f_{c,90,d}$ is the design compressive strength perpendicular to the grain.

$$f_{c,90,d} = \frac{k_{mod} \times f_{c,90,k}}{\gamma_M} \quad (4.12)$$

For cases where compression of the glulam can be considered to affect the load-bearing capacity (e.g. local pressure in trusses) or where deformations have a significant effect on the function (e.g. in buildings with more than two floors), $\gamma_M = 1.25$ should be used. For glulam structures in climate class 3, it is recommended that k_{mod} is selected according to *table 2.4, page 36*.

The effective contact area A_{ef} needed to calculate the occurring stress should be determined by taking into account an effective contact length l_{ef}:

4.13
$$l_{ef} = l + l^* \text{ where } l^* = \min \begin{cases} 30 \text{ mm} \\ a \\ l \\ l_1/2 \end{cases}$$

with notations according to *figure 4.10*.

Equation 4.13 applies to end supports where the outer support is in contact with the supported structure. For other supports where the supported structural element has a length of at least l^* on both sides of the support, l^* may be multiplied by the factor 2 in *equation 4.13*. It is assumed that the width of the support is at least equal to the width of the supported structural element.

Values for $k_{c,90}$ for different configurations can be found in *table 4.4*.

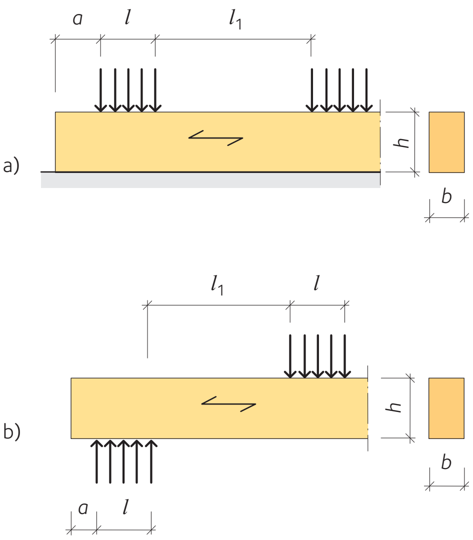


Figure 4.10 Member loaded in compression perpendicular to the grain, placed on a continuous support a) or on discrete supports b).

Table 4.4 Values for factor $k_{c,90}$ depending on type of support and distance l_1 between loads. Values in parenthesis are for solid structural timber, the other values for glulam. Notations according to *figure 4.10*.

Type of support	$l_1 < 2 \times h$	$l_1 \geq 2 \times h$
Member on continuous support	1.0	1.5 (1.25)
Member on discrete supports	1.0	1.75 (1.5) if $l \leq 400$ mm [1)]

[1)] If $l > 400$ mm, the effective length can be taken as $l_{ef} = 400 \text{ mm} + l^*$, and the magnification factor can be set to $k_{c,90} = 1.75$. Support lengths with $l > 600$ mm are not recommended.

Note that Eurocode 5 recommends $k_{c,90} = 1.75$ only if $l \leq 400$ mm. If $l > 400$ mm, Eurocode 5 recommends $l_{ef} = l$ and $k_{c,90} = 1.0$.

4.1.5 Compression stresses at an angle to the grain

Compressive stresses at an angle α to the grain should satisfy the following design expression:

$$\sigma_{c,\alpha,d} = \frac{F_{c,\alpha,d}}{A_{ef}} \leq f_{c,\alpha,d} = \frac{f_{c,0,d}}{\dfrac{f_{c,0,d}}{k_{c,90} \times k_1 \times f_{c,90,d}} \times \sin^2\alpha + \cos^2\alpha} \quad (4.14)$$

where:

- $\sigma_{c,\alpha,d}$ is the design compression stress at an angle α to the grain.
- $f_{c,\alpha,d}$ is the design compression strength at an angle α to the grain, *see figure 4.12*.
- $f_{c,90,d}$ is the design compression strength perpendicular to the grain.
- $f_{c,0,d}$ is the design compression strength along the grain.
- $F_{c,\alpha,d}$ is the design compression load at an angle α to the grain.
- $k_{c,90}$ is a magnification factor ($k_{c,90}$ = 1.75 if $l \leq$ 400 mm), *see figure 4.11*.
- k_1 is a factor that takes into account the ratio between permanent load and live load g_k/q_k.

  Note that Eurocode 5 assumes k_1 = 1, for all cases.

  Values for k_1:

  $$g_k/q_k \leq 0.4:\ k_1 = \frac{f_{c,90,k}}{f_{c,90,d}}$$

  $$g_k/q_k > 0.4:\ k_1 = 1$$

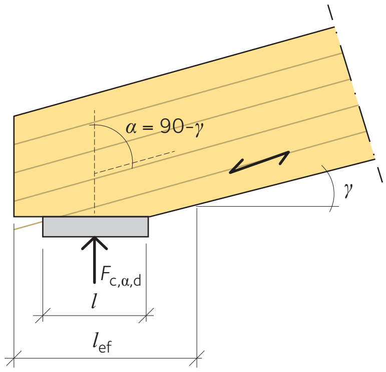


Figure 4.11 Member loaded in compression at an angle α to the grain

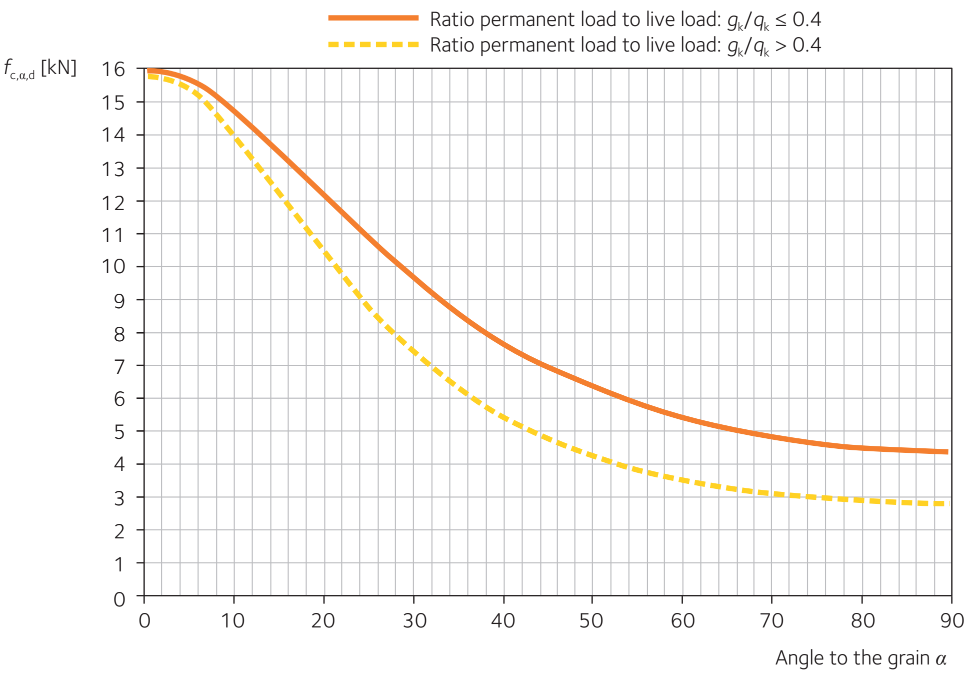


Figure 4.12 Strength for glulam member loaded in compression at an angle α to the grain
Glulam GL30c. Service class: 1 or 2. Load duration: medium term.

4.2 Axial buckling

The basis of column theory is the Euler column, a mathematically straight, prismatic, pin-ended, perfectly centrally loaded strut that is slender enough to buckle without the stress at any point of the cross-section exceeding the proportional limit of the material. The buckling load is defined as:

4.15
$$P_E = \pi^2 \frac{E \times I}{L^2}$$

where $E \times I$ is the bending stiffness and L is the length of the column.

4.2.1 Buckling lengths

The Euler load P_E is the reference value to which the strength of an actual column is usually compared. If an end condition other than perfect frictionless pins can be defined mathematically, the critical load is expressed by:

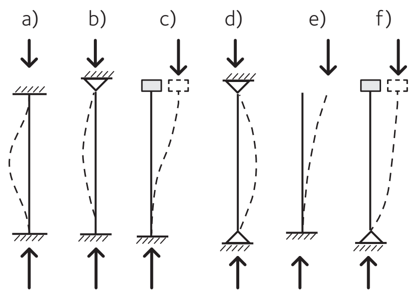
4.16
$$P_{cr} = \pi^2 \frac{E \times I}{(\beta \times L)^2}$$

where $\beta \times L$ is an "effective length" defining the portion of the deflected shape between points of zero curvature. In other words, $\beta \times L$ is the length of an equivalent pin-ended column buckling at the same load as the end-restrained column.

Figure 4.13 gives theoretical β values for idealized conditions in which the rotational and/or translational restraints at the ends of the column are either full or non-existent. At the base, shown for fixed conditions a), b), c) and e) in *figure 4.13*, the condition of full fixity is approached only when the column is anchored securely to a footing for which the rotation is negligible. Restraint conditions a), c), and f) at the top are approached when the top of the column is framed integrally to a girder several times more rigid than the column top. Column condition c) is the same as for a) except that transitional restraint is either absent or minimal at the top. Condition f) is the same as c) except that there is no rotational restraint at the bottom. The recommended design values of β are modifications of the ideal values, taking into account that neither perfect fixity, nor perfect flexibility is attained in practice.

The principle of using buckling length to determine the critical buckling load of a compression member can also be applied e.g. to non-prismatic columns, columns with non-constant compressive load, and columns with elastic restraints.

Buckled shape of column is shown by dashed line



	a)	b)	c)	d)	e)	f)
Theoretical β value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended β value	0.7	0.85	1.2	1.0	2.25	2.25

Explanation of end conditions

- Fixed joint
- Fixed joint (adjustable in horizontal direction)
- Hinged
- Free end

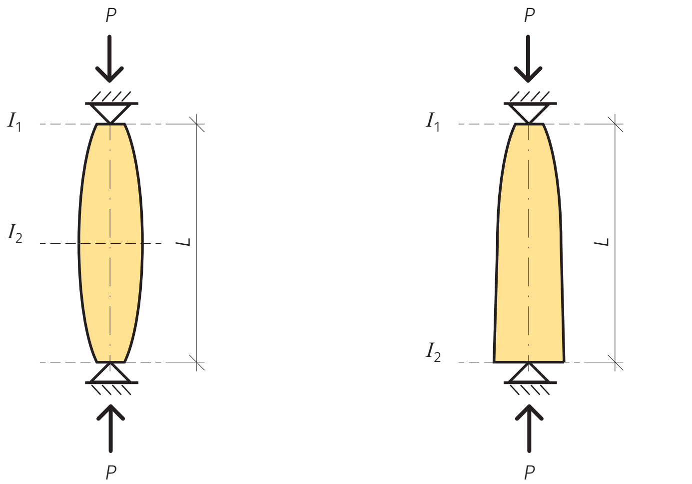
Figure 4.13 **Theoretical and practically applicable buckling lengths for columns with various end conditions**

4.2.2 Columns with varying cross-sections subjected to uniform and non-uniform compression load

For columns with a variation of the cross-sectional depth and/or columns subjected to non-uniform compression load, the buckling load can be approximately estimated by:

4.17
$$P_{cr} = \gamma \times \pi^2 \frac{E \times I_2}{L^2}$$

where I_2 is at the cross-section with the largest moment of inertia. For columns with a parabolic variation of the cross-sectional depth,



$$\gamma = 0.48 + 0.02 \times \sqrt{\frac{I_1}{I_2}} + 0.5 \times \sqrt[4]{\frac{I_1}{I_2}} \qquad \gamma = 0.18 + 0.32 \times \sqrt{\frac{I_1}{I_2}} + 0.5 \times \sqrt[4]{\frac{I_1}{I_2}}$$

Figure 4.14 **γ factor for determining the buckling load of columns whose cross-sectional depth has a parabolic variation**

the factor γ can be determined with the aid of *figure 4.14*; in such a case *equation 4.17* is valid within the range $0.1 \leq I_1/I_2 \leq 1.0$. *Equation 4.17* can also be used as a first approximate estimation of the buckling load of columns whose moment of inertia has a linear variation.

Figure 4.15 can be used to estimate the buckling load of columns with varying cross-sections and subjected to non-uniform compressive loading. The γ factor can be obtained as a function of the ratio between the smaller and larger moment of inertia of the column.

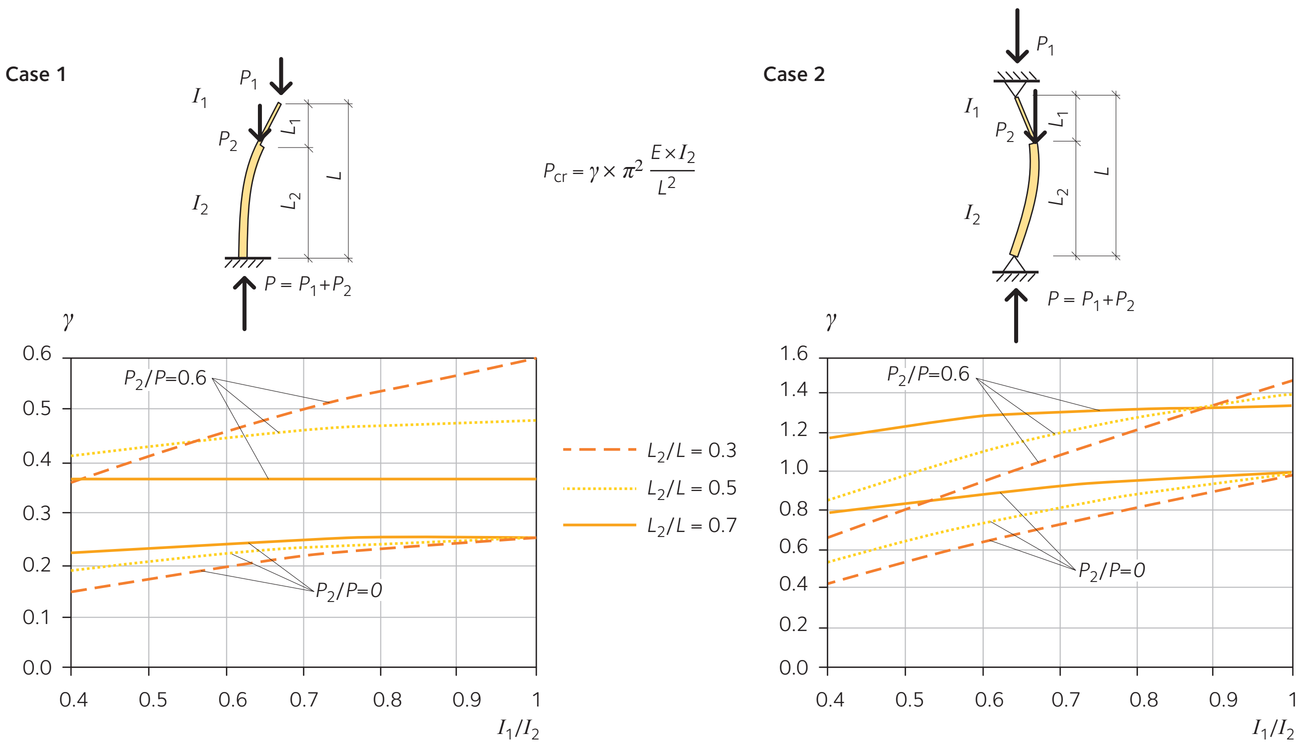


Figure 4.15 **γ factor for determining the buckling load of stepped columns with two different cross-sections and subjected to non-uniform compressive load.** Case 1 (left): Cantilever column. Case 2 (right): column with pinned ends.

Nordens Ark zoo, Hunnebostrand, Sweden.

4.2.3 Design approach

Geometric imperfections in the form of unavoidable out-of-straightness of the column and/or eccentricity of the axial load, will introduce bending from the onset of loading. Lateral deflection exists from the start of loading, and the failure load is reached when the internal moment capacity at the critical section is equal to the external moment caused by the product of the load and the deflection. The failure load thus depends on the imperfection. In general, the strength of timber columns must be determined by including both the geometric imperfections and the material variations, such as density, presence of knots, moisture content etc.

As a first approach, a column may be designed by means of a simplified second order analysis. For such a purpose, an initial value δ_0 for the out-of-straightness must be defined. For glulam structures a typical value is $\delta_0 = L/500$, where L is the length of the compressed member. In the case of a pin-ended column subjected to centric compressive load P, the design can be carried out by considering the interaction of compressive stresses $\sigma(P)$ and bending stresses $\sigma(M)$, *see figure 4.16*. Let the additional lateral displacement caused by P be denoted δ_p, *cf. figure 4.16*. The total lateral displacement δ_{tot} can be defined by:

4.18 $$\delta_{tot} = \delta_0 + \delta_p = \frac{\delta_0}{1 - P/P_{cr}}$$

where P_{cr} is the (theoretical) buckling load. The bending moment can then be calculated as the product of the applied axial load P and the final lateral deformation: $M = P \times (\delta_0 + \delta_p)$.

The design of the column is then carried out by checking that the combined effect of compressive stresses and the bending stress does not exceed the design strength of glulam.

However, this approach does not take into account some important factors, e.g. the possibility that timber yields at high compressive stresses.

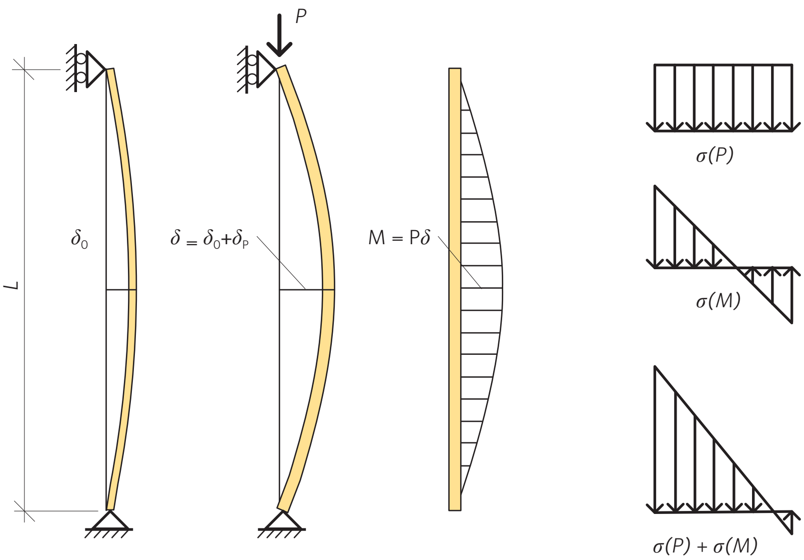


Figure 4.16 **Buckling of a timber column regarded as a nonlinear (second-order) problem**

4.2.4 Pure axial compression – designing according to EC5

Sports arena, Eksjö, Sweden.

The following parts of this section show how the minimum required strength of a glulam column can be determined according to Eurocode 5 (EC5). As for other Eurocodes, the design of columns according to EC5 is carried out on the basis of linear buckling analysis. The nonlinear (or second-order) effects are taken into account in the design by introducing a (strength) reduction factor k_c. The condition that shall be satisfied is:

$$\sigma_c = \frac{P}{A} \leq k_c \times f_{c,d} \qquad 4.19$$

where $f_{c,d}$ is the design value for the compression strength, A is the total cross-sectional area of the column and k_c is a reduction factor taking into account the risk of buckling.

The equation to calculate k_c was obtained by numerical simulations of a large number of columns having different imperfections of geometric and material properties, which are based on observations of real columns. Timber plasticity at the compression side was also taken into account in these numerical simulations.

The expression for the k_c values is given in modern design codes as a function of the relative slenderness ratio λ_{rel} defined as:

$$\lambda_{rel} = \sqrt{\frac{P_c}{P_{cr}}} = \sqrt{\frac{f_{c,0,k} \times A}{\pi^2 \frac{E_{0.05} \times I}{(\beta \times L)^2}}} = \frac{\lambda}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \qquad 4.20$$

where $f_{c,0,k}$ is the characteristic short term compressive strength of the timber parallel to the grain, and $E_{0.05}$ is the 5 % percentile of the modulus of elasticity parallel to the grain; λ is the slenderness of the column or strut, defined as indicated in *figure 4.17*.

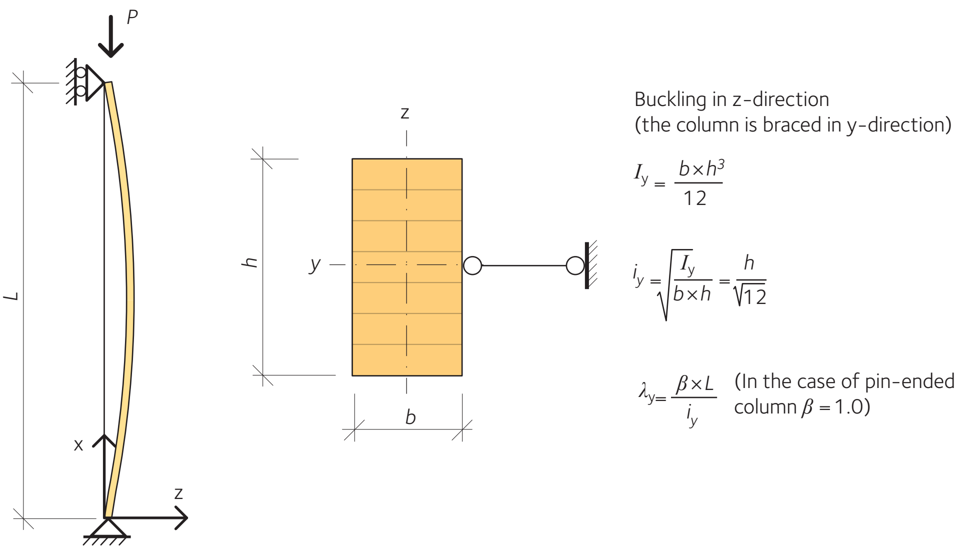


Figure 4.17 **Buckling of a timber column: definition of moment of inertia *(I)*, radius of gyration *(i)*, and column slenderness *(λ)***

Conservatory

The expression of the reduction factor k_c as a function of the relative slenderness ratio λ_{rel}, *cf. equation 4.20*, is given in the structural regulations and differs insignificantly between different codes. In EC5 the following applies:

4.21
$$k_c \begin{cases} 1 \text{ for } \lambda_{rel} \leq 0.3 \\ \dfrac{1}{k + \sqrt{k^2 - \lambda_{rel}^2}} \text{ for } \lambda_{rel} > 0.3 \end{cases}$$

where:

4.22
$$k = 0.5 \times \left[1 + 0.1\left(\lambda_{rel} - 0.3\right) + \lambda_{rel}^2\right]$$

The relationship between k_c and the relative slenderness ratio λ_{rel}, a so-called buckling curve, is shown in *figure 4.18*.

In practical applications, relative slenderness ratio λ_{rel} larger than 2.0 should be avoided.

Note that columns in external walls are often designed as being restrained from buckling in the weak direction, *see figure 4.19*, while internal columns normally are 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

4.23
$$\sigma_c = \frac{P}{A_{net}} \leq f_{c,d}$$

where A_{net} is the net cross-sectional area of the column.

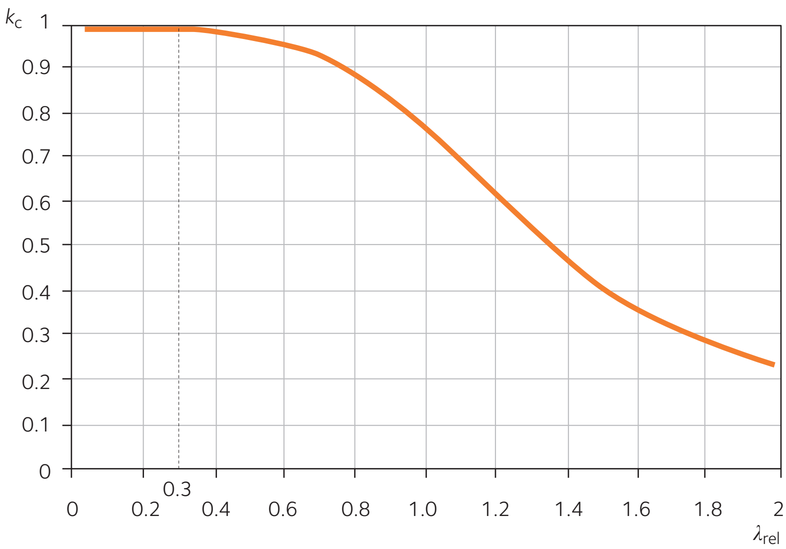


Figure 4.18 Reduction factor k_c as a function of the relative slenderness ratio λ_{rel} according to EC5

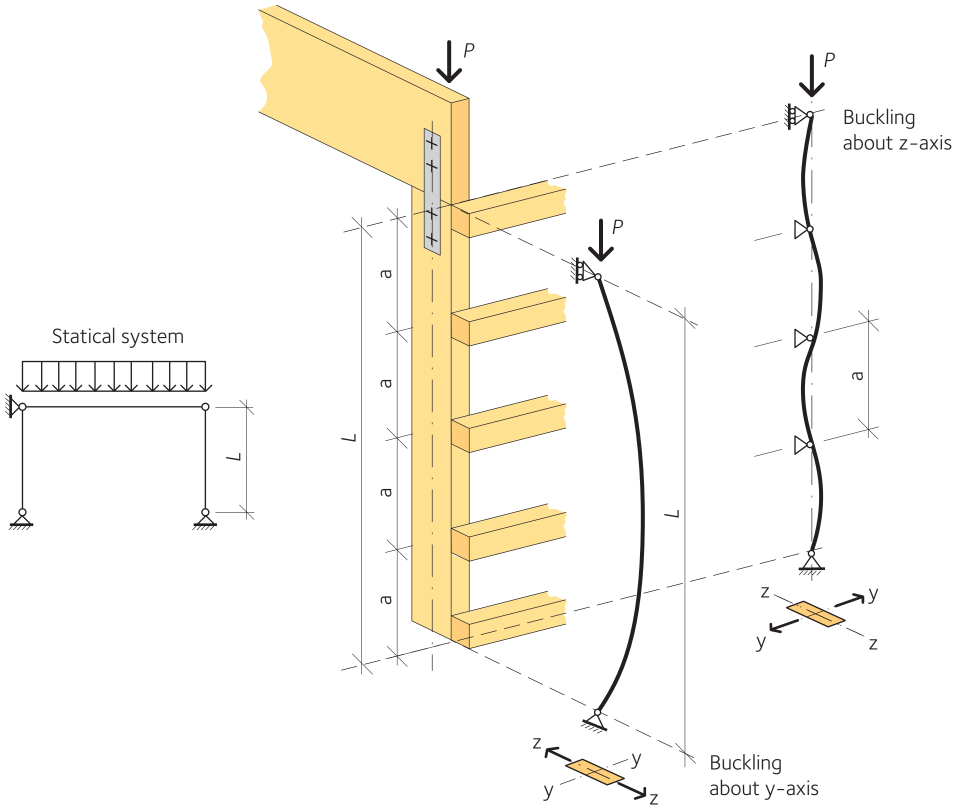


Figure 4.19 **Buckling of a timber column in an external wall**

4.2.5 Combined bending and axial compression

If lateral-torsional instability cannot occur, two failure modes can arise:

I) $\lambda_{rel} \leq 0.3$ – buckling behaviour is not relevant and failure will be based on the compressive strength of the member,

II) $\lambda_{rel} > 0.3$ – buckling can arise and failure will be based on the compressive strength of the member multiplied by the associated (instability based) reduction factor, i.e. k_c.

Theoretically, axial compression and biaxial bending can occur in a timber member. However, the most common load cases where compression and bending occur at the same time are those illustrated in *figure 4.20*. Only these cases will be discussed in the following paragraphs.

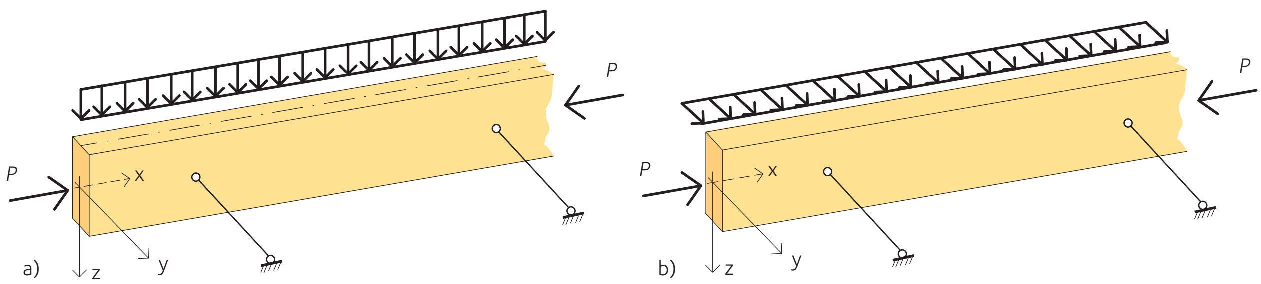


Figure 4.20 **Interaction bending and axial force**
a) Bending about the strong axis (y),
b) Bending about the weak axis (z).

Alpine Ski Lodge, Romme, Sweden.

Case I): $\lambda_{rel} \leq 0.3$

As there is no strength reduction due to buckling under this condition, advantage can be taken of the strength benefits associated with the plastic behaviour of timber, when subjected to compression stresses. The design conditions are:

4.24 $$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1$$ *Figure 4.20 a)*

4.25 $$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$ *Figure 4.20 b)*

where:

$\sigma_{c,0,d}$	is the design compression stress.
$\sigma_{m,y,d}$ and $\sigma_{m,z,d}$	are the design bending stresses about y and about z respectively.
$f_{c,0,d}$	is the design compression strength along the grain.
$f_{m,y,d}$ and $f_{m,z,d}$	are the design bending strengths about y and about z respectively.

Figure 4.21 shows the interaction diagram for a member subjected to combined bending moment and axial compression based on the application of plastic theory. When plastic theory applies, the material yields when it reaches the compressive strength allowing the stress in the section to extend over the surface and enhance its strength.

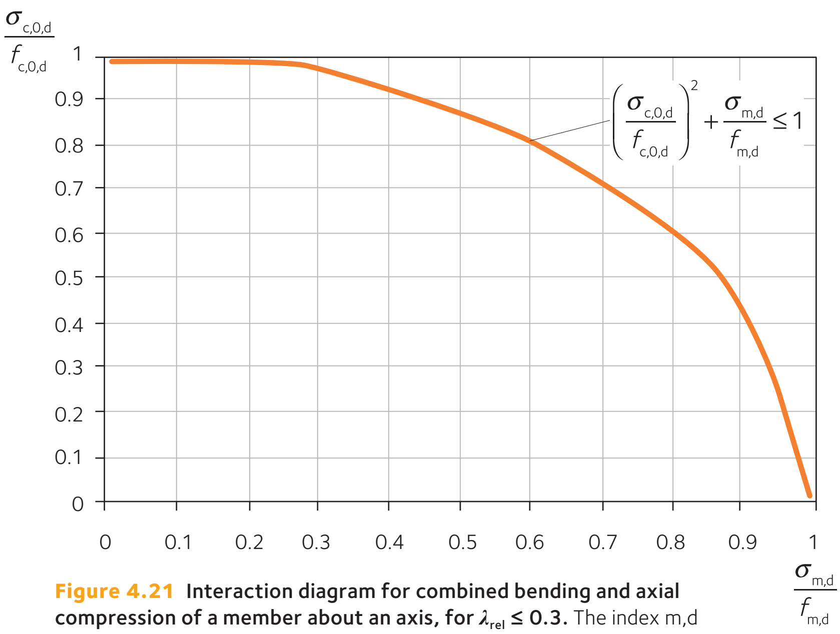


Figure 4.21 Interaction diagram for combined bending and axial compression of a member about an axis, for $\lambda_{rel} \leq 0.3$. The index m,d is used as a generic and refers to bending stress and bending strength respectively, caused by a bending moment either around "y" or around "z". The index c,0,d refers to axial compression.

Case II): λ_{rel} > 0.3
Under this condition, because axial load buckling effects have to be taken into account, no benefit is taken of any plastic behaviour in the member and the ultimate load is achieved when the material reaches its failure strength in the extreme fibre. The design conditions are:

$$\begin{cases} \dfrac{\sigma_{c,0,d}}{k_{c,y} \times f_{c,0,d}} + \dfrac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \\ \dfrac{\sigma_{c,0,d}}{k_{c,z} \times f_{c,0,d}} + 0.7 \times \dfrac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \end{cases} \quad \text{Figure 4.20 a)} \qquad (4.26)$$

$$\begin{cases} \dfrac{\sigma_{c,0,d}}{k_{c,y} \times f_{c,0,d}} + 0.7 \times \dfrac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \\ \dfrac{\sigma_{c,0,d}}{k_{c,z} \times f_{c,0,d}} + \dfrac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \end{cases} \quad \text{Figure 4.20 b)} \qquad (4.27)$$

Alpin Ski Lodge, Romme, Sweden.

where $k_{c,y}$ and $k_{c,z}$ are the reduction factors taking into account the risk of buckling, *see equation 4.21*. The subscripts "y" and "z" indicate that buckling occurs about the y-axis and z-axis respectively. Other symbols are the same as used in *equation 4.24* and *4.25*.

Note Concerning the factor 0.7 in *equation 4.26* and *4.27* (interaction of transversal load and axial compression): when the buckling induced by the axial force *P* occurs in a direction orthogonal to the transversal load, the bending stresses due to the transversal load may be reduced by the factor 0.7.

Note that 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, for columns in external walls restrained by horizontal studs it is important to check if the slenderness in lateral buckling between the studs is greater than in buckling outwards from the wall.

Figure 4.22 shows the interaction diagram for a member subjected to combined bending moment and axial compression based on application of the theory of elasticity.

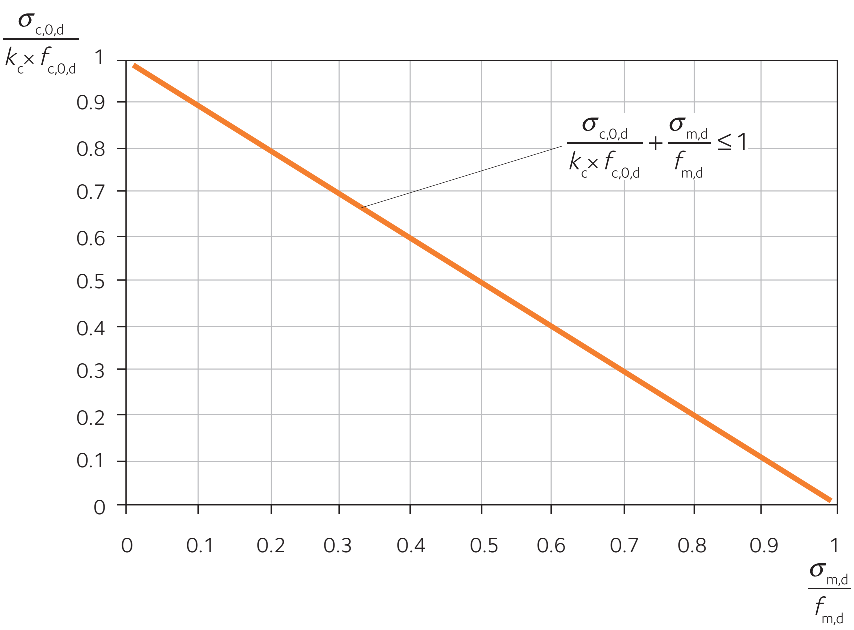


Figure 4.22 Interaction diagram for combined bending and axial compression of a member about an axis, λ_{rel} > 0.3
Elastic conditions. The index m,d is used as a generic and it refers to bending stress and bending strength respectively, caused by a bending moment either around "y" or around "z". The index c,0,d refers to axial compression.

Holes and notches

The introduction of a hole or a notch in a member constitutes a sudden change in the cross-section, which significantly influences the stress state and may reduce the strength in a timber member considerably. The flow of normal stress parallel to grain and of shear stress is disturbed. Instead, the local stress state will be characterized by concentrated, perpendicular-to-grain tensile stress and shear stress in the vicinity of the hole or the notch. This stress situation may lead to crack initiation for relatively low external loads and crack propagation in the fibre direction, which typically occurs in a very brittle manner. As for other applications with similar types of loading, experimental tests have shown a significant beam size dependency of the strength for both notched beams and beams with a hole.

Design considerations for unreinforced beams with a notch or hole at the beam end are treated in *section 5.1 and section 5.3, page 75,* respectively. Field notching on the tension side should not be allowed, unless the beam is properly reinforced. Due to the large strength reduction commonly related to beams with a notch or hole, some type of reinforcement is generally advisable. Reinforcement of end-notched beams is treated in *section 5.2, page 72,* and reinforcement of beams with a hole is treated in *section 5.4, page 79*.

5.1 End-notched beams

Notches at beam ends should be handled with great care in design, as even a small notch constitutes a starting point for a potential crack and may hence reduce the load carrying capacity considerably. For a beam with fibre direction coinciding with the beam length direction, an end notch at the tension side introduces concentrated tensile stresses perpendicular to grain and shear stresses, which according to linear elasticity tend to infinity at the tip of a right-angled notch. The failure type is typically very brittle, with crack initiation at the tip of the notch and propagation in the fibre direction. An end notch on the compression side gives a less severe reduction of the load-carrying capacity than an end notch on the tension side.

If notches cannot be avoided they should preferably, at least if placed on the tension side, be tapered or given a corner radius of at least 25 mm. Larger notches than $0.5h$ or 500 mm should not be allowed without reinforcement. Special care should be taken with structures where there is a risk of major variations in the moisture content. All surfaces in a notch shall be surface treated.

The load carrying capacity of an unreinforced beam with an end notch can be checked using the following method found in Eurocode 5, with notation according to *figure 5.1*. The method is based on a fracture mechanics analysis of right-angled notches presented by Gustafsson (1988). Although the design criterion formally reads as a comparison between a nominal shear stress τ and a reduced shear strength, the action of both perpendicular to grain tensile stress and shear stress is implicitly considered.

The decisive material properties in the fracture mechanics approach are the fracture energy in tension perpendicular to grain, the stiffness in beam length direction and the shear stiffness. These parameters enter the design equation through the factor k_n via assumptions of their relations with the shear strength f_v. The modification term related to the notch inclination is an addition to the original fracture mechanics equation, based on experimental tests of beams with tapered notches presented by Riberholt et al. (1992). For a beam with a rectangular cross-section and with fibre direction coinciding with the beam length direction, the following criterion should be fulfilled:

$$\tau = \frac{1.5V}{b_{ef}h_{ef}} \leq k_v f_v \tag{5.1}$$

where for beams with the notch at the compression side $k_v = 1.0$, while for beams with the notch at the tension side:

$$k_v = \min \begin{cases} 1 \\ \dfrac{k_n\left(1 + \frac{1.1 i^{1.5}}{\sqrt{h}}\right)}{\sqrt{h}\left(\alpha(1-\alpha) + 0.8\frac{x}{h}\sqrt{\frac{1}{\alpha} - \alpha^2}\right)} \end{cases} \tag{5.2}$$

and where:

V	is the shear force.
b_{ef}	is the beam width, *see section 4.1.1, page 53*.
h, h_{ef}	is the total beam depth and effective beam depth respectively, in mm.
x	is the distance from support load line of action to notch tip, in mm.
i	is notch inclination, *see figure 5.1*.
$\alpha = h_{ef}/h$	is the effective to total beam depth ratio.
$k_n = 6.5$	for glulam (4.5 for LVL and 5.0 for solid timber).


Sheep shed, Söderköping, Sweden.

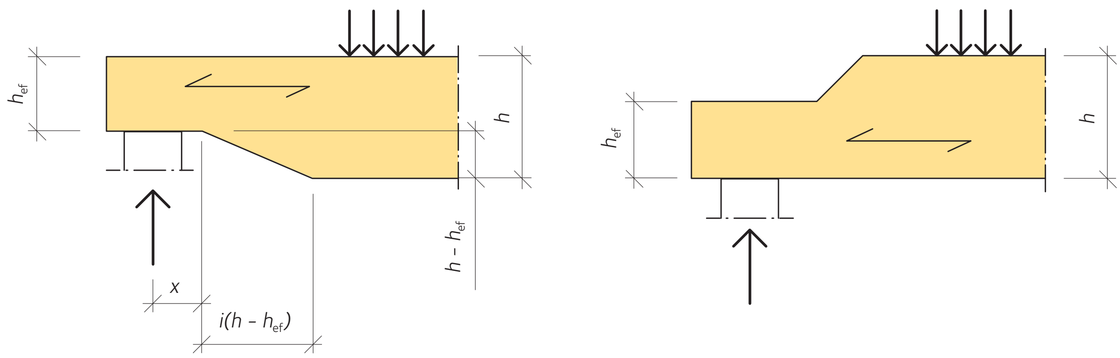


Figure 5.1 Notation for design of end-notched beams according to Eurocode 5

Swimming facility, Torsby, Sweden.

5.2 Reinforcement of end-notched beams

Beams with an end notch can be externally or internally reinforced in order to increase the beam capacity. Internal reinforcement may consist of glued-in threaded rods, glued-in concrete reinforcement bars or fully threaded screws. External reinforcement may consist of glued-on panels such as LVL or plywood, glued-on lamellas or pressed-in punched metal plate fasteners. Design approaches for external and internal reinforcement of beams with a rectangular cross-section and a rectangular notch (i = 0, *see figure 5.1*) are presented below, based on the German National Annex to Eurocode 5 (DIN EN 1995-1-1/NA). The basic idea is that the reinforcement should be designed to resist the entire force resultant of the damage relevant perpendicular to grain tensile stress along the potential crack plane starting from the corner of the notch. The perpendicular to grain tensile strength of the beam itself is neglected. The perpendicular to grain tensile force resultant is determined from integration of the beam theory shear stresses below the depth of the notch, as indicated in *figure 5.2*. A modification factor of 1.3 is applied to account for the discrepancy between the beam theory assumptions and the true behaviour, yielding the following expression for the tensile force resultant:

5.3 $$F_{t,90} = 1.3V\left(3(1-\alpha)^2 - 2(1-\alpha)^3\right)$$

where V is the shear force and $\alpha = h_{ef}/h$. Use of the modification factor 1.3 yields sufficiently accurate values for $x \leq h_{ef}/3$. For larger values of x, the expression given in *equation 5.3* may yield un-conservative values of $F_{t,90}$. The entire shear force V may then be assigned to $F_{t,90}$.

5.2.1 Internal reinforcement of end-notched beams

For glued-in rods it should be checked that the stress τ_{ef} in the glue "line" (cylinder), assumed to be evenly distributed, satisfies the following expression:

5.4 $$\tau_{ef} = \frac{F_{t,90}}{n \times d_r \times \pi \times l_{ad}} \leq f_{k,1}$$

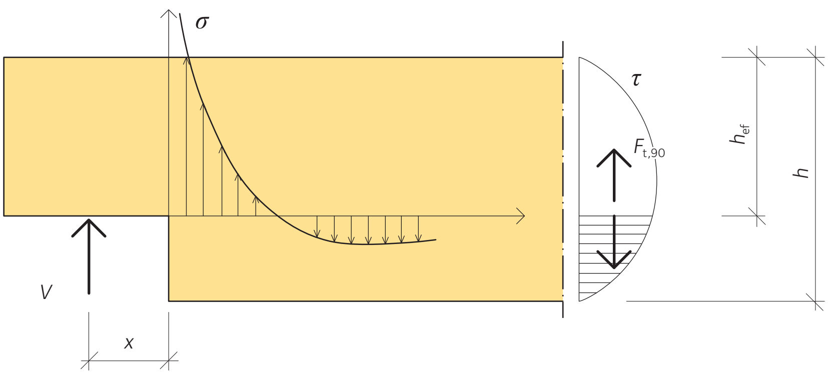


Figure 5.2 Schematic illustration of stress distribution at notch corner and illustration of tensile force resultant $F_{t,90}$

where:

$F_{t,90}$ is the force resultant of the perpendicular to grain tensile stress, *see equation 5.3*.

n is the number of rods, only one row in beam length direction may be considered active.

d_r is the outer thread diameter of the internal reinforcement, $d_r \leq 20$ mm.

l_{ad} is the effective anchorage length, *see figure 5.3*.

$f_{k,1}$ is the shear strength of the glue line, *see table 5.1* for characteristic values $f_{k,1,k}$.

The tensile axial capacity of the steel rods should also be checked.

Only one row of steel rods in the beam length direction should be considered as reinforcement. The minimum length of each steel rod is $2l_{ad}$ and the outer thread diameter is limited to $d_r \leq 20$ mm. Edge distances and spacing of the reinforcement elements should be such that $2.5d_r \leq a_{1,c} \leq 4d_r$ and $2.5d_r \leq a_{2,c}$ and $3d_r \leq a_2$ with notation according to *figure 5.3*. Since the perpendicular to grain tensile stress is highly concentrated to the vicinity of the notch corner, the edge distance $a_{1,c}$ should be kept as small as possible without violating the minimum required edge distance. For members exposed to tension parallel to grain, the reduction in net cross-section area due to the internal reinforcement should be considered. Fully threaded screws may also be used as internal reinforcement and should then be designed for the tensile force $F_{t,90}$ with respect to withdrawal and tensile axial capacity.

In addition to verification of the internal reinforcement capacity, the shear stress of the reduced cross-section should also be verified according to *equation 5.1* with $k_v = 1.0$. Attention should also be paid to the shear stress concentrations at the notch corner when using internal reinforcement.

Mountain station, Idre, Sweden.

Table 5.1 **Characteristic shear strength of glue line for glued-in rods when used for reinforcement**

Effective glued-in length l_{ad} of steel rod [mm]	≤ 250	$250 < l_{ad} \leq 500$	$500 < l_{ad} \leq 1000$
Characteristic shear strength $f_{k,1,k}$ of glue line [MPa]	4.0	$5.25 - 0.005l_{ad}$	$3.5 - 0.0015l_{ad}$

These values may be used if the applicability of the glue system has been proven (DIN EN 1995-1-1/NA).

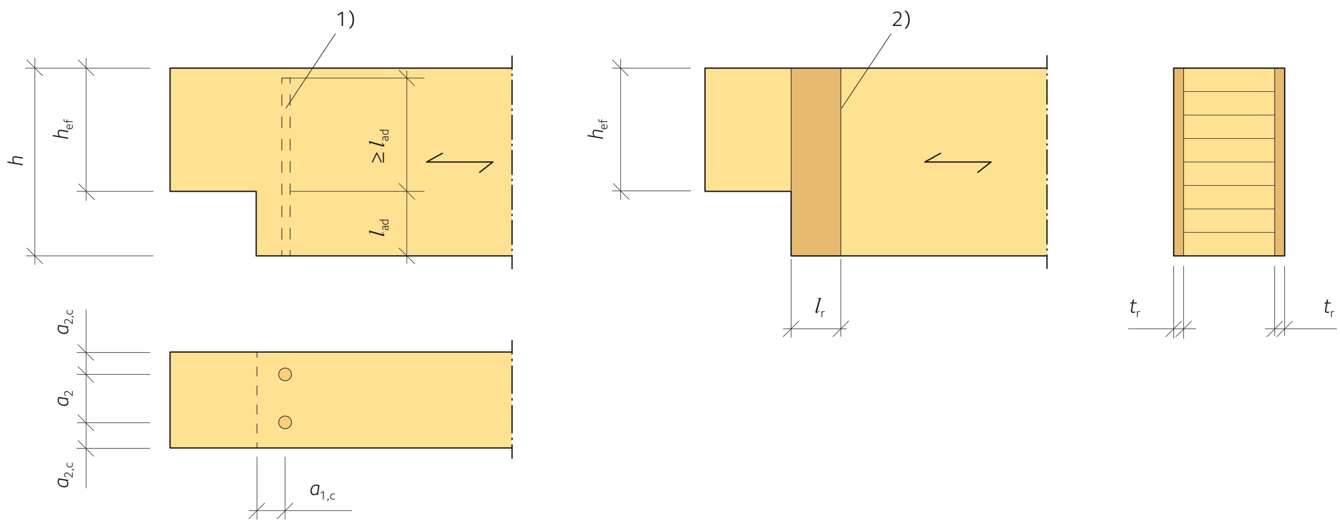


Figure 5.3 **Notation for 1) internal and 2) external reinforcement of end-notched beams (DIN EN 1995-1-1\NA)**

5.2.2 External reinforcement of end-notched beams

For glued-on panels it should be checked that the stress τ_{ef} in the glue line, assumed to be evenly distributed, satisfies the following expression:

5.5
$$\tau_{ef} = \frac{F_{t,90}}{2 \times (h - h_{ef}) \times l_r} \leq f_{k,2}$$

where:

$F_{t,90}$ is the force resultant of the perpendicular to grain tensile stress, *see equation 5.3*.
h, h_{ef} are the total beam depth and effective beam depth respectively, *see figure 5.3*.
l_r is the width of the reinforcement panels, *see figure 5.3*.
$f_{k,2}$ is the shear strength of the glue line. DIN EN 1995:1-1-1/NA states characteristic value $f_{k,2,k}$ = 0.75 MPa for glue systems which have been proven to be applicable.

The tensile stress σ_t in the panels should satisfy the following expression:

5.6
$$\sigma_t = \frac{F_{t,90}}{2 \times t_r \times l_r} \leq \frac{f_t}{k_k}$$

where:

$F_{t,90}$ is the force resultant of the perpendicular to grain tensile stress, *see equation 5.3*.
t_r is the thickness of one reinforcement panel, *see figure 5.3*.
l_r is the width of the reinforcement panels, *see figure 5.3*.
f_t is the tensile strength of the reinforcement panel in the direction of $F_{t,90}$.
k_k is a factor accounting for the non-uniform stress distribution. DIN EN 1995:1-1-1/NA states that k_k = 2.0 may be applied without further verification.

Reinforcement panels should be glued onto both sides of the member according to *figure 5.3* with panel width limited by $0.25 \leq l_r/(h - h_{ef}) \leq 0.5$ and panel thickness $t_r \geq 10$ mm. Sufficient pressure during gluing should be ensured, e.g. by the use of threaded nails or screws with appropriate anchorage length ($\geq 2t_r$) and spacing. The fasteners should be evenly distributed over the reinforcement panel and correspond in number to approximately 1 nail/wood screw per 6,000 mm^2, which equates to a nail/screw spacing of 75 mm.

Punched-in metal plate fasteners may also be used as external reinforcement, and should be designed in analogy with the above given recommendations.

In addition to verification of the external reinforcement capacity, the shear stress of the reduced cross-section should also be verified according to *equation 5.1* with k_v = 1.0. The capacity with respect to shear stress concentrations at the notch corner may be assumed to be sufficient when using external reinforcement designed in accordance with the above given recommendations.

5.3 Beams with a hole

Holes in members should preferably be avoided. A hole constitutes a sudden change in the cross-section that impedes the flow of forces in the member and generally reduces the beam strength considerably. For a beam loaded in bending with fibre direction coinciding with the beam length direction, the flow of parallel to grain normal stress and shear stress is disturbed and instead there appear concentrated perpendicular to grain tensile stress and shear stress in the vicinity of the hole. Such concentrated stresses also appear for members axially loaded in compression or tension. The magnitude and distribution of the unfavourable stress fields depend on many parameters, such as type of loading, hole shape, hole size and hole position relative to the beam depth direction. Schematic illustrations of the perpendicular to grain tensile stress distribution are shown in *figure 5.4* for a beam with a circular hole in different types of loading situations. The associated failure type, with crack initiation at the hole periphery and crack propagation in the beam direction, is typically very brittle.

If holes cannot be avoided there are some basic recommendations concerning hole shape and placing. Holes should preferably be placed at the neutral axis of the beam, especially holes placed in a position of dominating bending moment action. Circular holes are to be preferred over rectangular or quadratic ones. The sides of the hole should be surface treated to reduce variation in the moisture content and thus the risk of splitting. Hot pipes and ducts passing through holes shall be insulated. Holes should not be used in outdoor structures or elsewhere in places where there is a risk of large variations in the moisture content. Special care is necessary for members where the geometrical form itself causes perpendicular to grain tensile stress, for example in the apex region of double tapered beams. In curved structural members, including frame haunches and pitched cambered beams, holes should not be permitted at all. For beams with holes — as for end-notched beams and other applications where strength is limited predominantly by perpendicular to grain tensile stress — experimental tests show a significant beam size influence on the strength. Hence special attention should be paid when making holes in large members. Since the perpendicular to grain tensile stress is not limited to the absolute vicinity of the hole, the case of multiple holes in a beam should be treated with great care.

CNC machining of glulam beam.

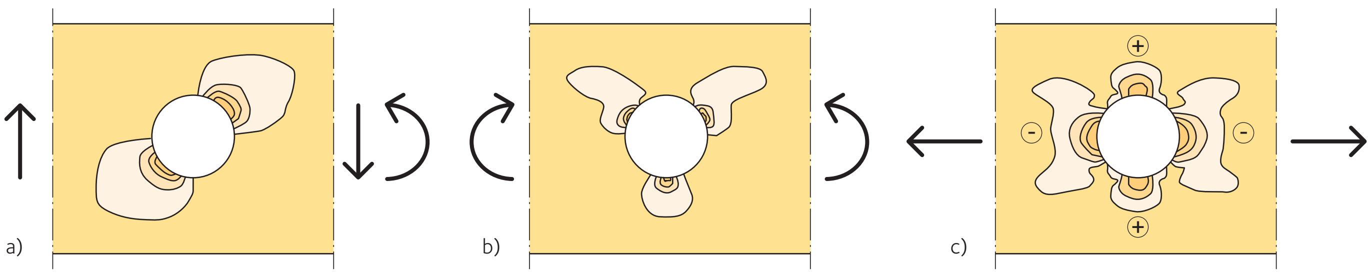


Figure 5.4 **Schematic illustrations of perpendicular to grain tensile stress distribution**
Hole placed in shear force dominated region (a), pure bending (b) and axially loaded member (c).


Machined glulam beam.

Design of unreinforced beams with a hole is a challenging task. Despite recent research efforts, there is at the moment no fully accepted and reliable design method that is based on a completely sound and rational mechanical background. The German National Annex to Eurocode 5 (DIN EN 1995-1-1/NA) does, however, state design equations for unreinforced beams with a hole. The method is based on linear elastic stress analysis and equilibrium considerations and originates from the work presented by Kolb and Epple (1985), although simplifications and empirical modifications have been added over time. Due to the uncertainties related to strength and design of such beams with a hole, it is recommended to reinforce the beam if holes cannot be avoided. Unreinforced beams with a hole may only be used in service class 1 and 2, while properly reinforced beams with a hole may be used also in service class 3. Reinforcement of beams with a hole is dealt with in *section 5.4, page 79*.

Regulations concerning hole size and placement are stated in *table 5.2* with notation according to *figure 5.5*. Holes with a diameter or diagonal length $d \leq 50$ mm and $h_d \leq 0.15h$ may be treated as a reduced cross-section, if placed close to the neutral axis.

The design criterion, *equation 5.7, page 77*, reads as a comparison of perpendicular to grain tensile stress with the corresponding strength, modified by an empirically based beam depth factor. Perpendicular to grain tensile stresses appear on both sides of the hole, at different locations depending on the type of loading and hole shape. Potential crack planes are for circular and rectangular holes, respectively, assumed at locations according to *figure 5.6*. The tensile stress along these planes is further assumed to have a triangular distribution. The magnitude of the perpendicular to grain tensile stress is determined by its force resultant $F_{t,90}$ which in turn is determined based on contributions from the shear force and bending moment.

Table 5.2 Regulations concerning hole size and location for beams with circular and rectangular holes

$l_v \geq h$	$l_z \geq 1.5h$ or at least 300 mm	$l_A \geq 0.5h$	$h_{ro} \geq 0.35h$ $h_{ru} \geq 0.35h$	$a \leq 0.4h$	$h_d \leq 0.15h$	$r \geq 25$ mm

According to DIN EN 1995-1-1/NA with the exception of minimum hole corner radius r, where DIN EN 1995-1-1/NA states $r \geq 15$ mm.

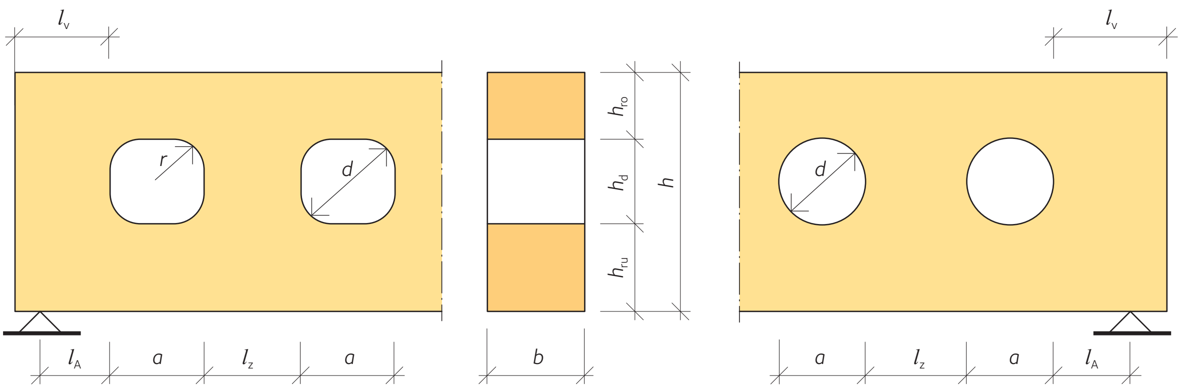


Figure 5.5 Notation for design of a beam with a rectangular or circular hole

The contribution $F_{t,90,V}$ from the shear force V is assumed to be equivalent to the integral of the beam theory shear stresses from beam mid-axis to the potential crack plane for a beam with a centrically placed hole, as illustrated schematically in *figure* 5.6. The contribution $F_{t,90,M}$ from the bending moment M is empirically based.

With a slight modification of notation compared to DIN EN 1995-1-1/NA, the design criterion is formulated as a comparison of perpendicular to grain tensile stress $\sigma_{t,90}$ and the corresponding strength $f_{t,90}$ according to:

$$\sigma_{t,90} = \frac{F_{t,90}}{0.5 l_{t,90} b} \leq k_{t,90} f_{t,90} \qquad 5.7$$

where:

$$F_{t,90} = F_{t,90,v} + F_{t,90,M} = \frac{V h_d}{4h}\left(3 - \frac{h_d^2}{h^2}\right) + 0.008\frac{M}{h_r} \qquad 5.8$$

and where for circular holes h_d may be replaced by $0.7h_d$ in *equation* 5.8 and

$$h_r = \min\begin{cases} h_{ru} \\ h_{ro} \end{cases} \quad \text{for rectangular holes.} \qquad 5.9$$

$$h_r = \min\begin{cases} h_{ru} + 0.15h_d \\ h_{ro} + 0.15h_d \end{cases} \quad \text{for circular holes.} \qquad 5.10$$

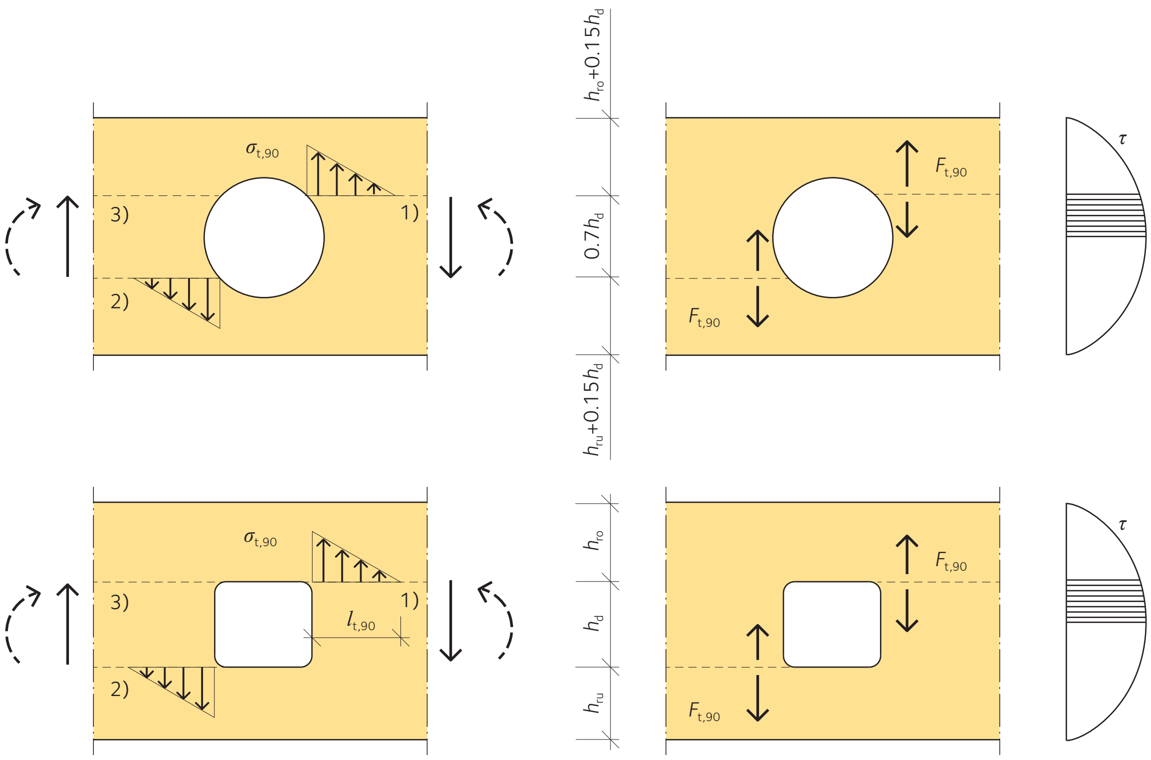


Figure 5.6 **Location of critical planes concerning cracking for rectangular and circular holes**
Planes 1) and 2) relevant for holes placed in shear force dominated region and planes 1) and 3) relevant when action is dominated by (positive) bending moment.

The length $l_{t,90}$ of the assumed triangular perpendicular to grain tensile stress distribution is given by:

5.11 $$l_{t,90} = 0.5\left(h_d + h\right) \qquad \text{for rectangular holes.}$$

5.12 $$l_{t,90} = 0.35h_d + 0.5h \qquad \text{for circular holes.}$$

and the strength reduction related to beam depth is given by:

5.13 $$k_{t,90} = \min\begin{cases}1\\ \left(450/h\right)^{0.5}\end{cases} \qquad \text{with } h \text{ in mm.}$$

In addition to the perpendicular to grain tensile stresses, in general being most relevant in design, shear stress concentrations also appear in the vicinity of a hole, especially for rectangular holes. The German National Annex to Eurocode 5 does not give explicit recommendations regarding this design issue. For rectangular holes, however, the maximum value of the shear stress at a hole corner may, according to Blaß & Bejtka (2003), be approximated with:

5.14 $$\tau_{corner} \approx \kappa_{corner} \times \frac{3V}{2bh} \quad \text{where} \quad \kappa_{corner} = 1.84 \times \frac{1 + a/h}{1 - h_d/h} \times \left(\frac{h_d}{h}\right)^{0.2}$$

where κ_{corner} expresses the increase of the maximum shear stress from the one in the conventional beam theory (for a beam without a hole). The exact shear stress is closely related to the hole corner radius and a smaller corner radius yields higher maximum shear stress. The approximation according to *equation 5.14* may yield un-conservative values for certain geometry and load configurations.

The capacity with respect to normal stress parallel to the grain, σ_0, due to bending moment M (and possibly also normal force N), should further be verified for the reduced cross-section. For rectangular holes, the additional bending stress in the upper and lower parts of the net cross-section with respect to their shear forces V_o and V_u and the lever arm $a/2$ should be taken into account, *see figure 5.7*.

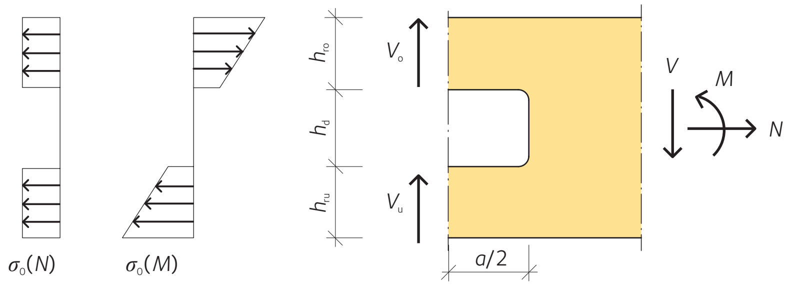


Figure 5.7 **Normal stress parallel to the grain for a beam with a hole**

5.4 Reinforcement of beams with a hole

Beams with a hole should in general be reinforced, since introduction of a hole commonly decreases the beam capacity significantly, and also since the design recommendations for unreinforced beams with a hole are related to uncertainties. Design recommendations for reinforcement of beams with a hole are given below, based on the approach in the German National Annex to Eurocode 5 (DIN EN 1995-1-1/NA). The design philosophy is equal to that for reinforcement of end-notched beams given in *section 5.2, page 72*; the reinforcement is designed to resist the force resultant of the perpendicular to grain tensile stress at a potential crack plane, while the perpendicular to grain strength of the beam itself is neglected. The perpendicular to grain tensile force $F_{t,90}$ may be approximated according to *equation 5.8, page 77*, and the potential crack planes are assumed to be located according to *figure 5.6, page 77*. Regulations concerning hole size and placing for a reinforced beam with a hole are given in *table 5.3*, with notation according to *figure 5.5, page 76*.

Hotel City, Gävle, Sweden.

5.4.1 Internal reinforcement of beams with a hole

Internal reinforcement may consist of glued-in threaded rods, glued-in concrete reinforcement bars or fully threaded screws. The beam should be reinforced with respect to the potential crack planes relevant for the specific loading condition according to *figure 5.6, page 77*. It should for the internal reinforcement on both sides of the hole be verified that the stress τ_{ef} in the glue line, assumed to be evenly distributed, satisfies the following expression:

$$\tau_{ef} = \frac{F_{t,90}}{n \times d_r \times \pi \times l_{ad}} \leq f_{k,1} \qquad 5.15$$

where:

- $F_{t,90}$ is the force resultant of the perpendicular to grain tensile stress, *see equation 5.8, page 77* .
- n is the number of rods, only one row in beam length direction may be considered active.
- d_r is the outer thread diameter, $d_r \leq 20$ mm.
- $f_{k,1}$ is the shear strength of the glue line, *see table 5.1, page 73*, for characteristic values $f_{k,1,k}$.

$l_{ad} = h_{ru}$ or h_{ro} for rectangular holes, *see figure 5.8, page 80*.

$l_{ad} = h_{ru}+0.15h_d$ or $h_{ro}+0.15h_d$ for circular holes, *see figure 5.8, page 80*.

Table 5.3 **Regulations concerning hole size and location for a beam with a reinforced circular or rectangular hole**

$l_v \geq h$	$l_z \geq 1.0h$ or at least 300 mm	$l_A \geq 0.5h$	$h_{ro} \geq 0.25h$ $h_{ru} \geq 0.25h$	$a \leq 1.0h$ $a \leq 2.5h_d$	$h_d \leq 0.30h$ [1)] $h_d \leq 0.40h$ [2)]	$r \geq 25$ mm

According to DIN EN 1995-1-1/NA with the exception of minimum hole corner radius r, where DIN EN 1995-1-1/NA states $r \geq 15$ mm.

[1)] Applicable to beams with internal reinforcement.
[2)] Applicable to beams with external reinforcement.

The tensile axial capacity of the rods should also be checked.

Only one row of steel rods in the beam length direction should be considered as reinforcement. The minimum length of each steel rod is $2l_{ad}$ and the outer thread diameter is limited to $d_r \leq 20$ mm. Edge distances and spacing of the internal reinforcement elements should be such that $2.5d_r \leq a_{1,c} \leq 4d_r$ and $2.5d_r \leq a_{2,c}$ and also $3d_r \leq a_2$, with notation according to *figure 5.8*. Since the perpendicular to grain tensile stress is highly concentrated to the vicinity of the hole, the edge distance $a_{1,c}$ should be kept as small as possible without violating the minimum required edge distance. Fully threaded screws may also be used as internal reinforcement and should then be designed for the tensile force $F_{t,90}$ with respect to withdrawal capacity and tensile axial capacity.

In addition, the shear stress concentrations at the hole corners should also be considered for beams with internal reinforcement and rectangular holes, *see section 5.3, page 75*. The capacity with respect to normal stress along grain should also be verified for the reduced cross-section at hole centre, *see section 5.3, page 75*.

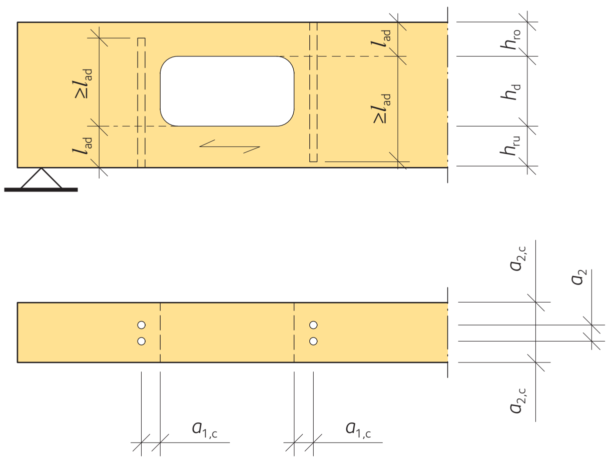


Figure 5.8 **Notation for internal reinforcement of a beam with a hole, crack planes relevant for shear force dominated loading**

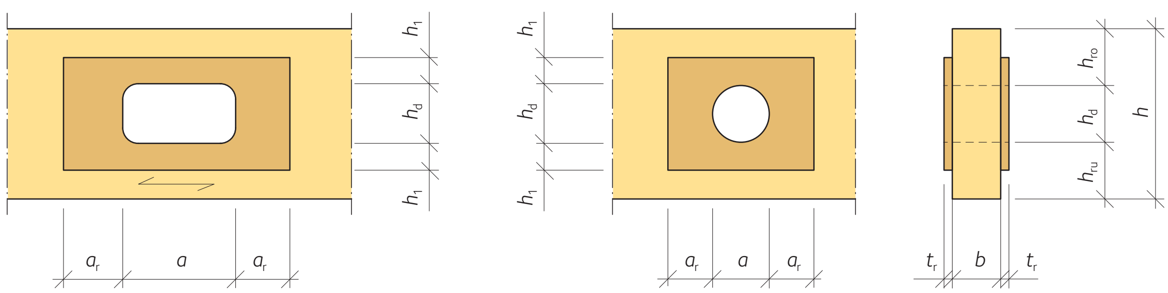


Figure 5.9 **Notation for external reinforcement of a beam with a hole**

5.4.2 External reinforcement of beams with a hole

External reinforcement may consist of LVL or plywood. It should be verified that the stress τ_{ef} in the glue line, assumed to be evenly distributed, satisfies the following expression:

$$\tau_{ef} = \frac{F_{t,90}}{2 \times a_r \times h_{ad}} \leq f_{k,2} \quad 5.16$$

where:

$F_{t,90}$ is the force resultant of the perpendicular to grain tensile stress, *see equation 5.8, page 77*.

a_r is effective length of reinforcement panels, *see figure 5.9, page 80*.

$h_{ad} = h_1$ for rectangular holes, with h_1 according to *figure 5.9, page 80*.

$h_{ad} = h_1 + 0.15h_d$ for circular holes, with h_1 and h_d according to *figure 5.9, page 80*.

$f_{k,2}$ is the shear strength of the glue line. DIN EN 1995:1-1-1/NA states characteristic value $f_{k,2,k}$ = 0.75 MPa for glue systems which have been proven to be applicable.

The tensile stress σ_t in the panels, glued onto the member, should satisfy the following expression:

$$\sigma_t = \frac{F_{t,90}}{2 \times t_r \times a_r} \leq \frac{f_t}{k_k} \quad 5.17$$

where:

$F_{t,90}$ is the force resultant of the perpendicular to grain tensile stress, *see equation 5.8, page 77*.

t_r is the thickness of one reinforcement panel, *see figure 5.9, page 80*.

a_r is effective length of reinforcement panels, *see figure 5.9, page 80*.

f_t is the tensile strength of the reinforcement panel in the direction of $F_{t,90}$.

k_k is a factor accounting for the non-uniform stress distribution. DIN EN 1995:1-1-1/NA states that k_k = 2.0 may be applied without further verification.

The reinforcement panels should be glued onto the member according to *figure 5.9, page 80*, with panel size limited by $0.25a \leq a_r \leq 0.3(h_d + h)$ and $h_1 \geq 0.25a$. The thickness t_r of the panels should be at least 10 mm. Sufficient pressure during gluing should be ensured, i.e. by the use of threaded nails or screws with appropriate anchorage length ($\geq 2t_r$) and spacing. The fasteners should be evenly distributed over the reinforcement panel and correspond in number to approximately 1 nail/wood screw per 6,000 mm^2, which equates to a nail/screw spacing of 75 mm.

The capacity with respect to normal stress along grain should also be verified for the reduced cross-section at hole centre, *see section 5.3, page 75*. The capacity with respect to shear stress concentrations at hole corner may however be assumed to be sufficient when using external reinforcement designed in accordance with the above given recommendations.

Swimming facility, Norrtälje, Sweden.

Serviceability limit state

Unlike strength considerations, it is very rare that serviceability considerations, such as functionality and appearance as well as comfort of the users, can alone lead to the collapse of a component or structure, but they are nevertheless very important in design for a number of reasons, e.g:

- To keep the visual appearance and functional requirements of the component or structure within acceptable limits.
- To prevent damage to brittle finishes, such as plaster, wallpaper and painting.
- To prevent undue deflection of roof structures so that, for instance, standing rainwater ponds will not cause leakage through the structure.
- To limit the effects of creep.
- To provide sufficient stiffness in the structure, so that vibrations do not lead to comfort problems.

6.1 Performance requirements

Deflections and movements in a building must be limited in order to avoid damage and other undesirable effects in service. A building must have sufficient strength to carry the maximum expected loads with an adequate margin of safety. Structures designed for the ultimate limit state may often be relatively slender, which means that they may be prone to large deformations. Therefore, the serviceability criteria used in design are of great importance, especially for lightweight structures such as timber and glulam.

Serviceability requirements will in many cases be decisive in the design process. This means that it is important that the magnitude of the deformations can be predicted with adequate accuracy, and that requirements and limits are correctly formulated. In the current design process this is a problem, since knowledge is often lacking with regard to both requirements and prediction methods.

Timber structures exhibit relatively large deflections when subjected to sustained load. Since wood is an anisotropic material, different loading modes will lead to different types of response. The sensitivity to environmental changes, for instance moisture induced movements, also has a significant effect on the deformations in structural glulam systems. Both long-term deformation (creep) and environmental effects must be considered when the serviceability behaviour of glulam structures is studied.

In principle, serviceability requirements should be set by the client in agreement with the builder and not by codes, although the codes provide a basis: general principles for loads, material parameters and calculation methods, which may be used for design in serviceability limit states.

Excessive deflection of structural components can be manifested by damage to structural as well as non-structural elements, by detrimental effects with regard to use and by various effects perceived as disagreeable by the persons occupying the building.

Deformation of a structural element seldom leads to failure of that single element, with the exception of deformations connected to the stabilising system. If a number of elements exhibit rather large deformations — or if the deformation in one element leads to subsequent movements in other elements — this may change the primary structural system or cause instability of the whole structure. Such system effects can also lead to unacceptable effects from the serviceability point of view. It is therefore of interest not only to investigate the behaviour of each element, but to consider the whole system, including the connections, in a deformation analysis. Such an analysis is, however, comparatively complex and is therefore only undertaken in very special cases.

Deformations may be unfavourable for the use of the building in many ways — for example, insufficient slope of roofs and problems with the opening of doors and windows. In special buildings such as gymnasiums or facilities with high requirements on the planeness of a surface, e.g. due to sensitive equipment, deflections may also lead to trouble. Serviceability limits should also be applied to buildings and their components in order to avoid the introduction of non-structural elements into the load path.

Excessive deflections of a structure do not look attractive to the public and may lead to a feeling of insecurity. These observations are subjective, and although the deflections will not endanger the structure or shorten its lifetime, it is appropriate to limit the deflection to ensure that confidence of the user is maintained. Such limits apply particularly to long-term deflections rather than the short-term, but recoverable, deflections that may occur due to high load peaks.

It is evident from the previous discussions that limitation of deformation is relevant in many different situations and for a variety of more or less important reasons. In practical design it is convenient to define two principally different categories of deformation control:

1) to avoid permanent damage and
2) to ensure a good appearance and general utility.

The designer may then refer each specific situation to one of these categories.

Problems with regard to serviceability also include vibrations of different origins. Vibrations due to footfall or machines of different types may lead to discomfort, but may also lead to problems for sensitive equipment and structural behaviour. The requirements in those cases are often even more complex than with static deflection. The same discussion as with static deformation is, however, valid in many cases, since it is a matter of usage of the structure that is decisive for the acceptable vibrations.

Gothenburg Central Railway Station, Sweden.

6.2 Static deformations

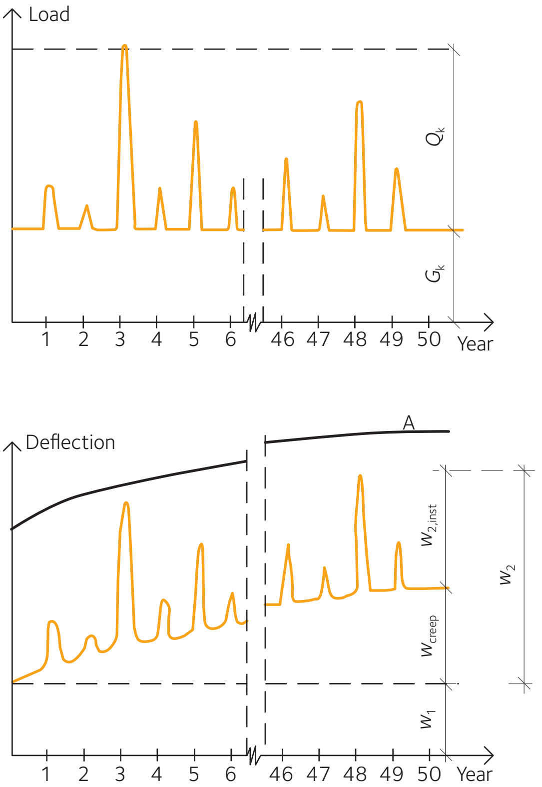


Figure 6.1 Time variation, in principle, for deflection (lower figure) of a beam with permanent and variable loads according to the upper figure. Curve A shows the creep deflection if the beam were loaded with the characteristic loads $G_k + Q_k$ during the whole period.

For most structures the loads consist of permanent loads G_i and variable loads Q_i. For timber and glulam structures, where variable loads often dominate, the deflection will fluctuate to a great extent during the lifetime of the structure.

Figure 6.1 illustrates in principle the deflection behaviour of a beam loaded with a permanent load G, and snow load Q. The total deflection can be subdivided into one part w_1 due to permanent load immediately after loading and one part w_2, which is variable during the lifetime of the structure. The variable part w_2 consists of a reversible portion $w_{2,inst}$ which is present only during those periods when the variable load is present, and a continuously increasing portion creep, which for all practical purposes may be considered as irreversible. Load peaks with short duration, such as those illustrated in *figure 6.1*, occur both for snow load and imposed (live) loads in the most common types of buildings.

6.2.1 Deformations

In *figure 6.2* a figure of principle as regards the deformation for a timber or glulam beam is given to show the different deflection parts. It can be assumed that the beam may have a pre-camber (which can be the case for a glulam beam), w_c. During short periods, there will be high load peaks, leading to a large deflection w_{inst}. This deflection may occur at any time during the service life of the structure.

One part of the load will be sustained for a longer time period and will give rise to a graduate increase in the deflection, w_{creep}. In order to determine the total deflection after a long time, $w_{net,fin}$, the sum of w_c, w_{inst} and w_{creep} can be calculated.

In the calculations of the deformations, the value of the modulus of elasticity – and in some cases the shear modulus – is used. The most reasonable value to use in the serviceability limit state is the mean value of each modulus.

When a member is subjected to bending, in addition to deformation due to the effect of the bending moment, it will also deform due to the effect of the shear forces, and the significance of the shear deformation will primarily be a function of the ratio of the modulus of elasticity E of the member to its shear modulus G and the depth to span ratio. Consider, for example, a simply supported rectangular beam of depth h and design span L carrying a concentrated load at mid-span. The ratio of the instantaneous deflection at mid-span caused by the shear forces, $w_{inst,s}$, to the instantaneous deflection at mid-span caused by the bending moment, $w_{inst,m}$, will be:

$$\frac{w_{inst,s}}{w_{inst,m}} = 1.2\frac{E}{G}\left(\frac{h}{L}\right)^2 \quad (6.1)$$

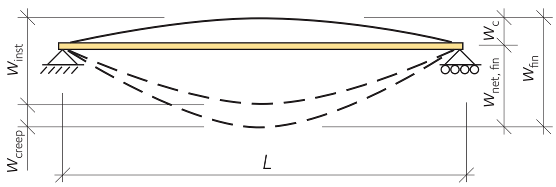


Figure 6.2 Definitions of deflections

For glulam, E/G is approximately 20 and for practical beam design, h/L will range between 0.1 and 0.05 resulting in a shear deformation between 5 and 20 percent of the flexural value. This shows that in some cases it can be of interest to take these deformations into account when designing glulam structures.

One special case of deformation is the settlement of a structure. This is often treated as an ultimate limit state design problem by checking the compressive stresses in contact areas where the glulam is compressed perpendicular to grain, as in Eurocode. It is, however, in practice often a serviceability problem, since focus should be on the settlements at the joints rather than on the actual failure of the joint. In a multi-storey wood frame building, forces of large magnitude can occur in the lower floors and at the foundation level. The compressive forces perpendicular to grain lead to deformations that may become relatively large, especially in comparison with deformations parallel to grain.

6.2.2 Long-term and climate effects

Creep behaviour, i.e. the increase in deflection with time, in timber and wood-related products, is a function of several factors. The magnitude of the creep depends on moisture content; the higher the moisture content, the larger the creep.

In practice there is also a problem with varying humidity, which influences behaviour, *see figure 6.3*. Variations in moisture content, even at relatively low moisture levels, lead to larger increases than at constant high humidity. Therefore it is important to try to estimate the moisture content levels and variations in the surrounding microclimate during the lifetime of the structure.

In addition to the effects of pure creep under constant load and creep under varying humidity there is the effect of varying load levels during the lifetime of the structure. All these effects have to be taken into account in the design process.

In principle, timber subjected to compression perpendicular to grain exhibits the same behaviour as a beam, but with some additional effects. The deformation in this case is further increased by wood shrinkage perpendicular to grain, which is far more severe than shrinkage parallel to the grain, which is negligible. The creep effects are also more severe than for a beam subjected to bending.

For a building subjected to a constant load over its lifetime, the creep deflection, w_{creep}, and the instantaneous deflection, w_{inst}, are related as follows:

$$w_{creep} = k_{def} w_{inst} \tag{6.2}$$

where k_{def} is a deformation factor whose value is dependent on the loaded material, its moisture content and the variations in moisture content.

For structures or members complying with the above conditions the final deformation, w_{fin}, can then be written as:

$$w_{fin} = w_{inst} + w_{creep} = w_{inst}\left(1 + k_{def}\right) \tag{6.3}$$

The final deformation under permanent and variable loading will then be as follows:

- For permanent actions G where the load is instantaneous, $w_{inst,G}$, the final deformation, $w_{fin,G}$ becomes:

$$w_{fin,G} = w_{inst,G} + w_{creep,G} = w_{inst,G}\left(1 + k_{def}\right) \tag{6.4}$$

- For variable actions, Q_i:

$$w_{fin,Q_i} = w_{inst,Q_i} + w_{creep,Q_i} = w_{inst,Q_i}\left(1 + \psi_2 k_{def}\right) \tag{6.5}$$

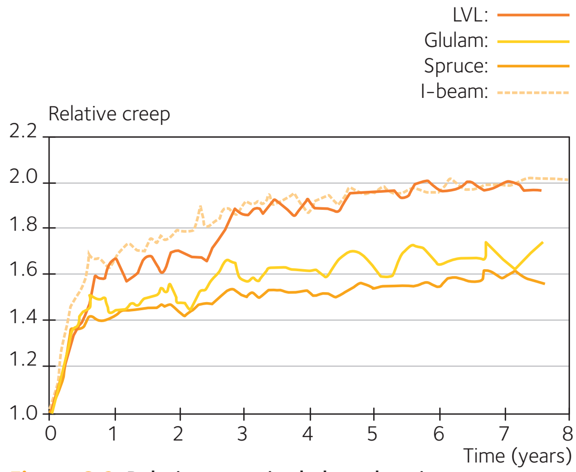


Figure 6.3 Relative creep in sheltered environment (relative creep = beam deflection at time *t* /initial deflection). Stress level 2 MPa. After Alpo Ranta-Maunus & Markku Kortesmaa (2000).

Detail of a joint at Malmö Central Station, Sweden.

The value of k_{def} is decided based on climate conditions. Since the creep effects also depend on the time period over which the load will be sustained, or in other terms on the mean value of the load, the factor ψ_2 is introduced to enable the description of this effect (ψ_2 = 1 for constant load). The values of k_{def} and ψ_2 are based on experimental results, but the uncertainties in the figures are relatively large. The value on k_{def} will increase with increasing humidity in the surrounding climate. The value of ψ_2 increases with the duration of time under load. It is then important to remember that variations in humidity as well as the maximum or average values of the relative humidity are of interest. If it seems reasonable to assume large variations in humidity will occur over time, it could be an idea to apply a more severe service class than when only the maximum humidity value is used.

For the final deformation analysis, the loading will be the same as that used for the instantaneous deformation, and the creep effect on displacement behaviour is achieved by using reduced stiffness properties:

6.6
$$E_{\text{fin}} = \frac{E}{1 + k_{\text{def}}}$$

where E_{fin} is the final mean value of the modulus of elasticity, E is the nominal mean value of the modulus of elasticity and k_{def} is the deformation factor for timber and wood-based products.

6.2.3 Load combinations

When calculating the deformation according to the previous sections, one has to define the relevant loads to be used. Three different load combinations can be formulated in order to define relevant load combinations. The formulations are given for the general case with several variable loads $Q_{k,i}$.

Characteristic combination

6.7
$$\sum_{j \geq 1} G_{\text{k,j}} + Q_{\text{k,1}} + \sum_{i>1} \psi_{0,\text{i}} Q_{\text{k,i}}$$

where $\psi_{0,i} Q_{k,i}$ is the characteristic combination value of the variable load. This combination gives a high value of the load and can normally be used in order to determine the short term deflection $w_{2,inst}$.

Frequent combination

6.8
$$\sum_{j \geq 1} G_{\text{k,j}} + \psi_{1,1} Q_{\text{k,1}} + \sum_{i>1} \psi_{2,\text{i}} Q_{\text{k,i}}$$

where $\psi_{1,1} Q_{k,1}$ is the frequent value of the load $Q_{k,1}$ and $\psi_{2,i} Q_{k,i}$ is the quasi-permanent value of the variable load $Q_{k,i}$. This is the combination to be used for the assessment of effects that are reversible, i.e. they occur with a certain frequency, but the effects will be reduced when the load decreases again.

Quasi-permanent combination

6.9
$$\sum_{j \geq 1} G_{\text{k,j}} + \sum_{i \geq 1} \psi_{2,\text{i}} Q_{\text{k,i}}$$

where $\psi_{2,i} Q_{k,i}$ is the quasi-permanent value of the variable load $Q_{k,i}$. This is the combination to be used for the assessment of long-term (creep) effects.

If *equations 6.3* to *6.5, page 85*, and *6.7* to *6.9, page 86*, are adopted, the final deformation under permanent and variable loading will be as follows:

For permanent actions, G:

$$w_{\text{fin,G}} = w_{\text{inst,G}} + w_{\text{creep,G}} = w_{\text{inst,G}} \left(1 + k_{\text{def}}\right) \tag{6.10}$$

For the leading variable action, Q_1:

$$w_{\text{fin,Q}_1} = w_{\text{inst,Q}_1} + w_{\text{creep,Q}_1} = w_{\text{inst,Q}_1} \left(1 + \psi_{2,1} k_{\text{def}}\right) \tag{6.11}$$

For accompanying variable actions, Q_i:

$$w_{\text{fin,Q}_\text{i}} = w_{\text{inst,Q}_\text{i}} + w_{\text{creep,Q}_\text{i}} = w_{\text{inst,Q}_\text{i}} \left(\psi_{0,\text{i}} + \psi_{2,\text{i}} k_{\text{def}}\right) \tag{6.12}$$

The final condition for permanent load and n variable actions will be:

$$w_{\text{fin}} = w_{\text{fin,G}} + w_{\text{fin,Q}_1} + \sum_{i=2}^{n} w_{\text{fin,Q}_\text{i}} \tag{6.13}$$

It is important to note that this load combination will give high values for the total deformation since the deformation from the variable load is calculated based on the characteristic value. This is reasonable if the control of deflection concerns permanent damage, since the effect of the high loads occurring for short time periods are of interest.

In the case of control with respect to appearance and utility it can be more suitable to calculate the deflection based on the frequent load combination *equation 6.8, page 86*, or the quasi-permanent load combination *equation 6.9, page 86*. The formulas *6.11* and *6.12* for calculating the final deformation from variable loads will then be modified. Using, for instance, the frequent load combination as a basis we get:

For the leading variable action, Q_1:

$$w_{\text{fin,Q}_1} = w_{\text{inst,Q}_1} + w_{\text{creep,Q}_1} = w_{\text{inst,Q}_1} \left(\psi_{1.1} + \psi_{2.1} k_{\text{def}}\right) \tag{6.14}$$

For accompanying variable actions, Q_i:

$$w_{\text{fin,Q}_\text{i}} = w_{\text{inst,Q}_\text{i}} + w_{\text{creep,Q}_\text{i}} = w_{\text{inst,Q}_\text{i}} \left(\psi_{2,\text{i}} + \psi_{2,\text{i}} k_{\text{def}}\right) \tag{6.15}$$

6.2.4 Limitation of deformations

Any specified deflection limit can be chosen due to functional reasons or for purely visual reasons. For example, it has been found by experience that beam deflections not exceeding $L/300$ are usually acceptable. This is often used as a value when designing in the serviceability limit state. It is, however, important to consider why a displacement control is done and on what type of structure and for which loads. If, for instance, the beam is pre-cambered to compensate for dead load deflection, the deflection limit should apply to movement under imposed loading only.

In the design of beams for storage and for roof structures, it is usually acceptable to permit deflections in the order of $L/200 - L/150$.

Production of glulam.

Vallen, Växjö, Sweden.

In *table 6.1* examples are given for different limit values in different situations. **Note** that these are only examples and that the present situation must be considered.

The following points should be considered in the decision of the deflection limit:

- The span,
- The type of structure and the usage,
- The possibility of damage to the ceiling or covering material,
- Aesthetic requirements,
- The number of times and length of time when maximum deflection is likely to occur,
- Roof drainage,
- The effect on such items as partitions over or under the position of deflection.

For different situations, it is important to define the problems that may arise due to deflections and whether they may lead to problems with regard to appearance or damage. Some examples are given here on combinations of limit values, load combinations and calculated deflection values:

- A non-bearing wall below the deflected beam may be damaged if the deflection of the beam is too large. Determine the required value on free space between the beam and the wall and use that as a limit. The design value of the deflection can be calculated with the load combination in *equation 6.7, page 86*. This can be seen as an example of control with respect to permanent damage.
- A floor in a living room without any sensitive materials and any risk of damage to adjacent structures can be checked with regard to long-term loads only. Large values of deformations for a shorter time period will not lead to any problems. In this case it can be enough to calculate the deformations with a quasi-permanent load combination, that is:

6.16
$$w_{\text{fin,qp}} = w_{\text{inst,G}} + w_{\text{creep,G}} + w_{\text{creep,Q}_1} + \sum_{i=2}^{n} w_{\text{creep,Q}_\text{i}} =$$
$$w_{\text{inst,G}}\left(1 + k_{\text{def}}\right) + \sum_{i=1}^{n} \psi_{2,\text{i}} k_{\text{def}} w_{\text{inst,Q}_\text{i}}$$

- In some cases, it can be more relevant to control the long-term deflections only, i.e. the extra deflection that occurs with time, omitting the instantaneous parts in the control. This can be the case when the beam has a pre-camber large enough to take care of the instantaneous effect of the permanent load. And also when the control is made with respect to appearance and utility and not damage.
- For cases where it is of interest to control deformations related to bearing stresses, these should be based on a serviceability load combination. Since the elastic deformations have been shown to be only a minor part of the deformation in most cases, short-term higher load levels will not affect the magnitude of the deformation significantly. The deformations depend to a great extent on the long-term behaviour and it is therefore reasonable to base the calculation on a quasi-permanent load combination. It is then also reasonable to use higher values for the factor k_{def} than those used for deflection controls, since the creep perpendicular to grain is higher than along the grain. In this case it can also be important to estimate the free shrinkage in the compressed part.

Table 6.1 Normally accepted limits for deformations in relation to the free span in the serviceability limit state
The table values are based on tried and tested methods and good engineering practice and have been converted to values in line with SS-EN 1990, SS-EN 1991, SS-EN 1995 and the current EKS. They should be seen as industry recommendations to guide developers and their agents, as well as a basis for evaluating competing alternatives.

Application	Non-cambered structural elements		
	$u_{max,inst}$	$u_{max,frekv}$	$u_{max,fin}$
Roof beams			
Manufacturing	*L*/300	*L*/300	*L*/250
Schools, shops, etc.	*L*/375	*L*/375	*L*/300
Animal sheds	–	*L*/200	*L*/200 (maximum 30 mm)
Machine halls, barns, etc.	–	*L*/150	*L*/150 (maximum 40 mm)
Floor beams			
General [1)]	*L*/500	*L*/375	*L*/300
Storerooms and other premises with no public access	*L*/275	*L*/250	*L*/200
Animal sheds	–	*L*/200	*L*/200 (maximum 30 mm)
Barns, etc.	–	*L*/150	*L*/150 (maximum 40 mm)
Trusses			
Generally, without consideration of node deformations	*L*/625	*L*/500	*L*/400
In agricultural buildings, without considering nodal deformations	–	*L*/400	–
Cantilevers	*L*/250	*L*/250	*L*/200
Roof ridges			
Generally, without a separate ceiling	*L*/375	*L*/375	*L*/300
In agricultural buildings, without a separate ceiling	–	*L*/200	–
Generally, with a separate ceiling	*L*/200	*L*/200	*L*/150
In agricultural buildings with a separate ceiling	–	*L*/100	–

1) The stiffness of wooden floors should also be checked regarding sagging and vibrations.

L denotes the free span. For structural elements with camber, the table value/1.5 applies.
$u_{max,inst}$ is calculated according to SS-EN 1990 (equation 6.14a), characteristic load combination and SS-EN 1995 (equation 2.2.3 (2)).
$u_{max,fin}$ is calculated according to SS-EN 1990 (equation 6.16a), quasi-permanent load combination and SS-EN 1995 (equation 2.2.3 (3) and (5)).
There are no instructions for the frequent load combination according to SS-EN 1990 (equation 6.15a) in SS-EN 1995. The frequent load combination $u_{max,frekv}$ is calculated according to *equation 6.8, page 86*.

The table must also be supplemented with the constraints imposed by the conditions of the construction project in question. For example, the outer layer of the roof may require a maximum deformation of 30 mm for the characteristic load, to avoid damage where a shallow pitch causes a risk of standing water freezing into ice. There is also a risk of damage to ceramic floors and stone tiles, where a stiffness of at least around *L*/300 is reasonable for the characteristic load. Lintels over doors and windows are examples of instances where absolute measures of deformation must not exceed the space available. Floors must also not place a load on non-load-bearing internal walls. Glass roofs are highly sensitive to vertical and horizontal movements. The values for agricultural buildings were chosen in accordance with SIS-TS 37:2012.

6.2.5 Methods to avoid deformation problems

In practice, it is impossible to avoid deformations when using glulam as a structural material. But with better prediction tools it ought to be possible to avoid deformations of a magnitude that could cause problems. In addition, to be able to predict the magnitude of the deformations during the service life, it is of interest to find design solutions that lead to smaller deformations. For beams, one important aspect is the effect of humidity variations. This means that one has to be aware of the climate conditions in the building where the timber is to be used. This is particularly important if the glulam is in a structure with different climates on different sides — for example with a climate separation structure. Different climate on the different sides may increase problems with deformation.

Villa Moelven, Nacka, Sweden.

Kolmården zoo, Sweden.

For joints subjected to compression, it is important to avoid compression perpendicular to grain, since this is the most sensitive direction both with regard to load and to effects of moisture variations. There may also be problems if different types of material are used in the building. There are good solutions to this, but if, for example, a hybrid frame using both glulam and concrete is chosen, it is important to pay extra attention to their respective deformation properties.

Another method to avoid the negative effect of moisture variations in glulam structures – and thereby decrease the deformations – is to use some sort of coating that makes it more difficult for the moisture to penetrate the glulam. This is not practical in most cases since it is laborious and uneconomical. Impregnation is another method that may be used, but as in the case with coating it is not very practical.

One common way to decrease negative effects of deformation is pre-camber, which induces a deformation in the direction opposite to the load-induced deflection, when producing the beam in the factory. The value of the pre-camber can be calculated from the load-induced deflection as already shown in this chapter.

It is also of major importance that the engineer really focuses on the question: what is the acceptable serviceability limit? In many cases the designer uses very simple rules of thumb for the limit values, which are adequate in some instances but too crude in others and can therefore lead to unnecessarily costly designs.

6.2.6 Calculation methods for different types of glulam structure

The calculation of the deflection is the same for all types of structure when it concerns the load combinations, *cf. section 6.2.3, page 86*. Shown here are some equations to be used for different types of structural element. In many cases computer-aided design is the best way to determine the deformation of a structure, but it is important to remember that when this is done, the load levels must be correct as well as the material properties, i.e. correct load combinations and mean values for elastic and shear moduli.

Straight beams of constant depth

Equations needed to calculate deflection of straight beams with constant depth with regard to bending are found in most design tables.

For beams where the ratio L/h is less than 10, it is of interest to determine the shear deformations in addition to the bending deflection; otherwise they can be ignored.

For the case with a simply supported beam subjected to uniformly distributed load, the deflection at mid-span is:

6.17 $$w_\mathrm{m} = \frac{5qL^4}{384EI}$$

where the index m indicates bending deflection. In this case, the shear deflection is given by:

6.18 $$w_\mathrm{s} = \left(1 + 0.96\left(\frac{E}{G}\right)\left(\frac{h}{L}\right)^2\right) \times w_\mathrm{m}$$

where the index s indicates shear deflection. The mean values should be used for both moduli, E and G.

As a rule, deflection is not a critical issue for continuous beams. The maximum deflection usually occurs in the end bays.

Straight beams with linear depth variation
Deflections in double-tapered and single-tapered beams can most easily be calculated by computer. Manual calculation with working equations is a lengthy process. Deflections in simply supported single-tapered beams, or symmetrical double-tapered beams, can be estimated using the following formula:

$$w = \frac{5qL^4}{384EI_e} + 0.35\frac{qL^2}{Gb\left(h_s + h_{max}\right)} \qquad 6.19$$

where:

I_e $= bh_e^3/12$.
h_e $= h_s + 0.33L \times \tan\alpha$ for double-tapered beams.
h_e $= h_s + 0.45L \times \tan\alpha$ for single-tapered beams.
h_{max} $= h_{ap}$ for double-tapered beams and h_l for single-tapered beams.

For beams where $2L/(h_s+h_{max}) > 25$ the second term in the above expression, which corresponds to the contribution of shear deformation to the deflection, can be ignored.

For more exact calculations the following equations and figures can be used, Jack Porteous & Abdy Kermani (2007).

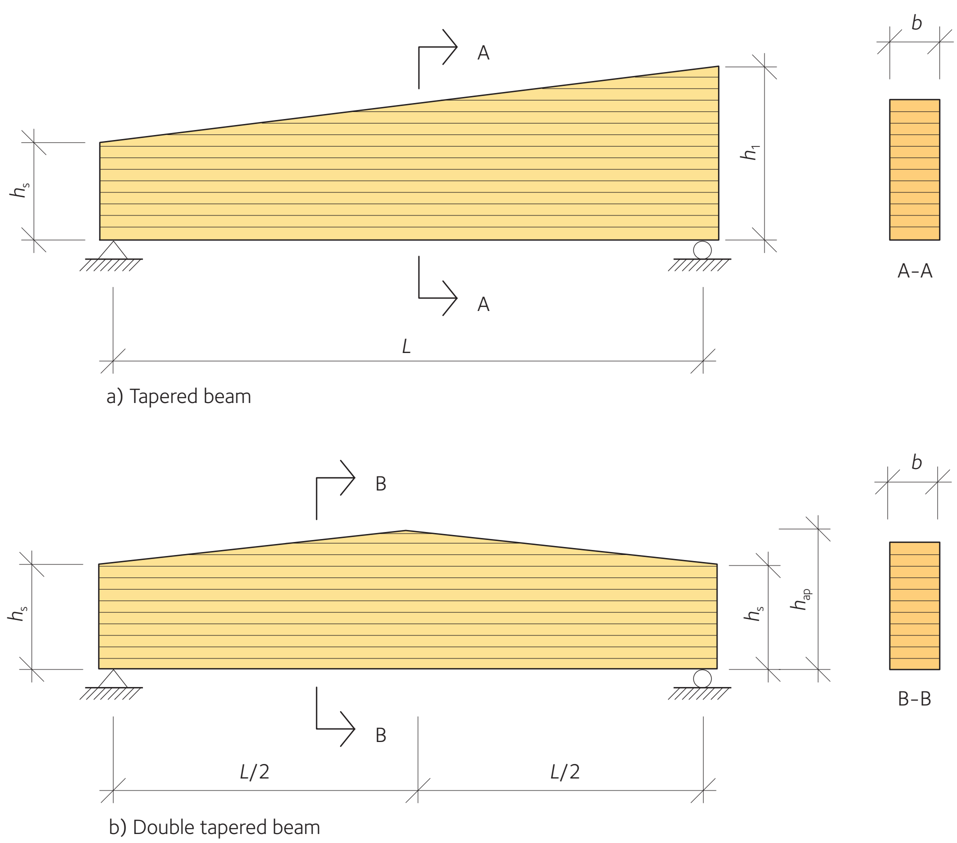


Figure 6.4 **Tapered beams**

Tapered beam with a concentrated load P at mid-span:

6.20
$$w_m = \frac{5ML^2}{96EI_{h_s}} k_{1\delta b} \quad \text{with} \quad M = \frac{PL}{4}$$

6.21
$$w_s = \frac{1.2M}{GA_{h_s}} k_{1\delta s}$$

Tapered beam with a uniformly distributed load:

6.22
$$w_m = \frac{5ML^2}{48EI_{h_s}} k_{2\delta b} \quad \text{with} \quad M = \frac{qL^2}{8}$$

6.23
$$w_s = \frac{1.2M}{GA_{h_s}} k_{2\delta s}$$

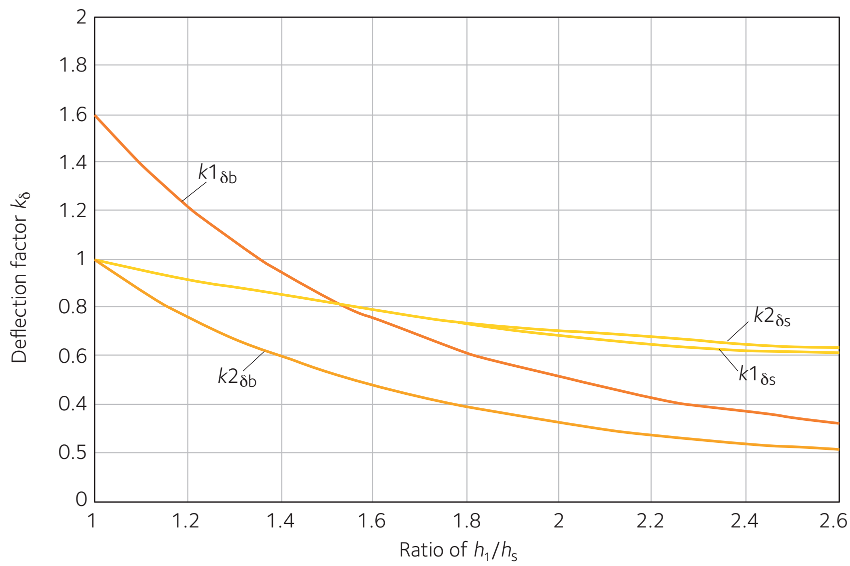


Figure 6.5 **Values of $k_{\delta b}$ and $k_{\delta s}$ for tapered beams**

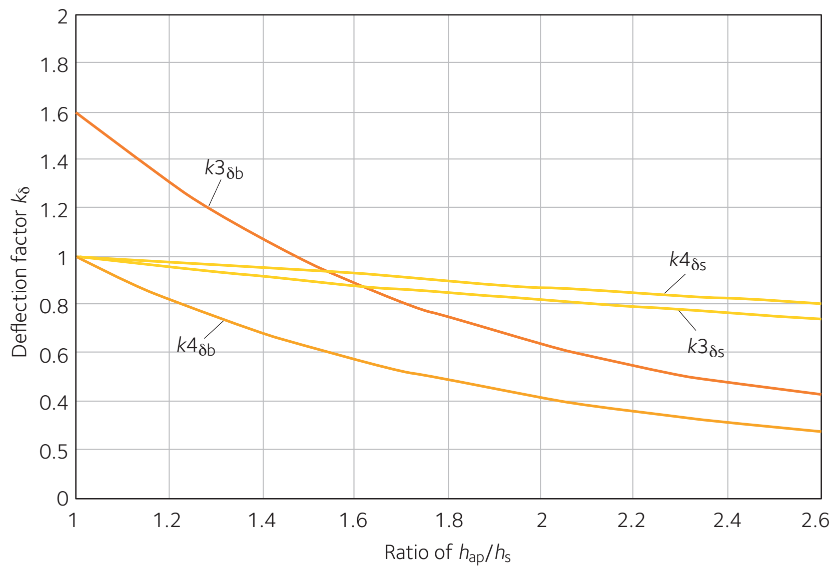


Figure 6.6 **Values of $k_{\delta b}$ and $k_{\delta s}$ for double-tapered beams**

Double-tapered beam with a concentrated load at mid-span:

$$w_{\mathrm{m}} = \frac{5ML^2}{96EI_{h_s}}k_{3\delta b} \quad \text{with} \quad M = \frac{PL}{4} \tag{6.24}$$

$$w_{\mathrm{s}} = \frac{1.2M}{GA_{h_s}}k_{3\delta s} \tag{6.25}$$

Double-tapered beam with a uniformly distributed load:

$$w_{\mathrm{m}} = \frac{5ML^2}{48EI_{h_s}}k_{4\delta b} \quad \text{with} \quad M = \frac{qL^2}{8} \tag{6.26}$$

$$w_{\mathrm{s}} = \frac{1.2M}{GA_{h_s}}k_{4\delta s} \tag{6.27}$$

Curved beams
Calculation of deflections in curved beams is complex, but the following equations can be used to give an estimate of the movements:

$$w = \frac{w_{\mathrm{q}}}{\left(\cos\frac{\alpha+\beta}{2}\right)} \tag{6.28}$$

Here w_q is the deflection calculated for a symmetrical double-tapered beam with the same span and the same cross-section as the curved beam at supports and at mid-span, *see Chapter 7, page 102*.

The horizontal movement at the 'free' support can be estimated using the expression:

$$w_{\mathrm{h}} = 4\frac{(f+0.8h)}{L}w \tag{6.29}$$

where:

- f is the vertical distance between the neutral zone at support and the ridge.
- h is the beam depth at support.
- L is the span.
- w is the vertical deflection at mid-span.

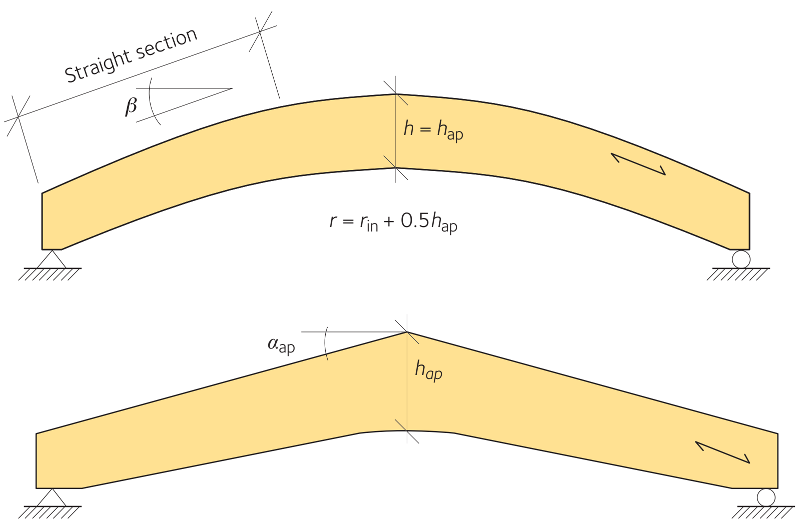


Figure 6.7 Curved beam (upper figure) and pitched cambered beam (lower figure)

Trusses

Deflection calculations are more complex for glulam trusses than single beams, since both the glulam members and the joints will exhibit deformations. 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:

6.30 $$I = \sum A_i \times a_i^2$$

where:

- A_i is the cross-sectional area of the external members.
- a_i is the distance between the system line of the external members and the centre of gravity of the truss.

Riding school, Gävle, Sweden.

Due to the movements that occur in the joints, the deformations in trusses are larger than in solid beams, and the increase in deflection depends on the number of joints in a truss as well as the type of joint. Measurements have shown that the deflection can increase by up to 10 – 15 percent compared with a case when the joints exhibit zero movements. This value is to be seen as an increase that has occurred after a number years. Tooth-plate connectors are the most rigid joints, while a joint with nails and plywood boards exhibits larger deformations.

It is therefore recommended that trusses are precambered (both the top and bottom chords), with approximately $L/150$ where $L/h = 12$ and $L/200$ where $L/h = 10$.

Three-hinged trusses

Three-hinged trusses are normally designed as a roof-supporting structure with rafters consisting of glulam and ties either of glulam or steel.

Under uniformly distributed downward load, as in *figure 6.8*, the vertical deflection of the ridge can be calculated using the expression:

6.31 $$w = \frac{(q_1 + q_2)\, L^2}{16(\tan\alpha)^2 (EA)_{\text{beam}}} \left(\frac{1}{(\cos\alpha)^3} + \frac{(EA)_{\text{beam}}}{(EA)_{\text{tie}}} \right)$$

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

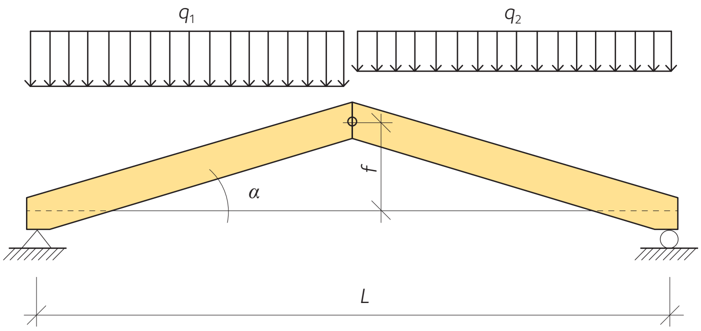


Figure 6.8 Three-hinged truss with simple roof beams and a tension member

6.3 Vibrations

Serviceability aspects include consideration of the comfort of the user when subjected to dynamic effects such as vibrations from people walking across a floor. This is a complex topic, since it relates to the mass of the floor structure and the actual arrangement of structural members in the floor, which provide lateral distribution of applied dynamic loads and damping of the generated vibration.

Users of buildings and other engineered structures sense low-frequency motions in three ways:

- Accelerations induce forces on the body that are felt by the balance organs.
- Visual cues (e.g. movement of objects resting on or hanging from the structure relative to the observer).
- Audio cues (e.g. creaking or rattling created by motion of the structure).

Human-induced vibrations in structures are almost always a problem of serviceability, in that they are a source of annoyance to the users. In some instances the person experiencing the motion is also the cause of it, while in other instances it is the activities of others that cause the annoyance. Thus, the activity of the person experiencing the vibration is important. When a person walks across a floor, he or she will tolerate much larger amplitude vibrations than when sitting quietly resting, reading or writing. Categorisation of human perception and tolerance needs to reflect both the activity being undertaken and the relationship between the source and the sensor. In this respect, the following definitions are often used:

- Springiness of a floor is associated with the sensation of self-generated floor deflection and vibration from a single footstep during the time of contact between foot and floor surface.
- Vibrational disturbances caused by footfall on a floor are characterised by perception of floor vibration induced by other persons than the one that is disturbed.

Springiness is usually a problem only when associated with lightweight floors or those that are flexible under concentrated load. Such floors are common in light-frame timber construction, and in other types of buildings with timber-joisted floors. In terms of the response of a floor system, springiness encompasses static flexibility and impulsive velocity response, while vibrational disturbance encompasses impulsive velocity response and stationary vibration response.

6.3.1 Dynamic loads

According to the various load sources and applicable countermeasures, structures affected by human-induced vibrations can be grouped as:

- Residential buildings.
- Office buildings.
- Industrial buildings.
- Pedestrian structures (footbridges, walkways in shopping malls).
- Gymnasia and sports halls.
- Dance and concert halls.

Askims torg, Gothenburg, Sweden.

There are very many variations of rhythmic body movements, leading to a large variety of dynamic loads. Activities generating synchronised rhythmic movements, such as those due to several or more people dancing or exercising, are especially problematic. Several people acting synchronously for 20 seconds or more can lead to approximately periodic loads that produce almost steady state structural vibration.

Forces from human motion depend upon many factors including the characteristics of the person(s) involved, the activity being undertaken (e.g. walking, running, jumping), the number of people, whether activities of different people are coordinated, and the characteristics of the floor surface. Annoying vibrations of glulam floors are commonly associated with walking and running forces.

Response to dynamic loads

From a structural standpoint, a glulam floor can be treated as a two-dimensional thin plate structure reinforced with a series of beams. Typically, this two-dimensional system is simplified as a one-dimensional beam structure for design under specified live and dead loads as in the previous sections. In many cases, the static stiffness properties of glulam floor systems are adequate to ensure satisfactory vibration performance. In some instances, however, floor systems – designed to meet traditional deflection criteria under uniformly distributed loads – have been found to exhibit vibration problems.

New construction practices have had a profound impact on the vibration characteristics of some glulam floors. Amongst these is the use of prefabricated engineered wood joists, concrete toppings and floating floors. The availability of engineered wood joists and trusses has led to more long-span and continuous multi-span floor systems, while the use of a concrete topping has dramatically altered the mass characteristics of glulam floors.

There are number of different design methods with regard to vibration problems and a number of studies have been made to improve the methods.

Malmö Central Railway Station, Sweden.

Factors affecting human response to floor vibrations

When considering human response to transient vibration in a floor system parameters such as frequency components, magnitude of response and damping of the vibration are the most important factors.

Frequency components

With respect to vibrations, floors are usually divided into low-frequency floors and high-frequency floors. Low frequency floors have a fundamental frequency below 7 – 8 Hz, and high-frequency floors have a fundamental frequency above 7 – 8 Hz. Low-frequency floors are generally heavy structures such as concrete floors. This classification of floors into low-frequency and high-frequency ones has its origin in the different responses of the floor types to human walking. For low-frequency floors the low frequency parts of human walking (the continuous parts) are the most important, because they cause a resonant response in the floor. This means that a person staying still may feel this resonance vibration. A high-frequency floor is more responsive to the impulsive parts of human walking. In this case a person standing still might feel the impacts caused by another person walking by, and the walking person might get a feeling of springiness.

The fundamental natural frequency f_1 of a two-way structural system, such as a glulam floor, is governed primarily by the system stiffness in the along-joist direction EI_x, the unit mass m and the span L. The spacing of two adjacent natural frequencies is controlled by the ratio between the across-joist direction stiffness EI_y and EI_x.

Damping

Damping is a property influencing vibration amplitudes under forced vibration and the rate of decay of vibration amplitudes under free vibration. Increased damping results in the rapid decay of a free vibration. The material damping depends on the materials used for construction and is usually small. The major contributor to damping appears to be due to friction: the structural detailing, such as the manner in which components are attached, and the boundary conditions at supports, contribute to the frictional damping.

End conditions of bending members can greatly affect damping. It has been shown that, for glulam beams with simply supported ends and console beams, the damping ratio associated with the fundamental mode is about 1 percent. When the same members had 'clamped' ends the damping ratio increased to about 8 percent. For timber-joisted floors, effective damping ratios are in the range of 1 to 3 percent depending upon the details of the floor and the mode being considered. Application of imposed masses (objects) on the surface of a floor can greatly increase the damping, especially if the system is lightweight or small.

Because of inherent low self-weight, glulam floors often do not exhibit a significant amount of inertial damping. Exceptions can occur with large systems, especially if they have a thick concrete topping or support large amounts of imposed mass.

Bridge, Gislaved, Sweden.

6.3.2 Design approaches to limit vibrations in glulam floors

The fundamental research in understanding the factors affecting human response to floor vibrations has paved the way for development of design approaches to prevent vibrations. Two examples are presented here.

Limiting 'point load' deflection

It has been shown that one possible method of predicting human response to floor vibration is to determine the static deflection under a concentrated load. A common method of doing this is to determine the static deflection under a 1 kN 'point load' at the centre of a single beam in the simplest model or under a two-way floor system. The deflection limits used in these rather simple models differ depending on regulation or handbook, but also on span length and aim with the design.

This approach can be seen as a method for modeling the effect of a step action. The static load simulating the foot force effect is 1 kN applied at the centre of the floor and the deflection of the floor at this point, a, must be no greater than a certain limit value. The deflection of a point load is given by:

$$a = \frac{PL^3}{48EI} \qquad 6.32$$

where in this case P = 1 kN. If the simplest approach is used, *equation 6.32* gives the value of the deflection with the flexural rigidity EI

Six-storey residential building with glulam frame.

for one single joist. In most cases, however, this will overestimate the deflection, since the joist is only one element in a two-way load-bearing system. In order to take this into account, the rigidity in both directions of the floor can be used to estimate the magnitude of the deflection.

6.33
$$a = \kappa \frac{PL^3}{48EI}$$

where κ is a load distribution factor that can be calculated according to the following expressions:

6.34
$$\kappa = \begin{cases} -4.7\beta^2 + 2.9\beta + 0.4 & \text{when} \quad 0 \leq \beta < 0.3 \\ 0.8 + 0.2\beta & \text{when} \quad 0.3 \leq \beta \leq 1.0 \end{cases}$$

with

6.35
$$\beta = \frac{(EI)_{\mathrm{L}}}{(EI)_{\mathrm{B}}} \left(\frac{s}{L}\right)^4$$

where $(EI)_L$ is the flexural rigidity of the floor in the stiffer direction, i.e. along-joist (Nm^2/m) and $(EI)_b$ is the flexural rigidity of the floor in the direction perpendicular to the stiffer direction, i.e. across-joist (Nm^2/m), s is the spacing between the joists and L is the span of the joists.

Limiting 'point-load' deflection and peak velocity due to unit impulse

The use of static response parameters, such as deflection, provides some control, but does not always produce satisfactory performance. Researchers are aware of this limitation, and recent research has focused more on studying dynamic parameters. Among the first to propose dynamic-based parameters for design was Sven Ohlsson (1991) and in order to account adequately for the important factors that affect human response to floor vibration, two parameters should be checked for light-weight floors having natural frequencies above 8 Hz:

1. Static deflection limit under 1 kN load at floor centre (1.5 mm)
2. Peak velocity v due to a "unit impulse of 1 Ns" $< 100^{[f_1\zeta - 1]}$ $m/(Ns^2)$, where f_1 is the fundamental natural frequency and ζ is the damping ratio for f_1.

The first criterion is similar to the one previously presented and Sven Ohlsson (1991) stated that this is a control of the low-frequency components (< 8 Hz) that are semi-static in nature. The second criterion is required to limit the magnitude of the transient response due to the heel impact of a footstep. The peak velocity, due to a unit impulse, for a rectangular floor system simply supported on all four sides, valid for $f < 40$ Hz, is then calculated as:

6.36
$$v = \frac{4\left(0.4 + 0.6n_{40}\right)}{mBL + 200}$$

where n_{40} represents the number of eigenmodes with eigenfrequencies lower then 40 Hz and is given by:

$$n_{40} = \left[\left(\left(\frac{40}{f_1}\right)^2 - 1\right)\left(\frac{B}{L}\right)^4\left(\frac{(EI)_L}{(EI)_B}\right)\right]^{0.25} \quad 6.37$$

where B is the width of floor (m), L the span (m); m the mass per unit area (kg/m^2), $(EI)_L$ is the flexural rigidity of floor in the stiffer direction, i.e. along-joist (Nm^2/m), and $(EI)_B$ is the flexural rigidity of the floor in the direction perpendicular to the stiffer direction, i.e. across-joist (Nm^2/m).

Sven Ohlsson also provides an equation for calculating the fundamental natural frequency of a floor:

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_L}{m}} \quad 6.38$$

Since the introduction of this method, it has been used quite extensively and has in many cases shown satisfying results, i.e. floors designed according to this method have shown satisfactory behaviour.

In the second criterion above the damping ratio ζ has to be determined, which is a difficult matter. Ohlsson states that the value for ζ can be 1 percent, but also mentions that higher values can be relevant.

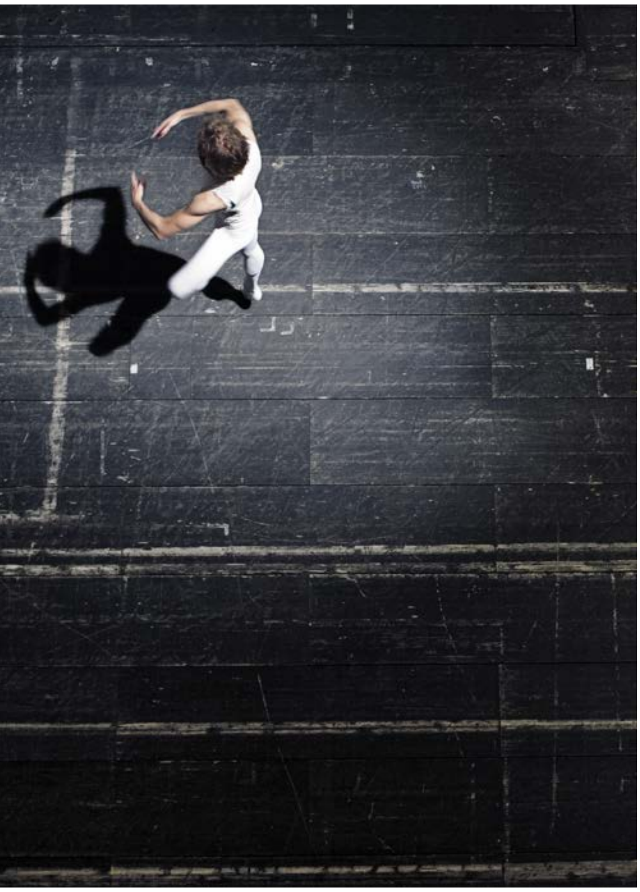
The methods described previously work well in some cases, but are less adequate in others. The design methods usually give only a limit value for floors, and it is often unclear for the designer what the limit value actually means. How much better is the floor if the limit value is lowered by 50 percent? A number of approaches to model and predict the dynamic response can be found in the literature, but for all of them it can be said that there are uncertainties and that it often is difficult to find a single model that can cope with all situations. All the design approaches are semi-empirical in nature and provide satisfactory solutions for the particular category of floors on which they are based. None appears to work entirely satisfactorily when applied to other categories of floors. In many cases it can be said that the best knowledge of the behaviour of a floor is still acquired by testing it.

6.3.3 Avoidance of vibration problems

Since the problems with vibrations are complex in nature, it is also difficult to give one recommendation that solves the problems. One practical strategy is to try to design structural systems that have relatively high natural frequencies, a method referred to as tuning the frequency. This method means that the lowest and most energetic structural frequencies of the structures are higher than the excitation frequency. The objective is to avoid coincidence between loading and response frequencies.

Addition of extra material (increasing the stiffness), or reduction of the span, are methods used. On the other hand, there has been a trend to decrease the amount of material and increase span lengths due to other aims in the design process. Therefore it is not surprising that reports of vibration problems in structures have increased recently. It is normally easier to increase the strength-to-weight ratio than to increase the stiffness-to-mass ratio of structural materials.

Grandstand, Sollentuna, Sweden.



The Opera house, Stockholm, Sweden.

Other means of avoiding 'problem floors' are obviously available. A potentially effective approach is to increase the damping (e.g. to add artificial damping or install tuned mass dampers), but this is normally a complex and/or expensive solution. Attention to details such as provision of adequate blocking or cross-bracing and to be observant of detailing and connections between different parts of the system are also important.

Floating floors and raised floors are increasingly used because of impact sound insulation requirements and because of the flexibility of mounting the installations. The vibration or movements of objects, such as the clinking of glassware or leaf movement of plants, are typical for these types of floor, and it has been shown that such effects are highly dependent on the flexural stiffness of the top surface board. Local deflections are caused by soft floating floors and need to be limited to avoid such disturbances. On the positive side: floating floors with sufficient bending stiffness of the top layer may effectively distribute concentrated loads to various floor joists and thus improve the floor vibration performance.

As local deflections are usually difficult to predict using engineering calculations, it is recommended that they are determined based on tests. The bending stiffness of the floating floor top layer has a major impact on the local deflection.

It is also important to be aware of the transmission of vibrations between different rooms via the floor joists. With continuous beams, vibration can be transferred from one apartment (or room) to another and, even if no problems are experienced in the room from where the vibration emanates, neighbours can experience nuisance from them. And it has been shown that vibrations coming from a neighbouring apartment are more irritating than if the vibration source is in the same apartment.

6.4 Moisture movements

Glulam components are normally supplied at a moisture content of 12 %. Under different climatic conditions the moisture content will in time adjust itself to the surrounding relative vapour pressure and to the temperature.

As a result of seasonal changes in the climate, the moisture content in a structure will vary continuously. 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 materials, swells when the moisture content increases and shrinks when it decreases. Movement is many times larger perpendicular to the grain than parallel, 0.2 percent and 0.01 percent respectively for each percent of the change in moisture content. This means that, for normal humidity variations, the change along the fibre (that is normally along the beam) is approximately 0.1 mm/m in an indoor climate and 0.2 mm/m in an outdoor climate. Across the fibre direction the changes will be 2 mm/m in an indoor climate and 4 mm/m in an outdoor climate.

Conference centre, Ekerö, Sweden.

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 load-carrying capacity in bolted connections can be 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:

$$w = \frac{L \times \Delta L}{8h} \tag{6.39}$$

where:

L	is the span.
ΔL	is the difference in length between outer face and inner face due to swelling or shrinking.
h	is the depth of the member.

It is also important to be aware of the effects of moisture movements in joints and when ties are used to transfer loads, since the movements can lead to decreasing capability to transfer forces between the different structural elements. In some cases it is necessary to check the joints or ties and tighten the joints during the lifetime of the structure.

Tapered, curved and pitched cambered beams

Glued laminated beams are often tapered and/or curved in order to meet architectural requirements, to provide pitched roofs, to obtain maximum interior clearance and to reduce wall height requirements at the end supports. The most commonly used types are the single tapered beam, the curved beam with constant cross-section, the double tapered beam and the pitched cambered beam, *see figure 7.1*.

A peculiarity of these special timber elements is that the distribution of bending stresses is non-linear and therefore ought to be calculated using the theory of anisotropic plates. However, for design purposes the maximum bending stress and shear stresses can be calculated with satisfactory approximation according to simple beam theory for isotropic materials.

7.1 Tapered beams

Structural timber elements, especially glulam components, are often designed with a varying cross-sectional depth, e.g. single and double pitched beams, continuous beams with a deeper section over the intermediate supports, etc. As a rule, variations in the cross-sectional depth are achieved by tapering the lamellas along one edge. The design approach for single and double tapered beams is similar. Therefore, only double tapered beams will be treated in this section. The material economy of double tapered beams is good, since the depth follows the trend of the bending moment diagram.

The distribution of bending stresses in tapered beams is non-linear. Moreover, for such beams the highest shear stress is in general not at the neutral axis. The location of maximum shear stress is, in fact, closer to the tapered edge, *see figure 7.2, page 103*. Only at the support of a simply supported beam or at the free end of a cantilever beam the shear stress is highest at the neutral axis.

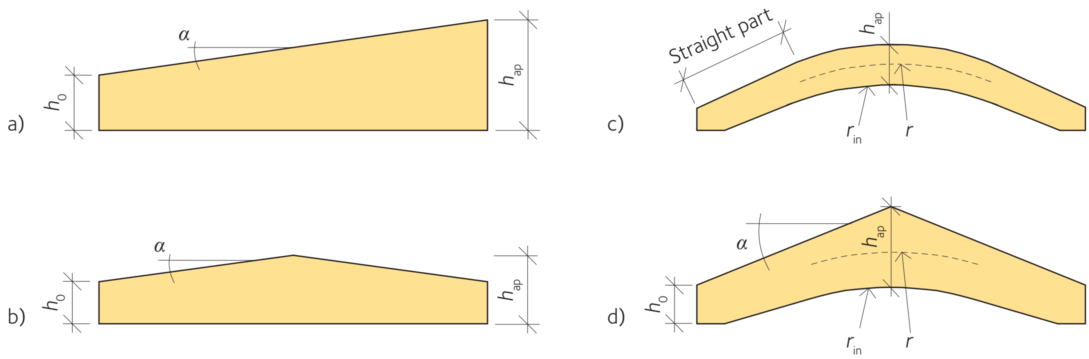


Figure 7.1 Special glulam beams. a) Single tapered, b) double tapered, c) curved, d) Pitched cambered.

Note that the shear and bending stresses shown in *figure 7.2* are in the direction parallel to the lamellas, not parallel to the tapered side.

Since both the moment and the depth vary along the axis of the beam, the maximum bending stress tends not to occur where the moment is greatest, but at a section nearer the supports, *see figure 7.2*.

For small slopes of the tapered edge, say $\alpha \leq 10°$, which covers most practical cases, the effect of tapering on the bending stress distribution is small. For design purposes, therefore, the maximum bending stress can be calculated as for a beam with constant cross-sectional depth (i.e. $\sigma_m = M/W$), both at the tapered edge and at the straight edge. However, the bending strength value f_m shall be reduced at the tapered edge to take into account the effect of the shear stress and the stress perpendicular to the grain, which are acting simultaneously with bending stress at the tapered edge, *see figure 7.3*. Eurocode 5 recommends reducing the bending strength of the tapered edge by a factor $k_{m,\alpha}$, *see section 7.3.1, page 106*.

It is possible to show that the magnitude of the perpendicular to grain stresses, σ_{90}, increases with the increasing slope of the tapered edge. If the tapered edge is located at the compression side of the beam, compression perpendicular to grain, $\sigma_{c,90}$, occurs during bending of the beam. On the other hand, if the tapered edge is located at the tension side of the beam (upside down tapered beams), *see figure 7.4*, tension perpendicular to grain, $\sigma_{t,90}$, occurs.

Pellet store, Tunadal, Sweden.

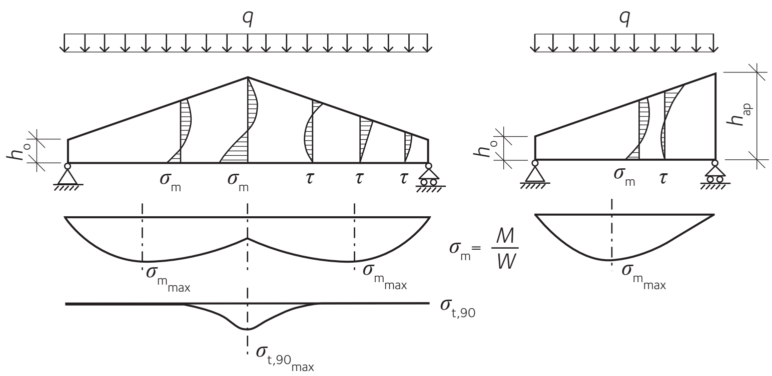


Figure 7.2 **Bending stresses (σ_m), shear stresses (τ) and perpendicular to the grain tensile stresses ($\sigma_{t,90}$) in double tapered beam (left) and single tapered beam (right)**

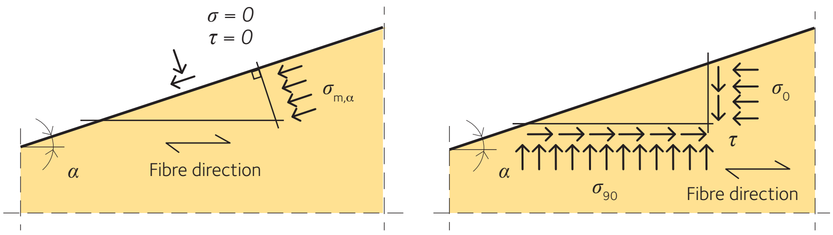


Figure 7.3 **Stresses at the tapered edge of a beam**
Left: bending stress parallel to the tapered edge $\sigma_{m,\alpha}$ (principal stress).
Right: bending stress σ_0 parallel to the grain.

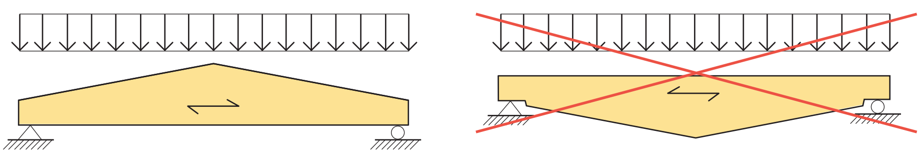


Figure 7.4 **Left: compression perpendicular to the grain occurs at the tapered edge.**
Right: tension perpendicular to the grain occurs at the tapered edge (upside down tapered beam. **Not recommended!**).

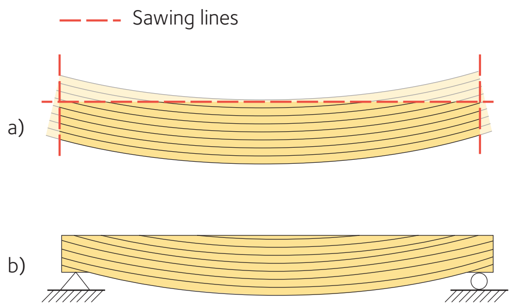


Figure 7.5 a) Gluing of the lamellas, b) a finished beam with a curved lower edge, known as a "fish belly" beam

Due to these reasons, the slope of the tapered edge should be limited. The angle of inclination α should not exceed 10° on the compression side and 2 – 3° on the tension side (it should be remembered that tension perpendicular to grain can cause brittle failure at a very low stress level and should therefore be kept as low as possible).

If the top side of a roof needs to be straight (i.e. to have a single slope), an alternative to using upside down tapered beams is the use of "fish belly" shaped beams, where the continuous lamellas are at the tension edge and tapered lamellas at the compression edge, *see figure 7.5*. In this case, dangerous tension stresses perpendicular to the grain are eliminated.

7.2 Curved and pitched cambered beams

Amongst the most prominent advantages of glulam is the possibility of designing curved structural members. During manufacture, the individual lamellas are bent to the desired form before the glue has hardened. From the point of view of structural behaviour, it is important to make a clear distinction between arches and curved beams. The fundamental difference between these two types of structures is that, in arches, both supports are impeded to translate (meaning that a horizontal thrust and thus a normal force can be developed), whereas in curved beams one of the supports is free to move along the longitudinal direction of the beam. Arch structures will be dealt with in *Chapter 11, page 153*.

To avoid damaging the lamellas as they are bent, the curvature must be limited. For a given radius of curvature, the bending stress in a lamination increases as its thickness increases. Thicker laminations cannot be bent as sharply as thinner ones. Curvature must also be limited so that high residual bending stresses are not present in the finished member. These initial stresses can often be ignored in design. However, when the ratio of radius of curvature, r_{in}, to lamination thickness, t, is too small, the bending strength of the beam is affected by residual bending stresses and should be reduced. According to Eurocode 5, such a reduction factor must be applied when $r_{in}/t < 240$. In practice, however, the ratio between the radius of curvature and the lamination thickness should never be lower than approximately 170.

When a bending moment is applied to a beam that is initially curved in the plane of bending, radial stresses occur as well as bending stresses. These radial stresses may be either tensile or compressive, *see figure 7.6*. When the applied moment tends to increase the curvature of the glulam member, the lamellas are pressed more firmly together, *see figure 7.6, upper*. This means that compressive radial stresses occur between lamellas. On the other hand, when the applied bending moment tends to straighten the glulam member, the lamellas try to move apart, *see figure 7.6, lower*. This means that tensile radial stresses occur between lamellas. Tensile radial stresses should be kept as low as possible since they may cause cracking of the element.

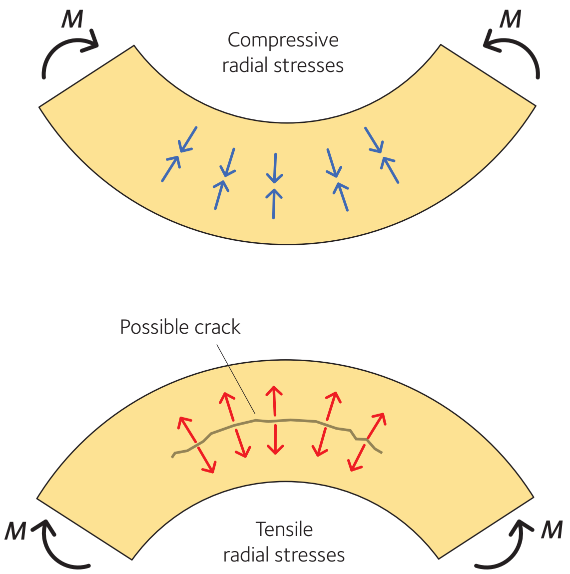


Figure 7.6 Upper: bending moment tending to increase the curvature of the glulam member. Lower: bending moment tending to straighten the glulam member

Figure 7.7 shows the apex zone of a curved beam subjected to constant bending moment. Assuming, for simplification, a linear bending stress distribution at the apex zone, it is evident that the resulting tensile and compressive forces, T and C respectively, give rise to a force T_{90} in the radial direction.

It can be shown that the perpendicular to grain tensile stress, generated by T_{90}, *see figure 7.7*, is:

$$\sigma_{t,90} = \frac{T_{90}}{b \times dl} = \frac{h}{4 \times r} \times \sigma_m = k_p \times \sigma_m \qquad 7.1$$

Equation 7.1 shows that the tension stress perpendicular to the grain $\sigma_{t,90}$ at the apex of a curved beam can be calculated approximately by modifying the bending stress parallel to the grain ($\sigma_m = M/W$) with a shape factor $k_p = h/(4r)$.

Note that the radius of curvature, r, should be kept reasonably large to reduce the magnitude of the stress perpendicular to the grain. In general, it is best to choose $r \geq 8$ m.

Various studies have shown that the tension strength perpendicular to the grain is highly dependent on the stressed volume of the timber. The basic design value of the tensile strength perpendicular to the grain must therefore be modified. According to Eurocode 5, the basic value of the tension strength perpendicular to the grain $f_{t,90}$ shall be multiplied by a modification factor k_{vol} (< 1).

In situations where the design tensile stress, $\sigma_{t,90}$, exceeds the value in EC5 for the tensile strength for stresses perpendicular to the grain, mechanical fasteners such as glued-in rods or full-threaded screws may be used as reinforcement, *see section 7.4, page 115*.

Production of glulam.

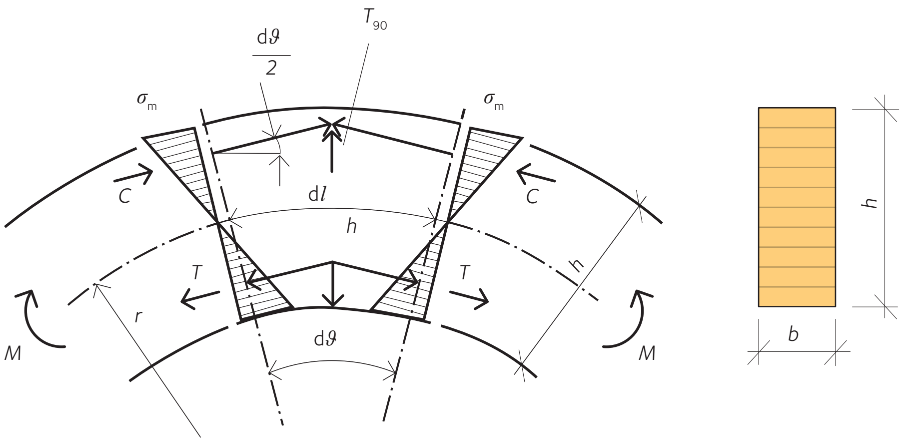
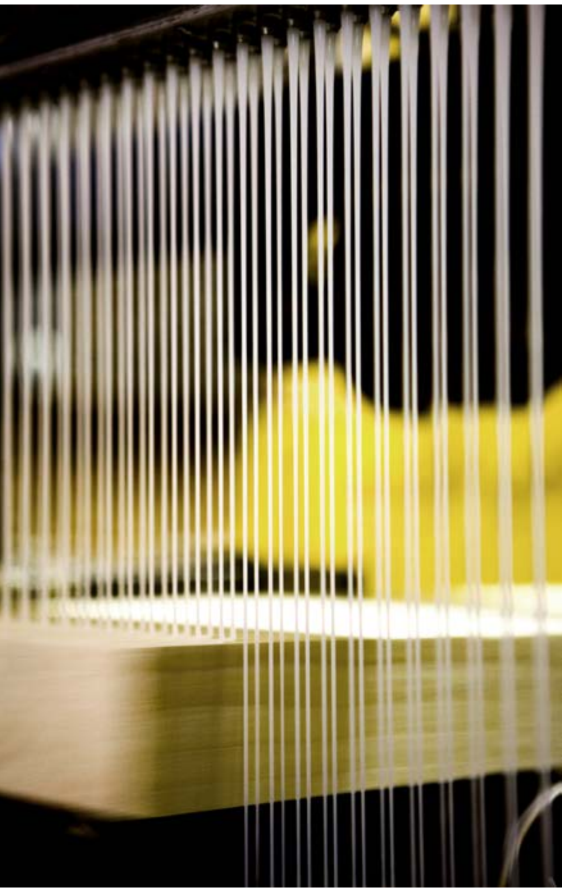


Figure 7.7 Simplified model for determining stresses perpendicular to grain at the apex of a curved beam subjected to pure positive bending moment



Glue application during production.

7.3 Design procedures

As a general rule, it is recommended that during manufacturing of both tapered and pitched cambered beams the continuous laminations are always placed at the tension side of the beam, i.e. at its bottom side if the beam is simply supported and subjected to predominantly gravity loads. As a consequence, the tapered laminations will be located at the compression side of the beam.

7.3.1 Tapered beams

Normally, the maximum span for such beams is limited to 30 m, mainly due to manufacturing limits and transport costs. The slope should never exceed 10°. According to Swedish practice such a slope is within 1/20 – 1/10, i.e. $\alpha \approx 2.9 - 5.7°$. The depth at the apex should not be less than about $l/20$, where l is the beam span. The breadth should not be less than 1/7 of the depth of the beam at the quarter point of the span, in order to reduce lateral-torsional buckling problems, especially during the erection stage.

Very often, tapered beams are manufactured with a pre-camber, the magnitude of which should be approximately that of the maximum deflection due to the permanent loads plus one half of the main variable load, e.g. snow load.

For symmetrical, simply supported beams with a uniformly distributed load, the bending strength of the apex (i.e. at mid-span) will never be critical. The cross-section with highest bending stress is found at a distance x_0 from the support. The position of this section can be determined analytically as the one where the derivative of the bending stress with respect to x is zero. This leads to the following result:

7.2 $$x_0 = \frac{h_0}{2 \times h_{ap}} \times l$$

where h_0 is the depth of the beam at the support and h_{ap} is the beam depth at the apex and l is the span. For common beam geometries, the location of the maximum bending stress is approximately located at the one-quarter point of the span ($x_0 \approx l/4$).

For slopes $\alpha \le 10°$, the design bending stresses $\sigma_{m,\alpha,d}$ and $\sigma_{m,0,d}$ may be assumed to be equal and determined in accordance with classic beam theory. **Note** that in *figure 7.8* the real stress distribution is shown, with $\sigma_{m,\alpha,d} \neq \sigma_{m,0,d}$. Our simple assumption gives:

7.3 $$\sigma_{m,\alpha,d} = \sigma_{m,0,d} = \frac{M_d}{W_{x0}}$$

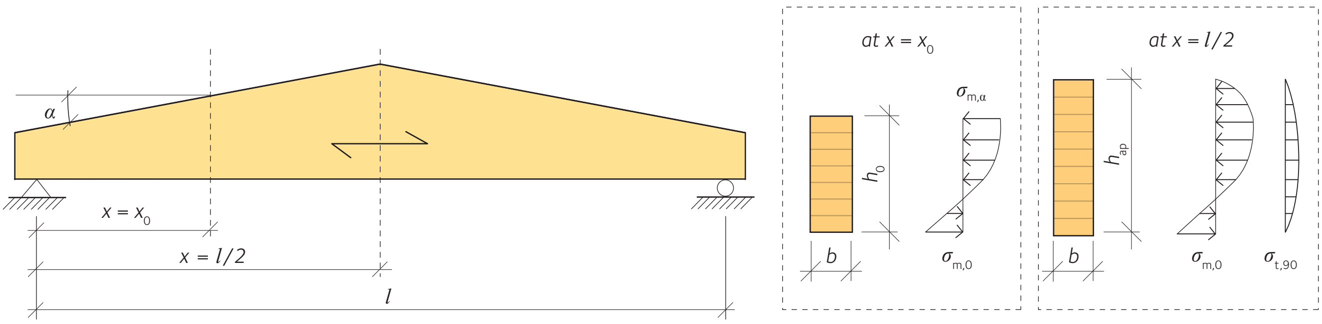


Figure 7.8 Single tapered beam showing the real, nonlinear distribution of bending stress and perpendicular to grain stress at $x = x_0$ and at mid-span

Pellet store, Tunadal, Sweden.

where:

M_d is the design bending moment in the section $x = x_0$.

W_{x_0} is the modulus of resistance at the cross-section $x = x_0$.

Very often, combined glulam is used for the manufacture of tapered beams, for example glulam class GL30c, which normally consists of lamellas T22 and T15 in the outer and the central part of the cross-section respectively. For this type of tapered beam, the largest compressive stress due to bending moment, at a distance x_0 from the support, generally occurs at lamellas with lower strength class, i.e T15. In theory, this would imply a reduction in strength of the beam, if compared to the strength of a similar beam with homogeneous cross-section and with lamellas T22. However, this reduction in strength is normally neglected, due to the fact that failure nearly always occurs at the tension side of the beam, where high strength lamellas, i.e. T22, are located.

At the outermost fibre of the tapered edge, the stresses should satisfy the following expression:

$$\sigma_{m,\alpha,d} \leq k_{m,\alpha} \times f_{m,d} \qquad 7.4$$

where:

$\sigma_{m,\alpha,d}$ and $\sigma_{m,0,d}$ are the design bending stresses at an angle to grain and at the straight edge, respectively.

$f_{m,d}$ is the design bending strength.

$k_{m,\alpha}$ is a reduction factor that takes into account the simultaneous action of bending stress, shear stress and compression/tension stress at the tapered edge.

According to Eurocode 5 the reduction factor $k_{m,\alpha}$ can be calculated in the following way:

a) For tensile stresses parallel to the edge:

$$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left(\frac{f_{m,d}}{0.75 f_{v,d}} \tan\alpha\right)^2 + \left(\frac{f_{m,d}}{f_{t,90,d}} \tan^2\alpha\right)^2}} \qquad 7.5$$

b) For compressive stresses parallel to the edge:

$$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left(\frac{f_{m,d}}{1.5 f_{v,d}} \tan\alpha\right)^2 + \left(\frac{f_{m,d}}{f_{c,90,d}} \tan^2\alpha\right)^2}} \qquad 7.6$$

where:

$f_{v,d}$, $f_{t,90,d}$ and $f_{c,90,d}$ are the design EC5 strengths for shear, tension perpendicular to grain and compression perpendicular to grain, respectively.

The values of $k_{m,\alpha}$ for different slopes of the tapered edge are shown in *figure 7.9*. The values were derived for the glulam class GL30c.

Note that the continuous curve can also be used — with satisfactory accuracy — for the design of fish belly beams.

Production of glulam.

In such a case the angle α at the location of the maximum bending stress can be derived in accordance with *equation 7.7*:

7.7 $$\alpha \approx \frac{l}{4 \times R} \times \frac{180}{\pi}$$

where l is the beam span and R is the radius of curvature of the laminations (usually R > 100 m).

The design tensile stress perpendicular to the grain due to bending moment can be calculated as follows:

7.8 $$\sigma_{t,90,d} = (0.2 \times \tan\alpha) \times \frac{M_{ap,d}}{W_{ap}}$$

where:

$M_{ap,d}$	is the design moment at the apex.
W_{ap}	is modulus of resistance of the beam at the apex.
$(0.2 \times \tan\alpha)$	is a factor determined by finite element analysis, defined as the ratio between the perpendicular to grain stress and the bending stress at the apex zone. The variation of such a factor – denoted as k_p in Eurocode 5 – with the slope α is also shown in *figure 7.12, page 112*; for tapered beams the radius of curvature $\rightarrow \infty$, therefore the k_p values in *figure 7.12, page 112*, should be taken at $h_{ap}/r = 0$.

The tensile strength perpendicular to the grain must then be reduced to take the volume effect into account. According to Eurocode 5, the following inequality shall be satisfied:

7.9 $$\sigma_{t,90,d} \leq k_{dis} \times k_{vol} \times f_{t,90,d} = k_{dis} \times \left(\frac{0.01}{V}\right)^{0.2} \times f_{t,90,d}$$

where $f_{t,90,d}$ is the design tensile strength perpendicular to the grain; k_{dis} is a factor that takes into account the effect of the stress distribution in the apex zone. Values of k_{dis} and V for beams loaded by uniformly distributed load can be taken from *table 7.1, page 109*.

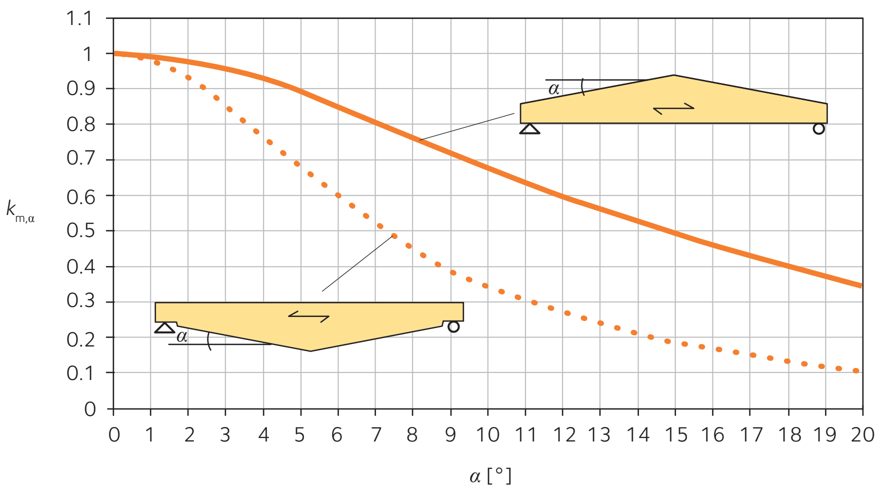
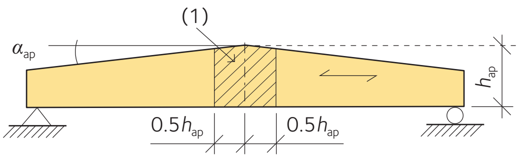
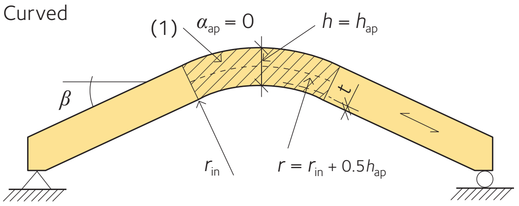
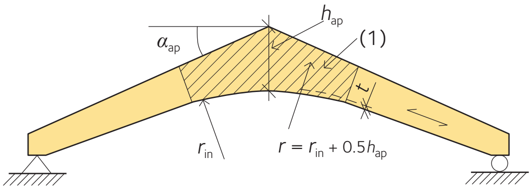


Figure 7.9 Values of $k_{m,\alpha}$ according to Eurocode 5 for different slopes of the tapered edge; Glulam class GL30c, service class 1, load duration: medium term

Table 7.1 Values of k_{dis} and V according to Eurocode 5 for typical beam types

Beam type	k_{dis}	V
Double tapered 	1.4	Volume of (1), see figure on the left $\sim b \times \left(h_{ap}\right)^2$
Curved 	1.4	Volume of the curved part (1)* $\frac{\beta\pi}{180} b \left(h^2{}_{ap} + 2h_{ap}r_{in}\right)$
Pitched cambered 	1.7	Volume of the curved part (1)* $b\left(\sin\left(\alpha_{ap}\right)\cos\left(\alpha_{ap}\right)\left(r_{in} + h_{ap}\right)^2 - r^2{}_{in}\frac{\alpha_{ap}\pi}{180}\right)$

* V needs, however, not be taken as more than $2/3V_b$, where V_b is the total volume of the beam.
Angles α and β in degrees. "b" denotes the breadth of the beam.

In normal cases, the final design of tapered beams will include the following checks:

- Bending strength at a distance x_0 from the support.
- Shear and bearing strength at the support perpendicular to the grain tensile strength at the apex.
- Perpendicular to the grain tensile strength at the apex.
- Lateral-torsional buckling (normally this check is performed for an isolated part of the beam between two adjacent purlins).

7.3.2 Preliminary design of simply supported double tapered beams subjected to uniformly distributed load

General indications concerning the preliminary design of a simply supported double tapered beam will be given in this section. A typical structure of such a type, with indication of the principal geometric parameters is shown in *figure 7.10*.

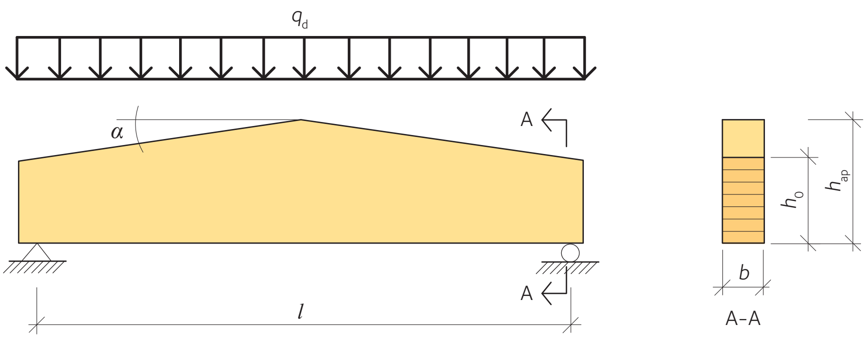


Figure 7.10 Typical double tapered glulam beam

Glulam pergola.

During the preliminary dimensioning stage, the following variables are generally known:

- Design load intensity q_d.
- Roof slope α (normally within the range 2 – 6°).
- Span l (normally ≤ 30 m).

Furthermore, all the material design strengths are known, since they depend on the choice of glulam strength class, e.g. GL30c.
The preliminary design will consist of estimating the cross-sectional dimensions both at the support and at the apex.

Beam breadth
In order to reduce problems with lateral instability, especially during the erection stage, the breadth of the beam should not be less than 1/7 of the beam depth, which in turn is approximately 1/20 – 1/15 of the beam span. This leads to:

7.10
$$b \approx \frac{l}{140} \cdots \frac{l}{110}$$

Beam depth
The strength of the beam is normally governed by its bending strength at the cross-section located at a distance x_0 from the support, *see section 7.1, page 106*. However, the derivation of the parameter x_0 is based on the knowledge of the beam depth both at the support and at the apex, which are not known a priori. During preliminary design it is assumed therefore, that the location of x_0 is at $l/4$ from the support. Moreover, it is assumed that the bending strength of the beam, $f_{m,d}$, at x_0 is somewhat reduced due to the tapering of the upper edge. A reasonable reduction factor for taper with $\alpha < 6°$ is $k_{m,\alpha} \approx 0.9$, *see figure 7.9*. With these assumptions, the required beam depth at the support and at the apex can be estimated by means of the following equations:

- Beam depth at the support:

7.11
$$h_0 = \frac{l}{4} \times \left(3 \times \sqrt{\frac{q_d}{b \times (0.9 \times f_{m,d})}} - \tan\alpha \right)$$

- Beam depth at the apex:

7.12
$$h_{ap} = \frac{l}{4} \times \left(3 \times \sqrt{\frac{q_d}{b \times (0.9 \times f_{m,d})}} + \tan\alpha \right)$$

EKS 10 states that unsymmetrical snow loads should be taken into account on pitched roofs. With shallow roof pitches, the difference between the leeward side and the windward side is small and a safe approximation is to calculate the leeward load evenly distributed over the entire beam. This can be used to determine initial design values. More accurate calculation methods can then be used in the final design.

7.3.3 Curved and pitched cambered beams

The maximum span for both curved and pitched cambered beams should not exceed 20 m, mainly due to the limitation imposed by the tensile stresses perpendicular to the grain occurring at the apex zone. Also, in order to reduce the risk of failure due to tension perpendicular to the grain, the slope of these beams, α, should be below 15° (for curved beams this slope is that of the straight parts).

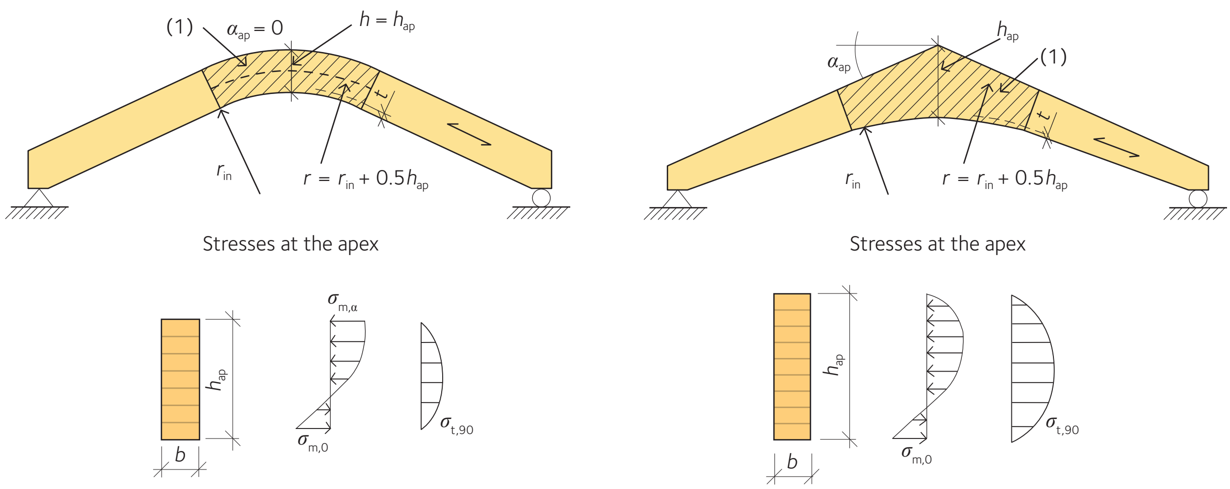


Figure 7.11 Bending stresses $\sigma_{m,0}$ and tension stresses perpendicular to the grain $\sigma_{t,90}$ for: curved beam (left) and pitched cambered beam (right)

The depth at the support should not be less then $l/30$ for both beam types. The depth at the apex is normally between $l/20$ and $l/15$ for curved beams and between $l/15$ and $l/10$ for pitched cambered beams. The breadth should not be less than 1/7 of the depth of the beam at the quarter point of the span. The radius of curvature r, *see figure 7.11*, is normally greater than 10 m.

The vertical deflections of these beams are normally of no importance. The horizontal displacements at the supports may be relatively large, however; therefore, it is necessary to ensure that these displacements can occur without giving rise to unforeseen horizontal forces on the supporting structures (normally, walls or columns).

The cross-section with higher bending stress is located at a distance x_0 from the support. The position of this section can be determined using *equation 7.2, page 106*. The design of curved and pitched cambered beams, however, is normally governed by tensile stresses perpendicular to the grain, which are mainly caused by gravity loads, but also by moisture variations in the wood. These stresses are greatest at the apex area of the beam, this zone is denoted as (1) in *figure 7.11*.

According to Eurocode 5, the design tensile stress perpendicular to the grain due to the design bending moment at the apex, $M_{ap,d}$, can be calculated as follows, see symbols in *figure 7.11*:

$$\sigma_{t,90,d} = k_p \times \frac{M_{ap,d}}{W_{ap}} \quad \text{7.13}$$

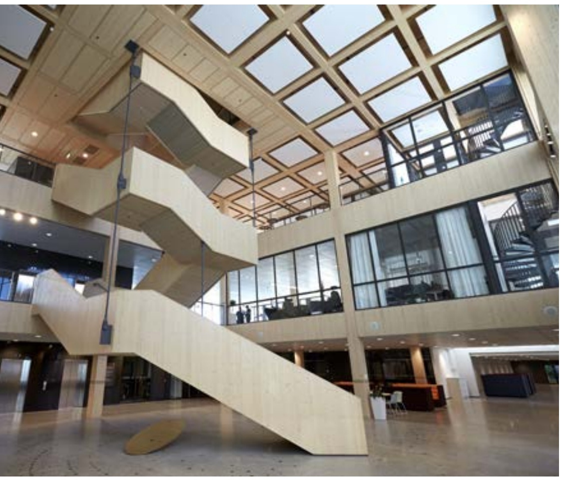
where:

$$k_p = k_5 + k_6 \times \left(\frac{h_{ap}}{r}\right) + k_7 \times \left(\frac{h_{ap}}{r}\right)^2 \quad \text{7.14}$$

$$k_5 = 0.2 \times \tan \alpha_{ap} \quad \text{7.15}$$

$$k_6 = 0.25 - 1.5 \times \tan \alpha_{ap} + 2.6 \times \tan^2 \alpha_{ap} \quad \text{7.16}$$

$$k_7 = 2.1 \times \tan \alpha_{ap} - 4 \times \tan^2 \alpha_{ap} \quad \text{7.17}$$



Ulls hus, Uppsala, Sweden.

k_p is a factor determined by finite element analysis, defined as the ratio between the design stress perpendicular to grain and the design bending stress at the apex. *Figure 7.12* shows the variation of k_p as a function of h_{ap}/r. Separate curves are plotted for different apex angles, α_{ap}. The material is glulam GL30c.

The check for perpendicular to grain strength should be performed in the same manner as for tapered beams, *see equation 7.9*.

The bending strength at the apex can sometimes be the governing parameter for curved beams with large radius of curvature; on the other hand it is very seldom critical for pitched cambered beams. According to Eurocode 5, the bending stress due to the design bending moment at the apex, $M_{ap,d}$, can be calculated as follows:

7.18 $$\sigma_{m,d} = k_l \times \frac{M_{ap,d}}{W_{ap}}$$

with:

7.19 $$k_l = k_1 + k_2 \times \left(\frac{h_{ap}}{r}\right) + k_3 \times \left(\frac{h_{ap}}{r}\right)^2 + k_4 \times \left(\frac{h_{ap}}{r}\right)^3$$

7.20 $$k_1 = 1 + 1.4 \times \tan\alpha_{ap} + 5.4 \times \tan^2\alpha_{ap}$$

7.21 $$k_2 = 0.35 - 8 \times \tan\alpha_{ap}$$

7.22 $$k_3 = 0.6 + 8.3 \times \tan\alpha_{ap} - 7.8 \times \tan^2\alpha_{ap}$$

7.23 $$k_4 = 6 \times \tan^2\alpha_{ap}$$

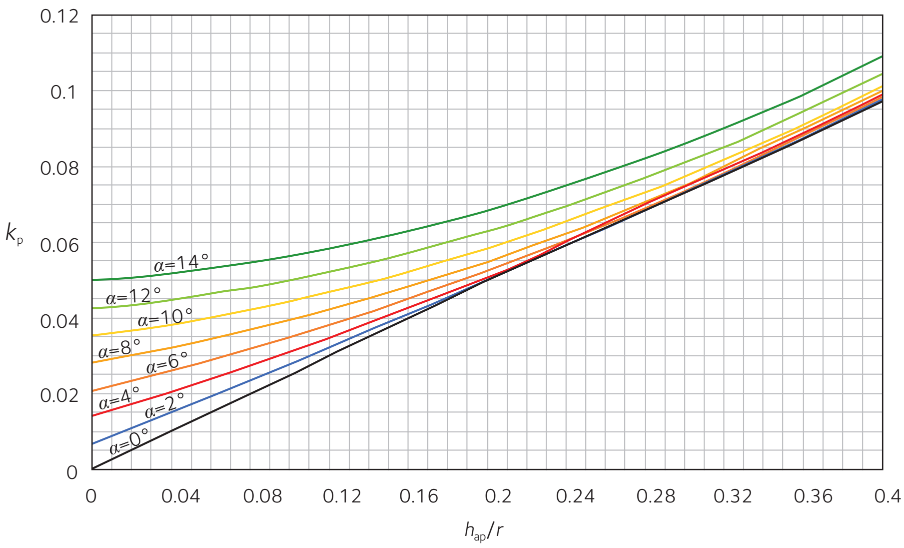


Figure 7.12 Factor k_p according to Eurocode 5 for different values of h_{ap}/r, glulam class GL30c

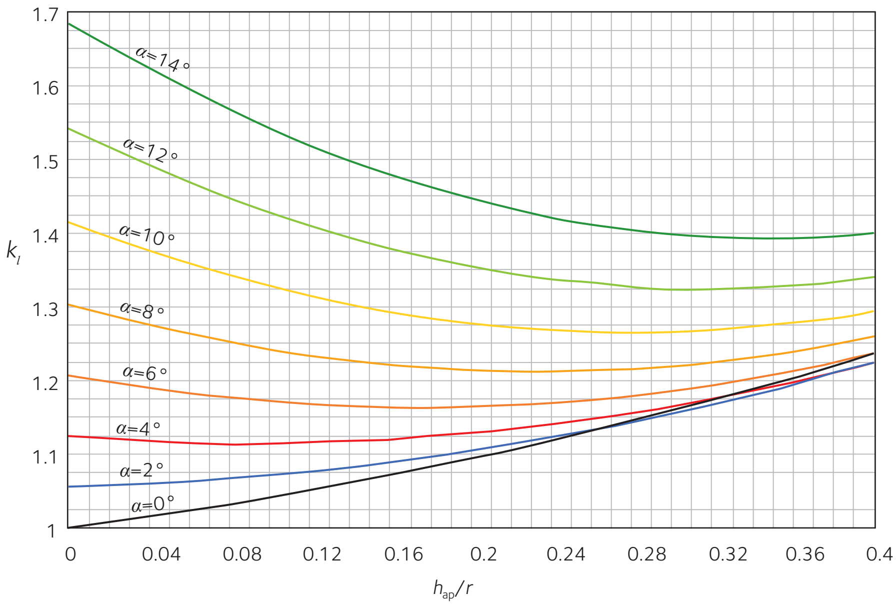


Figure 7.13 Factor k_l according to Eurocode 5 for different values of h_{ap}/r, glulam class GL30c

k_l is a factor determined by finite element analysis that takes into account the geometry of the beam. *Figure 7.13* shows the variation of k_l as a function of h_{ap}/r. Separate curves are plotted for different apex angles, α_{ap}.

The bending strength of curved lamellas shall be reduced by taking into account the eigenstresses that occur when the lamellas are bent during the manufacturing of the structural element. This can be done by multiplying the basic value of the bending strength $f_{m,d}$ by a reduction factor k_r.

$$\sigma_{m,d} = k_r \times f_{m,d} \quad (7.24)$$

The value of k_r decreases with decreasing ratio r_{in}/t, *see table 7.2*.

In normal cases, the final design of curved and pitched cambered beams will include the following checks:

- Bending strength at a distance x_0 from the support, for pitched cambered beams and curved beams with varying cross sectional depth. (Sometimes: also check bending stress at apex).
- Shear and bearing strength at the support.
- Perpendicular to the grain tensile strength at the apex.
- Lateral-torsional buckling (normally this check is performed for an isolated part of the beam between two adjacent purlins. For curved beams this part can be approximated as a straight member).

Table 7.2 Reduction factor for bending strength k_r according to Eurocode 5 as a function of the ratio r_{in}/t, where r_{in} is the radius of curvature and t: thickness of the lamination

r_{in}/t	k_r
≥ 240	1
< 240	$0.76 + 0.001 \times r_{in}/t$

Cow shed, Lyrestad, Sweden.

7.3.4 Preliminary design of simply supported curved and pitched cambered beams subjected to uniformly distributed load

General indications concerning the preliminary design of simply supported curved and pitched cambered beams will be given in this section. Some relevant geometric parameters useful for their design are reported in *table 7.3*.

As for tapered beams, the design load q_d, the roof slope α and the span l, are normally known parameters.

The first step is to determine the depth of the beam at the position of maximum bending stress, i.e. at $x = x_0$ from the support. As for tapered beams, it is assumed therefore, that the location of x_0 is at $l/4$ from the support. Moreover, it is assumed that the bending strength of the beam $f_{m,d}$ should be reduced by a factor $k_{m,\alpha} \approx 0.9$. With these assumptions, the required beam depth at $x = x_0$ can be estimated by means of the following equation:

7.25 $$h_{x_0} = \frac{3 \times l}{4} \times \sqrt{\frac{q_d}{b \times (0.9 \times f_{m,d})}}$$

The beam depths at the support and at the apex can then be calculated taking into consideration the geometric relationships between different parameters given in *figure 7.14, page 115*.

Unsymmetrical snow load according to EKS 10 can be treated in the same way as for pitched beams. When establishing the initial design values, the load on the leeward side is calculated as being evenly distributed over the entire beam. More accurate calculation methods can then be used in the final design.

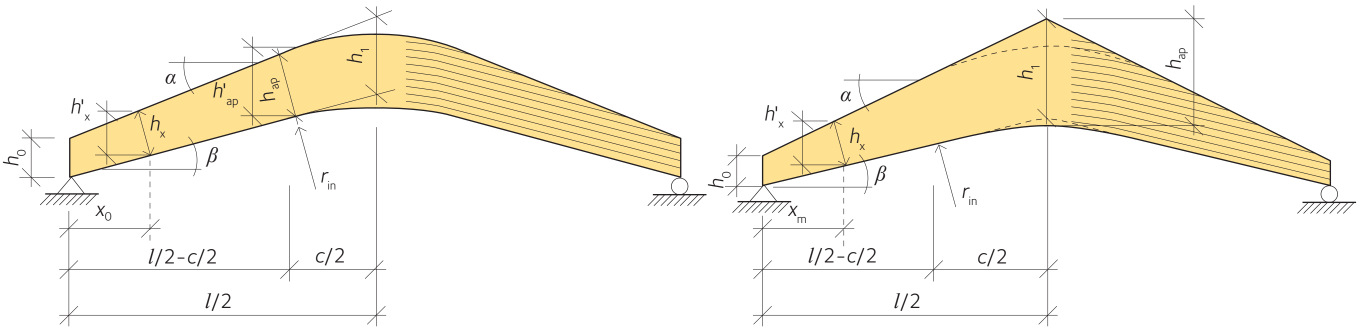
Table 7.3 Geometric parameters for preliminary design of curved and pitched cambered beams

Type of beam	Breadth b	Depth at the support h_0	Depth at the apex h_{ap}	Radius of curvature r
Curved	$l/140 - l/120$	$\geq l/30$	$\approx l/17$	≥ 10 m
Pitched cambered	$l/100 - l/80$	$\geq l/30$	$\approx l/13$	≥ 10 m

7.4 Strengthening for tensile stresses perpendicular to the grain in double tapered, curved and pitched cambered beams

In the previous sections, equations for calculation of tensile stresses perpendicular to the grain in the apex areas of curved, pitched cambered or double tapered beams according to Eurocode 5 have been shown. Of these three beam types, the pitched cambered beam is normally the one most prone to develop cracks due to perpendicular-to-grain stresses at the apex area. As a first attempt to reduce the negative effect of such stresses, the top of the apex can be made loose. In practice, this means that the top of the beam is connected to the underlying part of the beam only by means of a few mechanical connectors, e.g. screws, *see figure 7.15*. In this way, the static behaviour of the pitched cambered shaped beam will be practically identical to the behaviour of a similar curved beam, thus with smaller tensile stresses perpendicular to the grain.

With increasing distance from the apex cross-section, the tensile stresses perpendicular to the grain decrease, depending on the beam shape and load arrangement. *Figure 7.16* shows a typical distribution of such stresses in a pitched cambered beam subjected to uniformly distributed load.



a)

$\gamma = \alpha - \beta$

$c/2 = r_{in} \times \sin\beta$

$h_1 = h_0 + l/2 \times (\tan\alpha - \tan\beta)$

$h'_{ap} = h_0 + (l/2 - c/2) \times (\tan\alpha - \tan\beta)$

$h_x \approx h'_x \times \cos\gamma$

b)

$\gamma = \alpha - \beta$

$c/2 = r_{in} \times \sin\beta$

$h_1 = h_0 + l/2 \times (\tan\alpha - \tan\beta)$

$h'_{ap} = h_0 + l/2 \times (\tan\alpha - \tan\beta) + r_{in} \times (1 - \cos\beta)/\cos\beta$

$h_x \approx h'_x \times \cos\gamma$

Figure 7.14 Geometric relationships for a) curved and b) pitched cambered beams

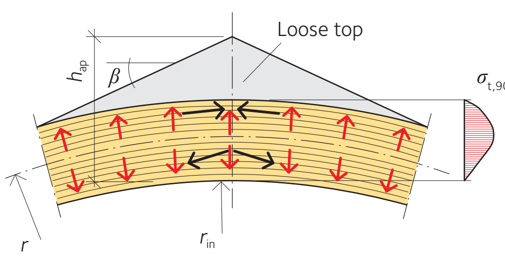


Figure 7.15 Pitched cambered shaped beam with loose top in order to reduce tensile perpendicular-to-grain stresses

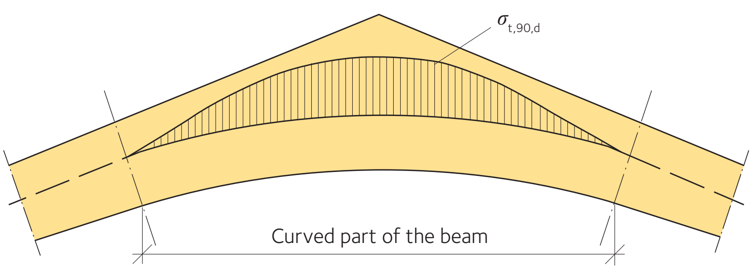


Figure 7.16 Typical distribution of tensile stresses perpendicular to grain in a pitched cambered beam

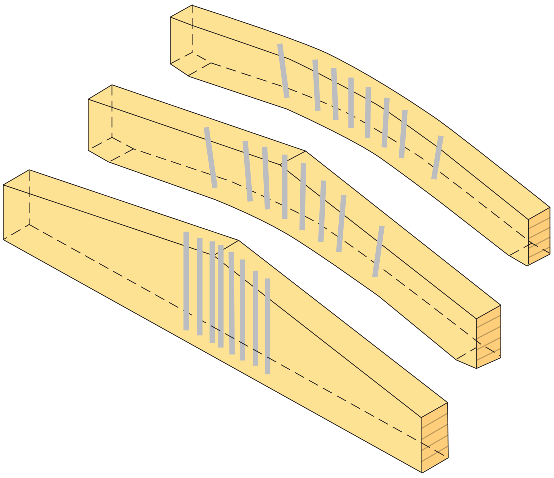


Figure 7.17 Typical arrangements of internal reinforcements for tension perpendicular to the grain in glulam beams

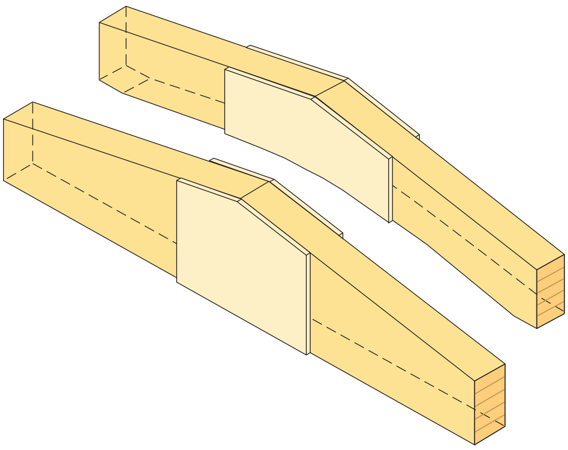


Figure 7.18 Typical arrangements of external reinforcements for tension perpendicular to the grain in glulam beams

Shopping centre, Linköping, Sweden.

7.4.1 Common strengthening methods for adverse tension perpendicular to the grain

Strengthening of curved, pitched cambered or double tapered beams is typically performed by means of:
- Internal reinforcement.
- External reinforcement.

The internal reinforcement technique includes:
- Glued-in threaded rods.
- Glued-in ribbed concrete reinforcement bars.
- Threaded screws.

Figure 7.17 shows typical arrangements of internal reinforcements.
When the *internal* reinforcement technique is used, the reduction in the cross-sectional area due to the presence of bars or screws should be considered during checking of the tensile strength perpendicular to the grain.

The *external* reinforcement technique includes:
- Glued-on plywood or LVL.
- Glued-on boards with grain direction perpendicular to the grain direction of the beam.

In the following, only a proposal for design of internal reinforcement will be given. External reinforcement can be designed in a similar manner.

7.4.2 Design of the reinforcement

Reinforcement should be applied in those areas of the beam with significant tension perpendicular to the grain, mainly where these calculated stresses exceed the corresponding strength values. Typically, reinforcement is applied according to *figure 7.19*.

Pitched cambered beams and curved beams are normally manufactured with straight parts at their ends and with a curvature in their central parts. For such beams, tensile stresses not only occur within the curved apex area, but also reach into the straight parts of the beams, *see figure 7.16, page 115*. However, due to the small magnitude of tension stresses in the straight parts, reinforcement is commonly only applied in the part of the beam that has a curvature.

Occasionally, curved beams can be manufactured without straight parts. In such cases, it is generally not necessary to reinforce the beam over its entire length. With ordinary curved beams, if the radius of curvature is not too small, it is sufficient to reinforce the beam only around the corner section, over a length $l/8$ on either side of the corner line, where l is the total span of the beam. The reinforcement is of course only needed in cases where the calculated shear stress exceeds the corresponding design stress.

In double tapered beams, when necessary, reinforcement should be applied at the apex area symmetrically with respect to mid-span, along a part of the beam with a length equal to the depth of the beam at the apex.

The following proposal takes into account the resulting tensile force in a simplified way: in the central half of the apex area the maximum tensile stress is assumed to be present, whereas in the outer quarters two third of the maximum value is considered.

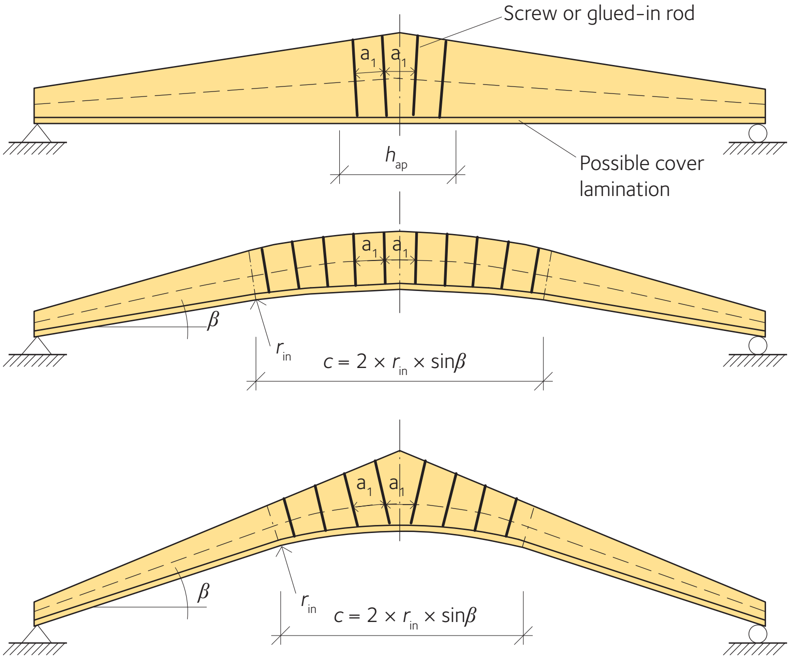


Figure 7.19 Reinforcement arrangement in double tapered, curved and pitched cambered beams

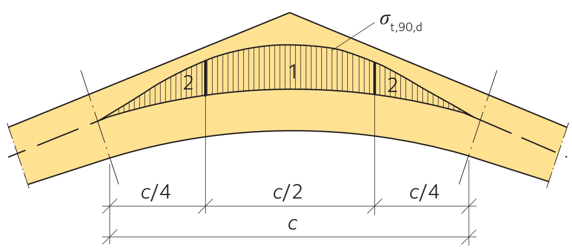


Figure 7.20 Definition of "central half (1)" and "outer quarters (2)" of the apex in a pitched cambered beam

The design tensile force in the reinforcement (i.e. in each distinct rod) in the central half of the apex area consequently results as:

$$F_{t,90,d} = \frac{\sigma_{t,90,d} \times b \times a_1}{n} \qquad 7.26$$

where:

- $\sigma_{t,90,d}$ is the design tensile stress perpendicular to grain determined according to *equation 7.13, page 111*.
- b is the beam width.
- a_1 is the spacing of reinforcement along the beam (recommended: 250 mm ≤ a_1 ≤ 0.75 × h_{ap}, where h_{ap} is the depth of the beam at the apex).
- n is the number of groups of reinforcement bars in the direction perpendicular to the longitudinal axis of the beam (n = 1 or 2, *see figure 7.21*).

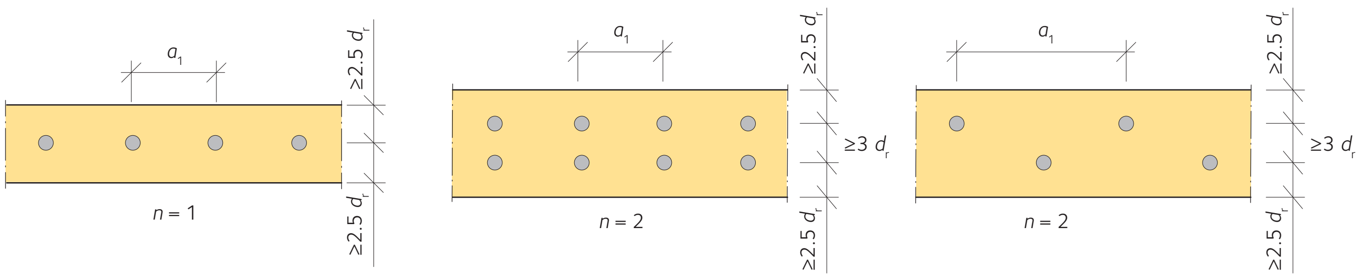


Figure 7.21 Definition of "*n*" and a_1 in *equation 7.26* and *equation 7.27* ("*n*" is the number of groups of reinforcement bars in the direction perpendicular to the longitudinal axis of the beam). d_r is the outer thread diameter of the reinforcement.

Malmö Central Railway Station, Sweden.

For the outer quarters of the apex area follows:

7.27
$$F_{t,90,d} = \frac{2}{3} \times \frac{\sigma_{t,90,d} \times b \times a_1}{n}$$

The design check to be performed is:

7.28
$$F_{t,90,d} \leq R_{ax,d}$$

where $R_{ax,d}$ is the design axial strength of one glued-in or screwed-in steel rod.

When applying *equation 7.28*, it should be kept in mind that the tensile force $F_{t,90,d}$ is resisted by shear stresses in the surface area between the steel rod and the wood, which are not evenly distributed over the length. However, these stresses are normally considered as uniformly distributed, *see figure 7.22*. Moreover, the above part of the rod (or of the screw) is subjected to upwards pull whereas its lower part is subjected to downwards pull. For this reason, in *equation 7.28* the withdrawal capacity of the rod (or of the screw) shall be determined for a length equal to l_{ad}, *see figure 7.22*. For fully-threaded screws or fully glued-in rods inserted through the entire beam depth, l_{ad} can be taken as one half of the rod (or screw) length.

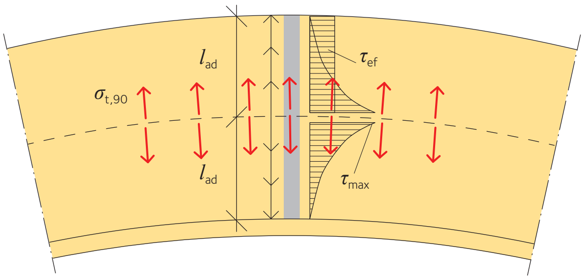


Figure 7.22 Shear stress in the reinforcement of a curved beam

Trusses

A truss is a structure comprising one or more triangular units constructed with straight (or nearly straight) members the ends of which are connected at joints referred to as nodes. Trusses are composed of triangles, which are geometrically stable shapes. In fact, in a triangle the angles are fixed and cannot get larger or smaller without breaking at the joints, unlike a rectangle, for example, which can turn into a parallelogram.

In a planar truss all the members and nodes lie in a two-dimensional plane, while a space truss has members and nodes extending into three dimensions. Timber trusses are usually planar, and normally simply supported at two points. The remainder of this chapter deals with such trusses.

Timber trusses generally provide an economic solution for spans over 25 – 30 m. The truss design for roofs allows the ducts and pipes required for the building's services to be installed through the truss web.

For large spans, where lighter-weight roof specifications can be expected, trusses are spaced at 5 – 12 m centres, normally carrying purlins at 1.2 – 2.4 m spacings and supporting corrugated sheeting. Alternatively purlins are replaced by heavier corrugated sheeting directly applied to the trusses. Economy is usually achieved if the truss spacing increases with the truss span.

8.1 Geometry of trusses

The structural function of a truss is to support and transfer loads from the points of application, usually purlins, to the points of support as efficiently and economically as possible. The efficiency depends on the choice of a suitable truss type consistent with the architectural requirements and compatible with the loading conditions. Typical 'idealised' truss types for two loading conditions are sketched in *figure 8.1*.

With a symmetrical system of loading (particularly important in the case in *figure 8.1 b)*, which is a four-hinged frame and therefore unstable) the transfer of loading is achieved without internal web members, because the funicular of the forces matches the geometry of the structure. Unfortunately, it is seldom possible to use a lay-out omitting internal members, because unbalanced loading conditions can nearly always occur from snow, wind or permanent loading. Unbalanced conditions can also occur due to manufacturing and erection tolerances. Nevertheless, the engineer should try to use a truss contour closely related to the idealized one (the funicular of loads), adding a web system capable of accommodating unbalanced loading. In this way the forces in the internal members and their connections are minimized, with consequent design simplicity and economy. Two possible web systems for the case of *figure 8.1 b)* are shown in *figure 8.2, page 120*.

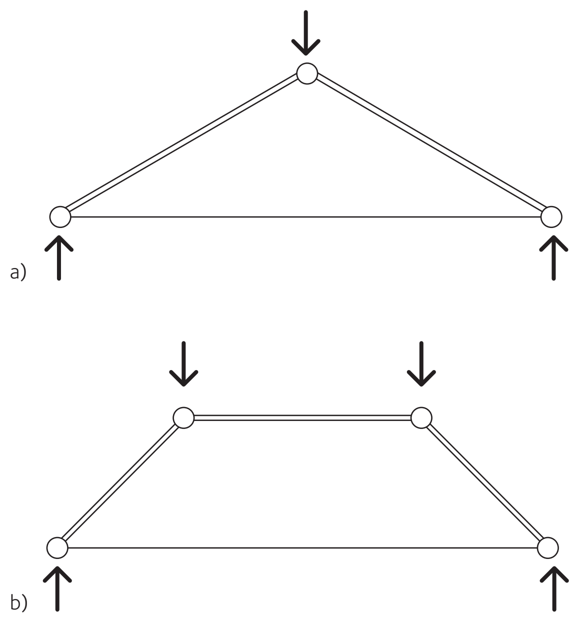


Figure 8.1 Examples of idealised truss arrangements for two different loadings
a) A stable three-hinged truss,
b) an unstable four-hinged mechanism.

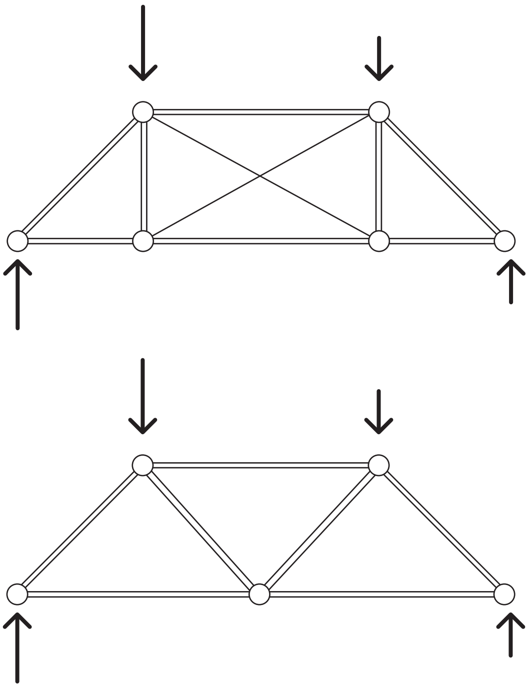

a)
b)

Figure 8.2 **Examples of web system for unbalanced load conditions**

Explanations of terms:
Chord members = members that bound a planar truss at the top and bottom.
Web members = members that connect the chords together.
Vertical web members = web members that run perpendicular to a chord.
Diagonal web members = oblique web members.
Transverse web members = cross-bracing web members.

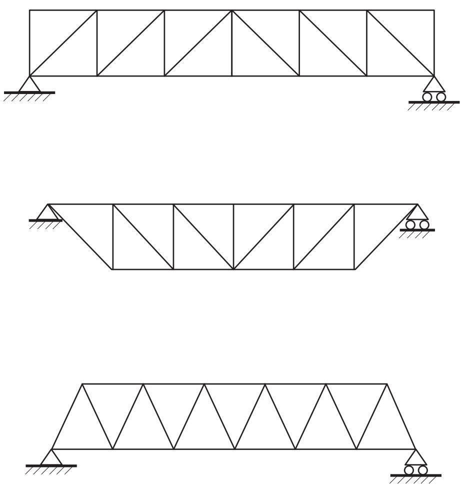

a)
b)
c)

Figure 8.3 **Examples of parallel-chord trusses**
a) Howe (diagonal in compression),
b) Pratt (diagonal in tension),
c) Warren (diagonals in alternating compression and tension).

Where the required architectural profile conflicts with the optimum structural profile, high stresses may be introduced into the web system and the connections. Economy must then be achieved by adopting the most suitable structural arrangement of internal members with which it is possible to create an economical balance between materials and workmanship. In order to achieve this, the following aspects should be considered:

- The number of joints should be kept as low as possible because the workmanship for each joint is expensive, and also the joint slip at each node generally adds to the overall deflection of the truss.
- The slenderness of the compression chords and the internal struts must not be excessive.
- Local bending of the chords must not be too large.
- The angle between internal diagonals and the chords must not be too small.

Transportation is frequently a limiting factor for deep or long-span trusses. Trusses deeper than 3 m, or longer than 20 – 25 m, require special attention. The transport problem can usually be overcome by a partial or complete disassembly of the truss.

Large trusses may have main members spliced at one or more locations in the span, and in some cases the entire assembly can be carried out on site, although it is always preferable to carry out an initial assembly in the glulam factory to ensure correct fit before disassembly for transport.

Large span pitched trusses, for example as that shown in *figure 8.4 c), page 121*, can be fabricated in two halves linked together on site with a loose centre tie.

A wide range of alternatives is available for the general shapes of the trusses. One of the most common shapes for glulam trusses is the three-hinged truss. This is dealt with in *Chapter 9, page 131*. Some other commonly used shapes are described in the following sections.

8.1.1 Parallel-chord truss

Parallel-chord trusses are frequently specified as an alternative to ply web or glulam beams for long spans (say 25 – 30 m), where beams may be uneconomical. The loads in the web members are frequently very large, which causes some difficulty in providing adequate joints. The choice of web configuration is usually between the Howe (diagonals in compression, *see figure 8.3 a)*) the Pratt (diagonals in tension, *see figure 8.3 b)*) and the Warren type (diagonals in alternating compression and tension, *see figure 8.3 c)*).

The advantage of choosing a configuration with diagonals in compression rather than in tension is that the joints between diagonals and chords are relatively simple to construct, due to the fact that they can transmit loads by bearing stress. The disadvantage is that the relatively long diagonals will be subjected to compression, thus prone to buckling. The Pratt-type configuration has the advantage that it can also be supported at its upper chord, as shown in *figure 8.3 b)*, which means that the centre of gravity of the truss is below the line between the two supports. This allows for easier erection, due to the fact that, in case of initial out-of-plumb, the self-weight of the truss acts as a stabilising force against overturning.

Parallel-chord trusses are often designed with a pre-camber that corresponds approximately to the deflection due to self-weight plus one half of the main variable load (e.g. snow load).

8.1.2 Pitched trusses

For uniformly distributed loads, the shape of pitched trusses fits the bending moment diagram reasonably well and is compatible with traditional roofing materials, such as purlins and corrugated sheeting. A portion of the applied load is transferred directly through the top chord members to the points of support, while the web members transfer loads of relatively small to medium magnitude, and the joints can usually be designed to take these loads with little difficulty. For trusses with a horizontal bottom chord, web configurations similar to those illustrated in *figure 8.3* can be adopted. In *figure 8.4* some different types of double pitched trusses are shown.

Double pitched trusses may have raised bottom chords to give extra central clearance, *see figure 8.4 c)* and *d)*. This can be particularly useful in storage buildings with central access.

8.1.3 Bowstring and lenticular trusses

For large-span uses, both bowstring and lenticular trusses, *see figure 8.5*, can be very economical.

With uniform loading and no large concentrated loads, the chords of the truss support almost all of the applied loading. This means that web members are only lightly loaded, so that the connections between diagonal and chords can be simple and inexpensive. With these static systems, glulam trusses with spans in excess of 60 – 70 m are easily achieved.

From the point of view of statics, a parabolic profile is the most efficient choice to support uniform loading. However, practical manufacturing considerations usually make it more convenient or necessary to adopt a circular contour for chord members. The top chord member of bowstring trusses and each chord in lenticular trusses usually consists of two or more rigidly jointed curved glulam elements. The bottom chord of bowstring trusses usually consists of a number of steel rods.

a)
b)
c)
d)

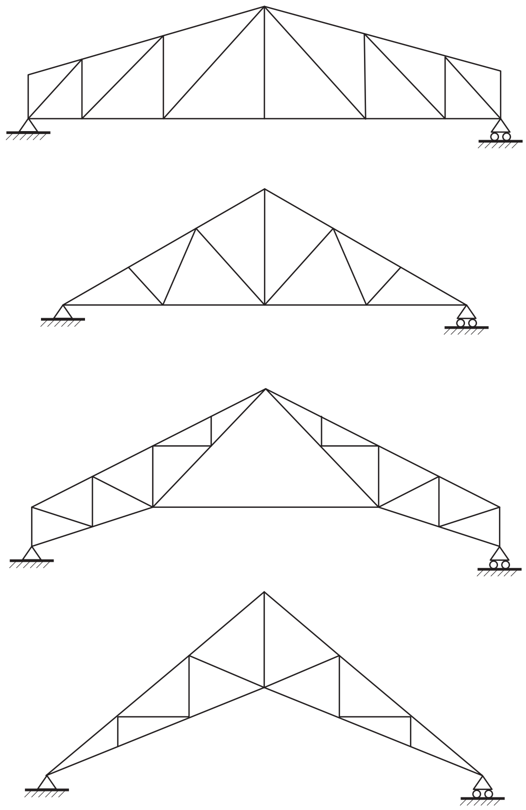


Figure 8.4 Examples of double pitched trusses
a) Trapezoidal (Howe-type),
b) triangular with horizontal bottom chord,
c) triangular with raised bottom chord,
d) scissor type.

a)
b)
c)

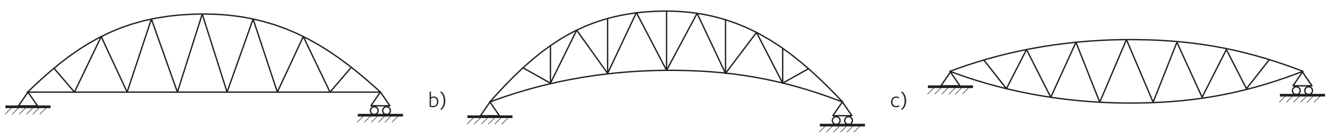


Figure 8.5 Examples of bowstring and lenticular trusses
a) Bowstring with horizontal bottom chord,
b) bowstring with raised bottom chord,
c) lenticular truss.

8.1.4 Forces in truss members

As previously mentioned, the choice of structural type has a great influence on the magnitude of normal forces in the truss bars. As an indication of the difference in magnitude of forces in members dictated by the choice of truss type, force coefficients are presented in *figure 8.6* for three basic truss types at typical span-to-depth ratios. These coefficients indicate the magnitude of the forces in the bars and are defined as the ratio $N/(q \times l)$,where N is the force acting in the bar, q is a uniformly distributed unitary load and l is the span of the truss. The left hand side of the figure refers to simply supported trusses subjected to a uniformly distributed unitary load ($q = 1$). The right hand part refers to similar trusses, but subjected to an unbalanced load ($q_l = 1$ on the left half span and $q_r = 0.5$ on the right half span). Red digits indicate tension, blue digits indicate compression (under downward load).

As shown in *figure 8.6*, tension and compression are nearly constant in both upper and bottom chords of bowstring trusses with a para-

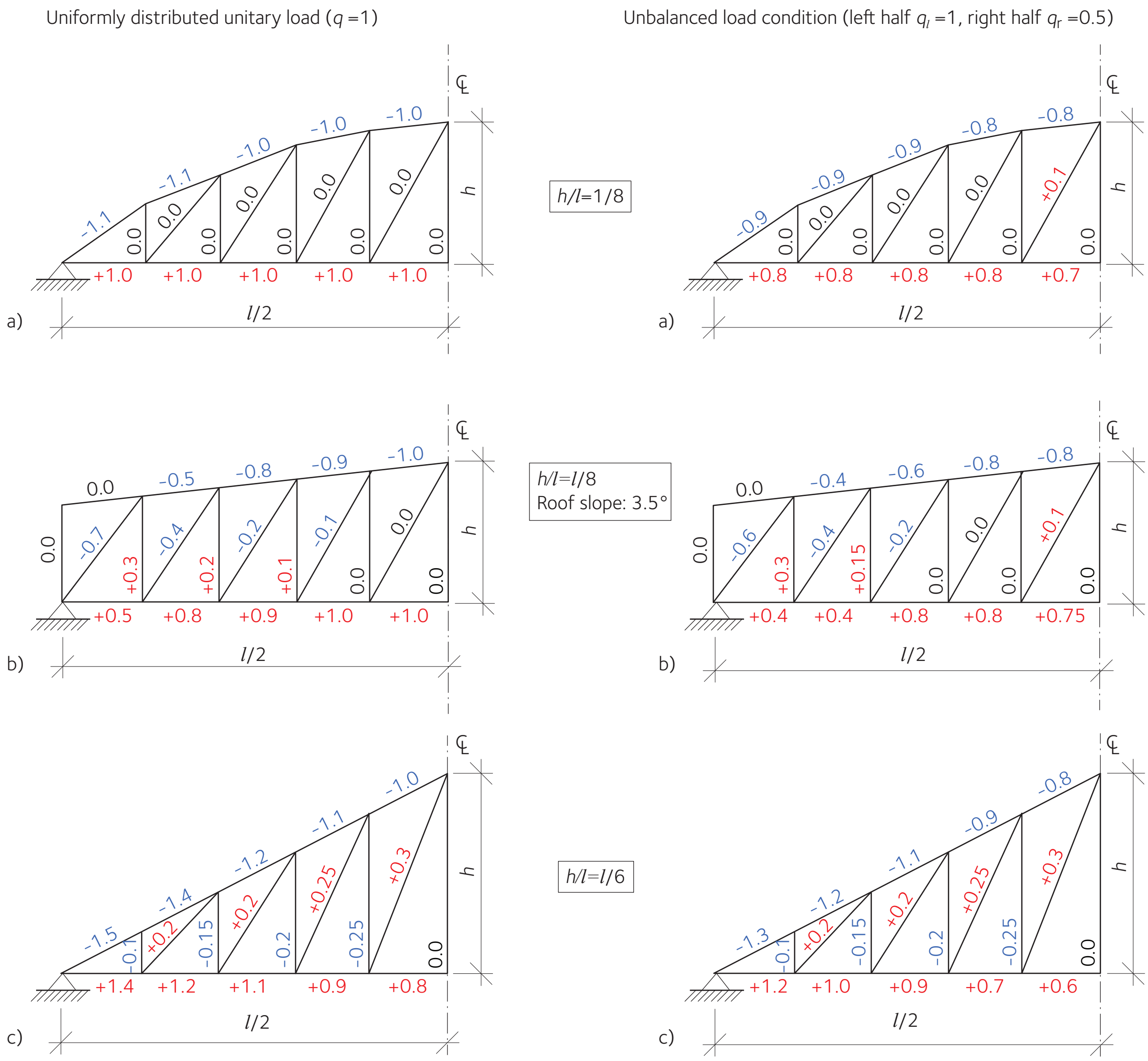


Figure 8.6 Magnitude of normal forces in three different truss types: a) Bowstring, b) pitched trapezoidal and c) pitched triangular (bottom).
Left: uniformly distributed unitary load ($q = 1$).
Right: unbalanced load ($q_l = 1$ on the left half span and $q_r = 0.5$ on the right half span).

bolic upper chord. This implies that the web members (verticals and diagonals) will be nearly unloaded. Therefore, joints — with the exception of the joints between top and bottom chords at the support and possible erection joints in the bottom chord — will be relatively inexpensive for this kind of truss.

Large normal forces occur in the diagonals of pitched trapezoidal trusses, especially those closest to the supports. Therefore, joints may be rather complicated at these locations. Moreover, particular attention should be paid to buckling of these diagonal members.

In pitched triangular trusses, relatively small normal forces occur in the web members. However, in the upper chord and bottom chord, respectively, very large compression and tension forces occur at locations close to the supports.

It should be noted, however, that some drifted snow load distributions, such as that with triangular load distributions on each half of the truss, may give rise to forces in the truss members that can differ quite significantly from those shown in *figure 8.6, page 122*.

Sports hall, Umeå, Sweden.

8.2 Conceptual design

Generally, architectural considerations determine the shape and possibly the slope of the roof. The need to accommodate services, such as ventilation ducts that pass through the truss, can also influence the choice of profile. However, for economic reasons "rules of thumb" concerning depth-to-span ratios, maximum span etc. should be followed. *Figure 8.7* gives indications for preliminary design of three typical truss types.

The following issues should be kept in mind during the preliminary design process:

- The secondary load bearing system, e.g. purlins in the direction perpendicular to the trusses, should be compatible with the truss triangulation system so that the transmission of loads occurs at (or as close as possible to) the nodes.
- For the sake of economy, design with diagonals in compression (e.g. the Howe type) is often preferred for timber trusses designed for moderate loads in the members. If the diagonals are in tension, the nodes will have to transmit tensile forces. In timber structures, nodes subjected to tension are normally rather complicated and time-consuming to construct, and almost necessarily require steel assemblies in order to transfer forces from one member to another. On the other hand, truss design with diagonals in compression allows the diagonal members to transfer their forces to the vertical and horizontal members by bearing directly against them; and timber members tend to be thick enough to make it less likely for buckling to be a problem, than e.g. in steel members. When the loads in a truss member are very large, however, load transmission by bearing pressure may not be sufficient. In such a case, the use of steel assemblies would be indispensable and from the point of view of economy it does not matter whether the diagonals are arranged so that they are in tension or in compression.
- In order to ensure an efficient truss, the angle between the diagonals and the chords should be close to 45° ± 15°.

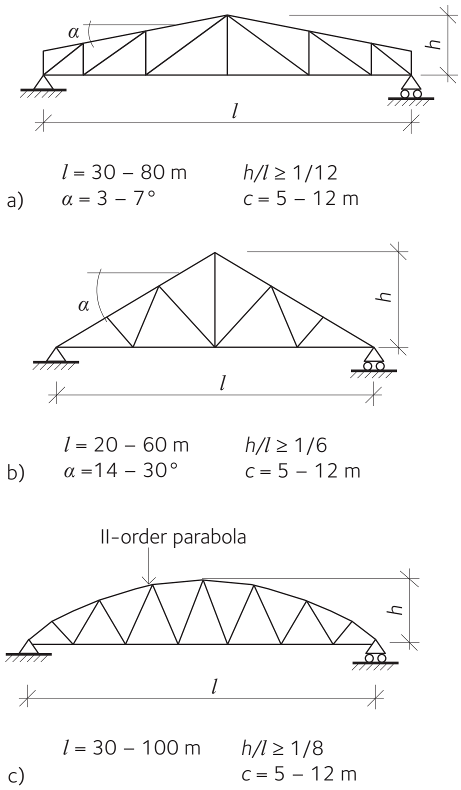
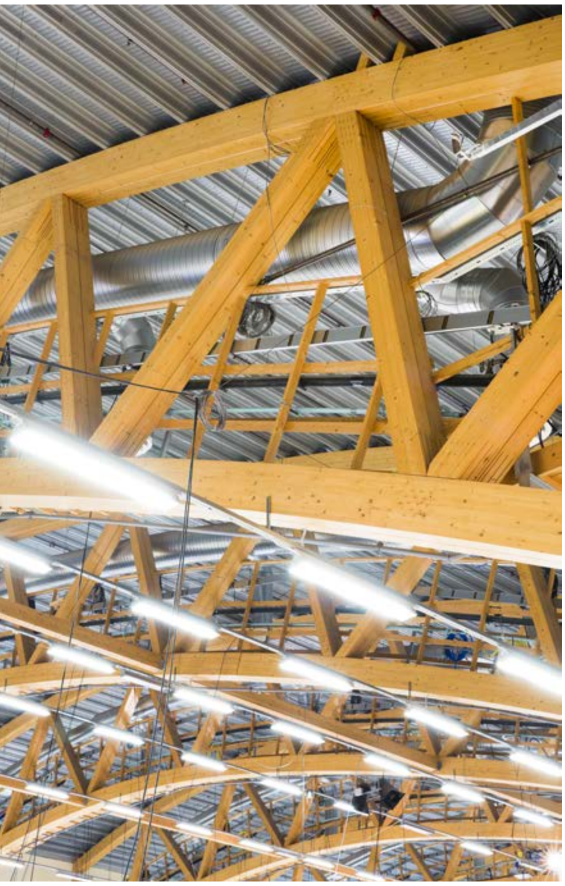


Figure 8.7 Preliminary design of three different truss types. "c" is the centre-to-centre distance between adjacent trusses.



Concert and Congress hall, Jönköping.

8.2.1 Preliminary dimensioning of the members

The cross-sectional area of the members can be preliminary decided on the basis of a simplified analysis using a simple model with hinges at each member end and without eccentricities. This allows for a rapid evaluation of the forces in the members, which will later be used for the preliminary design. The static system used for the simplified analysis of two typical truss profiles is shown in *figure 8.8*. It should be pointed out that the approach shown below for calculation of internal forces is often on the safe side, since it is based on the assumption of perfectly hinged bars.

Pitched triangular truss

The maximum forces occur at the members closest to the support. By equilibrium considerations at the support node, the maximum tension force can be calculated by:

8.1 $$B_{max} = \frac{(3q_1 + q_2)\, l}{8 \times \tan\alpha}$$

The maximum compression force is:

8.2 $$H_{max} = \frac{(3q_1 + q_2)\, l}{8 \times \sin\alpha}$$

For common pitched triangular truss profiles, the force in the most loaded web element is of the order of magnitude 1/5 – 1/3 of H_{max}.

Howe-type triangular truss

The maximum forces in the chord members occur in the region at mid-span. The maximum tension force B_{max} and maximum compression force H_{max} can be calculated by:

8.3 $$B_{max} = H_{max} = \frac{q \times l^2}{8 \times h}$$

The maximum force in web members occurs at the diagonal closest to the support. By equilibrium considerations:

8.4 $$D_{max} = D_1 = \frac{q \times l}{2 \times \sin\alpha}$$

Once the forces in the most loaded bar have been estimated, the members can be sized. Below, some general rules concerning the design process of trusses are discussed.

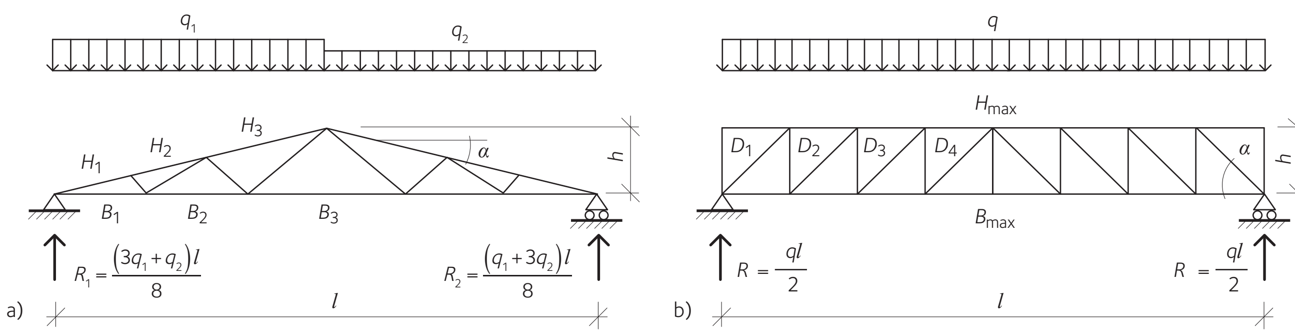


Figure 8.8 Static system used for the preliminary design of:
a) pitched triangular truss
b) Howe-type truss.

8.2.2 General rules for sizing of the members

The breadth-to-depth ratio of the members should be chosen taking into consideration the type of connection that will be used for the nodes of the truss. As an example, consider a connection made of slotted-in plates and dowels, which is very often used in nodes of large-span trusses, *see figure 8.9*. In order to increase the load-bearing capacity of the nodes, it is often necessary to use a large number of slotted-in plates; this requires the choice of relatively wide cross-sections so that the plates can be accommodated.

The bending stiffness of the single members in the plane of the truss should be kept reasonably small in comparison to the bending stiffness of the assembled truss. In fact, in such a case, the bending moments at the nodes will be small and can normally be neglected; thus, the truss can be analysed with satisfactory approximation by assuming all its members to be hinged at their ends. The presumption that the bending stiffness of the members is small in comparison to the bending stiffness of the assembled truss, is normally fulfilled if the chord depths do not exceed 1/7 of the truss depth, *see figure 8.10*.

For this reason, nearly square-shaped cross-sections are often used for compression members and square or rectangular cross-sections (with the largest side in the direction perpendicular to the plane of the truss) for tension members. The choice of relatively shallow cross-sections (in the plane of the truss) also has the advantage of facilitating the design of nodes without eccentricities.

Eccentricities at the nodes should be avoided in all cases, mainly due to the risk of brittle failure caused by tensile stresses perpendicular to the grain, which originates from secondary bending moments, *see figure 8.11*. This means that member centre-lines that converge to a node should meet at one and the same point, i.e. the node axis. With a well-planned conceptual design, it is most often possible to create nodes without eccentricities.

During design, it is very important to take into account the reduction in strength due to the presence of slots and dowel holes, particularly for members subjected to tension. As a rule, during preliminary design, the net area A_{net} can be assumed as 60 to 80 percent of the gross area A of tension members to take into account holes and slots, typically $A_{net} = 0.7 \times A$.

The choice of the size of the truss member cross-section, as well as the choice of connection types to be adopted for the assembly of nodes, is often influenced by the required fire resistance for the given structure, typically fire class R30.

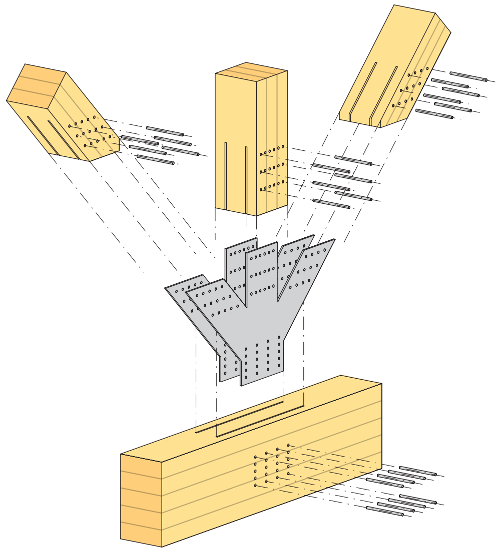


Figure 8.9 **Typical truss node with slotted-in plates and dowels**

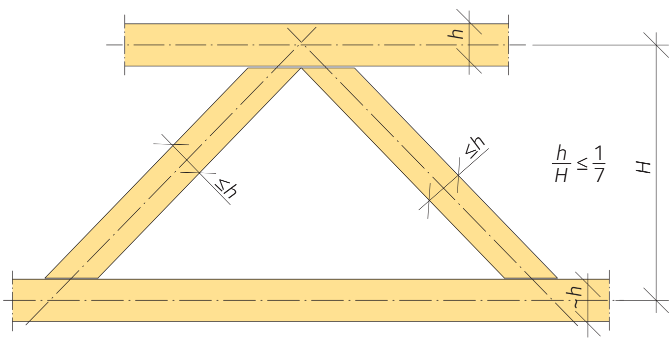


Figure 8.10 **Ratio between member depths and truss depth to reduce the influence of bending moments**

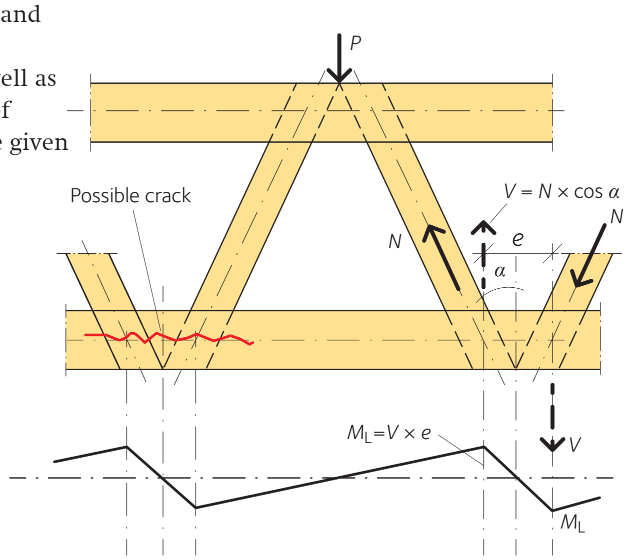


Figure 8.11 **Bad example of truss with eccentricities at the nodes.** In the figure, the diagram of eccentricity moments and a possible crack location due to tension perpendicular to the grain are also indicated.

8.3 Calculation of forces in the members and connections

An ideal truss would be represented by a static system with "perfect" hinges at each node, no eccentricities at the nodes and "point" loads applied only at the nodes, *see figure 8.12*. In such a model there are only axial forces in the members, with no moment or shear forces.

In timber structures, however, such an ideal situation very seldom occurs. For example, at node positions, the joints give a certain degree of rotational restraint, and slip also occurs due to the deformability of the connections. Moreover, the upper chord and bottom chord of the truss are normally continuous members, not hinged at their intersections with web members as in the ideal case. A more advanced model would include translational and rotational springs at the node positions, *see figure 8.13*.

However, a rigorous estimation of the rotational and translational stiffness of the connections is nearly impossible to achieve in practice. Therefore, it is recommended to analyse one upper and one lower bound situation, namely: a static system with hinges at the end of each web member, *figure 8.14 a)*, or a static system with web members clamped to the chord members, *figure 8.14 b)*.

The design of members can be performed by using the most severe stress situation between case *a)* and case *b)* of *figure 8.14*.

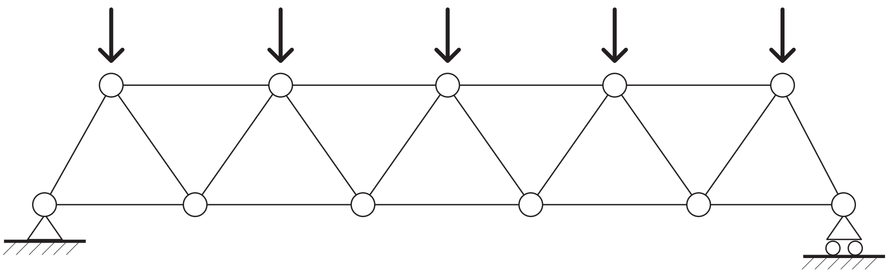


Figure 8.12 **Schematisation of an "ideal" truss, with hinged nodes and loads applied only at the nodes**

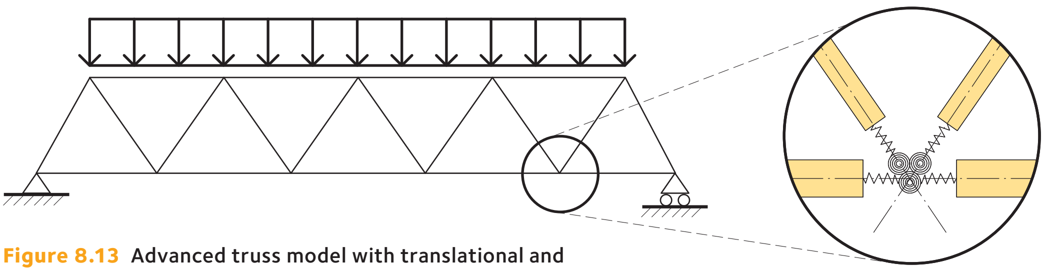


Figure 8.13 **Advanced truss model with translational and rotational springs at the ends of the web members**

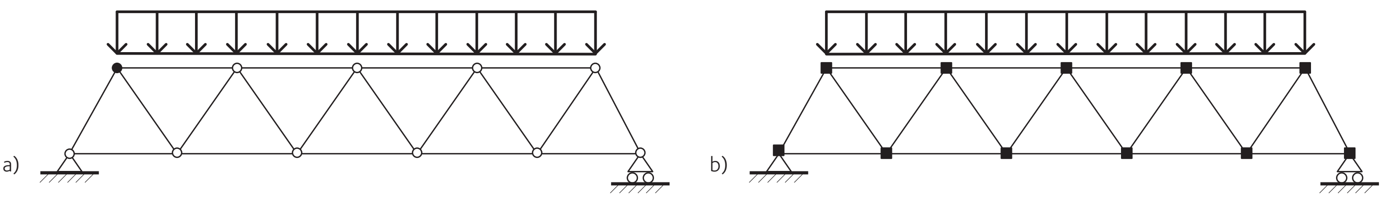


Figure 8.14 **Static system that should be analysed for the determination of bounds for the forces and moments in the members**
Both trusses have continuous top chord and bottom chord. a) Truss with hinged web members, b) truss with moment stiff web members.

The members can be single or composite. Double chords at top and bottom are often combined with single diagonals. Glulam compression members are sometimes combined with tension members of steel, *see figure 8.19*.

Tension members should 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 such members.

Normally, the connections at the nodes are designed as if they were subjected to pure axial forces. However, the calculated axial forces should be incremented by approximately 10 – 15 percent to take into account possible (unplanned) eccentricities and possible rotational fixity provided by the connection.

Buckling of compression members

Compression members and members subjected to combined compression and bending (normally the members in the upper chord of the truss) should be designed according to *Chapter 4, page 53*, taking into account the risk of buckling both in the plane of the truss and out-of-plane.

For chord members in general, and for out-of-plane buckling of web members, the buckling length l_{cr} may be taken as equal to the system length l. The in-plane system length is the distance between the joints. The out-of-plane system length is the distance between the lateral supports.

Table 8.1 Buckling lengths of truss members

Member	Buckling length l_{cr}	
	In-plane	Out-of-plane
Top chord	a	$a_{out,1}$ or $a_{out,2}$
Vertical web member	b	b [1)]
Diagonal web member	c	c [2)]

1) For nodes assembled with e.g. a group of dowel-like connectors, $l_{cr} = 0.9 \times b$.
2) For nodes assembled with e.g. a group of dowel-like connectors, $l_{cr} = 0.9 \times c$.

Symbols shown in *figure 8.15*.

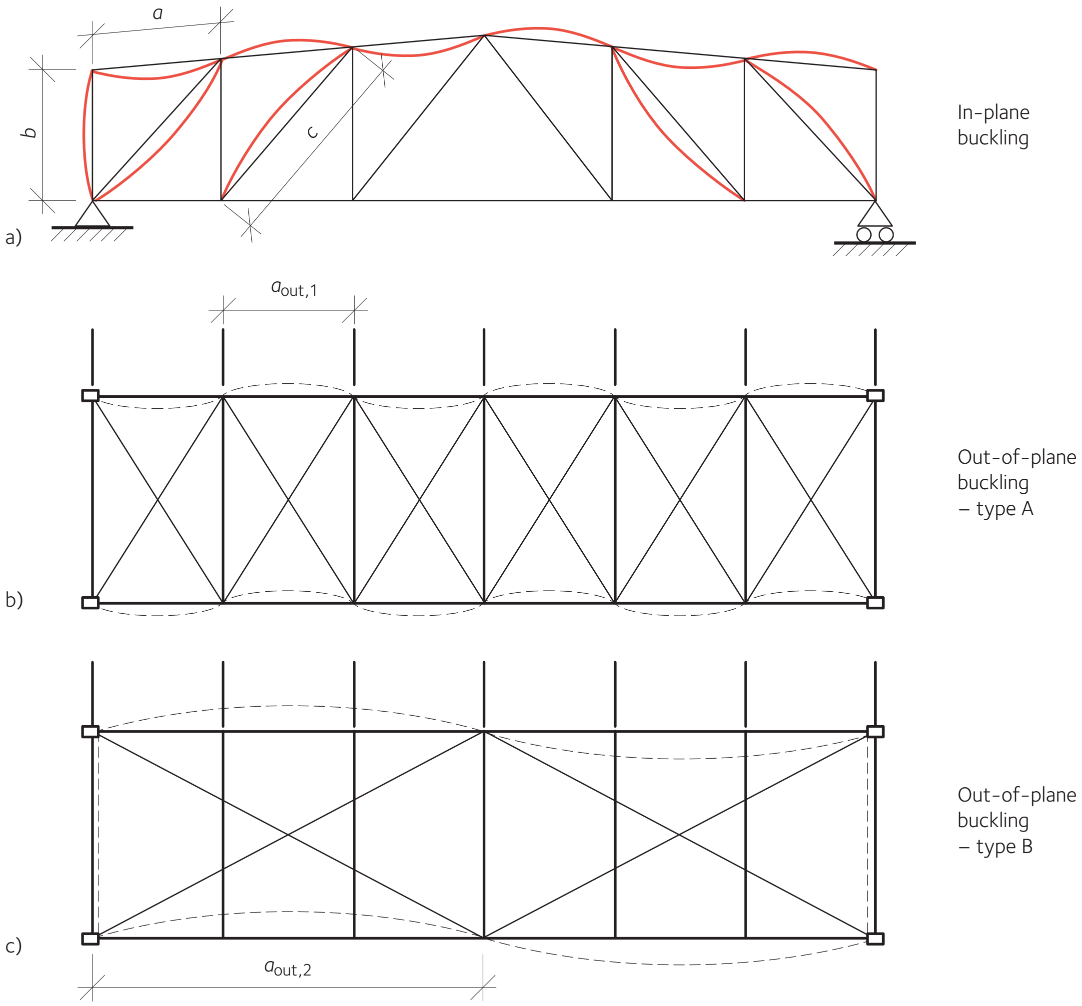


Figure 8.15 Examples of assessing the theoretical buckling lengths in trusses

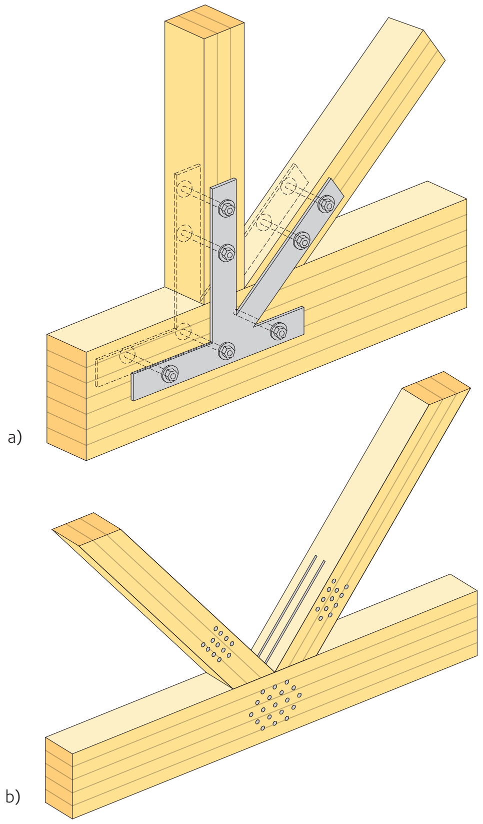


Figure 8.16 a) Truss node with external steel plates and bolts and b) truss node with slotted-in plates and dowels

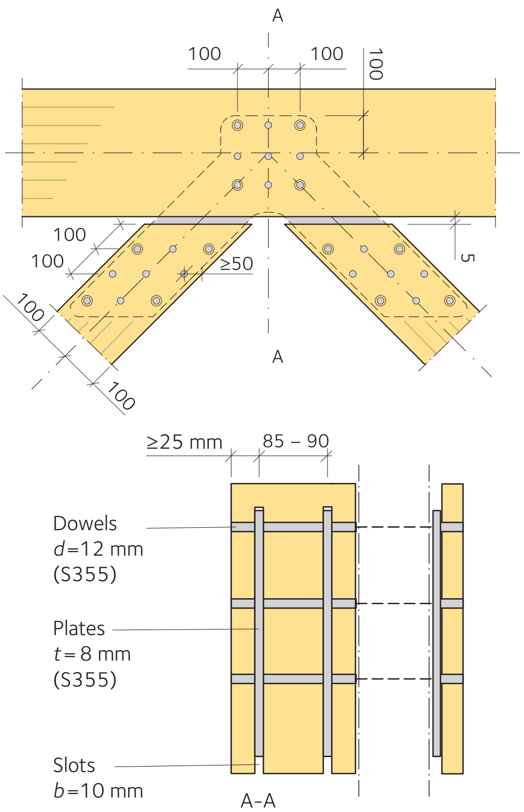


Figure 8.17 Typical truss node used in Norway and Sweden with spacing indications. As many as 8 or 9 inset plates have been used for very heavy glulam trusses, but fewer inset plates across the width are sufficient for shorter spans.

Web members may be designed for in-plane buckling using a buckling length smaller than the system length, provided that the chords supply appropriate end restraint and that the end connections supply appropriate fixity. For nodes assembled with e.g. a group of dowel-like connectors, *see figures 8.16 b)* and *8.17*, the buckling length l_{cr} of web members for in-plane buckling may be taken as $0.9 \times l$.

However, where rotations at the joints are possible, e.g. single dowel-like connections, a value of 1.0 would normally be applicable. *See figure 8.18, page 129.*

In order to reduce the out-of-plane buckling length, it is possible to brace the members, e.g. as shown in *figure 8.15 b)* and *c), page 127*. For a roof with a bracing system type A shown in *figure 8.15, page 127*, the supporting points are defined by the purlins, which are attached to the immovable joints of transverse roof bracing. For a roof with bracing system type B, the upper chord may buckle out-of-plane along a few panels of the web of the truss. It should be noticed that the bottom chord may also be subjected to buckling, when the wind uplift is larger than the permanent loads.

8.4 Serviceability check

Finally, the serviceability limit state of the truss should be controlled, *cf. Chapter 6, page 82*. This is done by checking that the maximum deflection due to serviceability loads is less than that allowed by the codes (normally $L/300$ and $L/200$ for instantaneous deflection and final deflection, respectively). In the deflection calculations it should be kept in mind that slip at the connections has a great influence on the total deflection of a truss. Such an effect can be taken into account in two ways, namely: directly, i.e. by modeling the structures with springs at the nodes, or indirectly, i.e. by modeling the structure as hinged at the nodes and then increasing the calculated deflection by a given factor to take into account slip and rotations at the connections. Such a factor may be taken equal to 1.3 – 1.5 for nodes assembled with bolts, *see figure 8.16 a)* and equal to 1.1 – 1.2 for nodes assembled with dowels, *see figure 8.16 b)* and *8.17)*.

If hand calculations are performed, instead of computer analysis, the effect of changes in member length must also be taken into account. Generally speaking, the serviceability limit state check seldom governs the design of timber trusses, due to the relatively large bending stiffness of such structures.

Very often, trusses are manufactured with a pre-camber (both at the top and bottom chords), the magnitude of which should be approximately $L/150 – L/200$.

8.5 Details

The quality, durability and above all, manufacturing costs of trusses depend to a large extent on the choice of the joints used to build the nodes. Trusses normally include a large number of nodes. It is advisable, therefore, to choose node solutions with the following properties:

- Member centre-lines should meet at one and the same point in nodes.
- Has a concentrated lay-out with small extension.
- Be easy and fast to assemble.

- Appropriate fire resistance, if required.
- Limit the amount of steel parts.
- Be of "standard type", i.e, it should be possible to use the same type of connection for as many nodes of the same truss as possible.

The design of connections is discussed in *Chapter 14, page 198*. Below, some typical truss nodes are described.

Truss nodes joining web members and chords
For trusses consisting of single chords and single web members, the connection types used at the nodes normally include steel plates and dowel-type connections (typically dowels or bolts). See also *section 14.9, page 227*. *Figure 8.16, page 128* shows two types of typical node solutions, *a)* with external steel plate and bolts, and *b)* with slotted-in plates and dowels. Nodes that adopt solutions similar to that of *figure 8.16 b), page 128*, but with several slotted-in plates as shown in *figure 8.17, page 128*, are suitable for trusses with very large spans (up to 70 – 80 m) and/or heavy loads.

In Norway and Sweden, several large glulam trusses, for industrial, commercial, sports buildings and bridges, have been constructed using connections with slotted-in plates and dowels. For such connections it has been common practice in these countries since the beginning of the 1990's to use slotted-in plates with thickness t = 8 mm and dowels with diameter d = 12 mm. Glulam type GL30c or GL30h is used for truss members and common mild steel for the steel parts, e.g. S355. In order to optimise the connection from the point of view of strength, and to ensure satisfactory structural ductility, the spacing between slotted-in plates, the spacing between connectors, the distance between dowel and loaded end and the distance between dowel and unloaded edge should be chosen according to *figure 8.17, page 128*. Special attention should be paid to possible block shear failures, which are particularly relevant in case of several slotted-in plates in combination with a large number of dowels, see *Chapter 14, page 198*.

A node with inset plates and dowels should also contain a number of set screws. This is mainly for two reasons: a) there may be a small risk of dowels falling out, especially in dynamically loaded structures and/or in structures subject to significant moisture cycling; b) eccentricity between the steel plates and the wooden elements may cause the different sub-components to separate under load.

Trusses that consist of several parallel connected members (usually two or three separate elements) are also common. The larger the span and/or the applied loads, the larger the number of parallel members. *Figure 8.18* shows some possible node solutions. In the nodes shown in *figure 8.18 a)* and *b)*, the transmission of forces between diagonal members and chord members is performed via a single bolt that goes through all members. The transmission of forces between the bolt and the individual truss members occurs by bearing stresses between the bolt and nail plates, rather than bearing between bolt and timber. The holes in the timber members of *figure 8.18 a)* are normally manufactured slightly bigger than the corresponding holes in the nailing plates. The nail plates, which are normally reinforced around the hole, are connected to the timber members through a number of nails sufficient to carry the load that the member is designed for. In the node shown in *figure 8.18 c)* the transmission of forces between the tension diagonal members and chord members takes place in a manner similar to that discussed for *figure 8.18 a)* and *b)* . In joint *c)* the transmission of forces between the compression

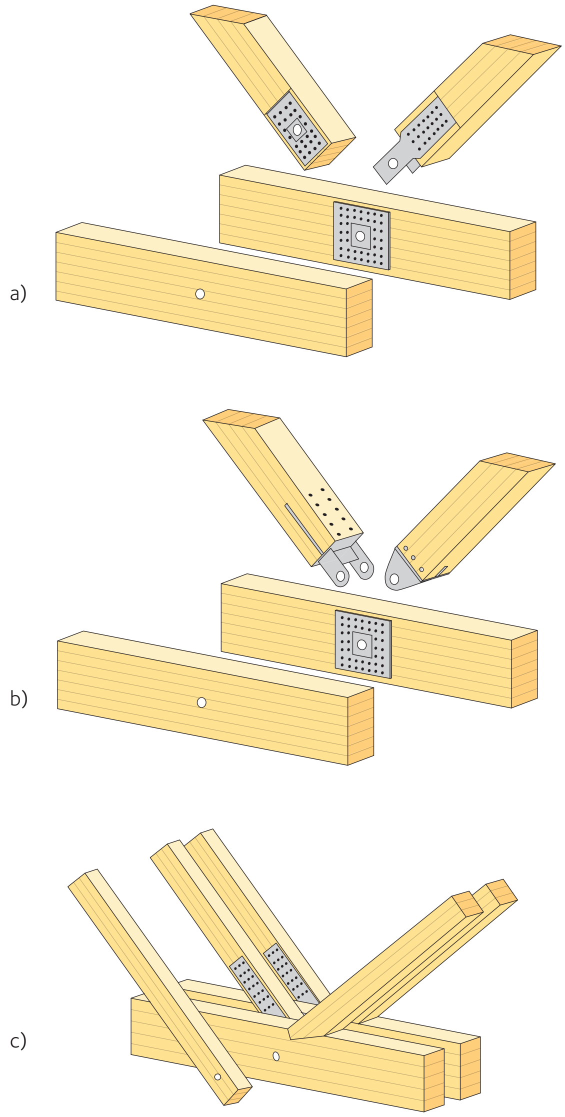


Figure 8.18 **Truss nodes with:**
a) nail plates and single bolt/pin,
b) nail plates , slotted in plates and single bolt/pin,
c) as a) for tension diagonals and "dado joint" (i.e. carpentry joint) for compression diagonals.

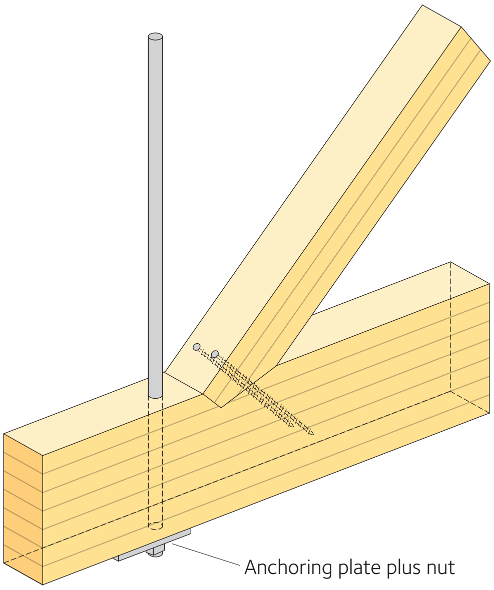


Figure 8.19 **Truss node with timber diagonal in compression and steel rod in tension**

diagonal members and the bottom chord occurs, on the other hand, via direct timber-to-timber bearing stresses.

Howe-type trusses, i.e. trusses with diagonals in compression, can be designed using steel rods as web vertical members, provided that compression does not occur there under any load circumstances, avoiding difficult tensile connections. In fact, the connection at the ends of the steel rod is achieved by simply predrilling a hole through the chords and anchoring the rod at its end side with a sufficiently large plate and a nut. A small pre-tension is normally applied to the rod. *Figure 8.19* shows a possible node solution.

Truss nodes at ridge and support

Figure 8.20 shows two types of possible node solution for ridge connections of pitched trusses. Joint type a) can be produced by means of gusset plates made either of plywood, LVL (with cross-laminated veneer) or nail plates of steel. This solution is normally suitable for moderate loads. Solution b) includes slotted-in steel plates and dowels. For large spans and/or heavy loads, several slotted-in plates can be used to increase the strength of the node.

Figure 8.21 shows two types of possible node solution for support connections in trusses. Both solutions shown in this figure include slotted-in plates and dowels. Solution a) shows a node for single-member trusses whereas solution b) shows a node for double-member trusses.

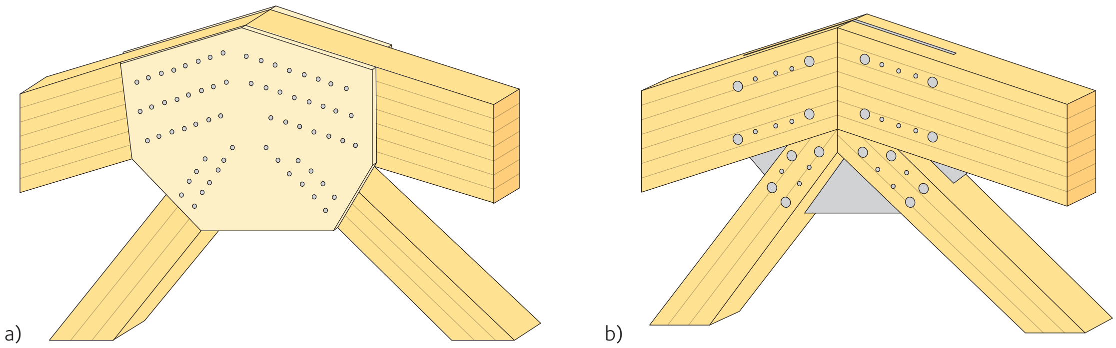


Figure 8.20 **Nodes at the ridge of pitched trusses**
a) With gusset plates of plywood, LVL, or nail plates,
b) with slotted-in plate and dowels.

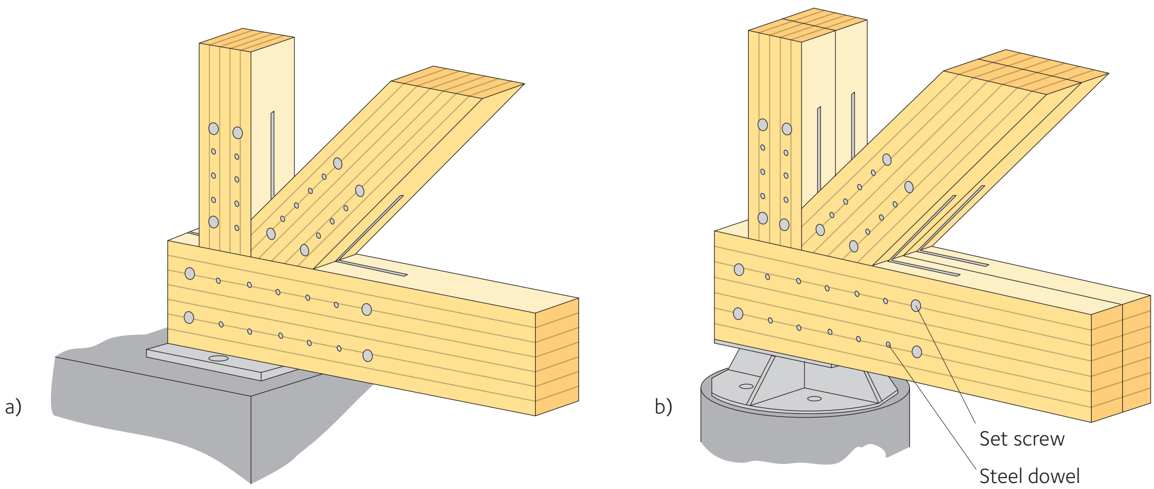


Figure 8.21 **Two examples of nodes at the support of trusses using steel plates and dowels**
a) Single-member truss,
b) double-member truss.

Three-hinged trusses

Three-hinged trusses (or three-pin trusses) are normally designed with two inclined rafters forming the slope of the roof. If the three-hinged truss is placed on stiff ground supports, the horizontal thrust is taken directly by the supports, *see figure 9.1 b)*. If the truss is supported on walls or columns, a tie is normally required to take the horizontal thrust at the support points, *see figure 9.1 a)*.

Three-hinged trusses are normally used for structures with a large roof slope and for spans where ordinary glulam beams are insufficient. Typically, the secondary structure of the roof consists of either purlins or a load-bearing corrugated plate that normally has the trusses at 6 – 8 m centres. The roof slope for three-hinged trusses that rest on column or walls should preferably exceed 14° ($f/l \geq 1/8$) in order to limit the deflection at the ridge and also to limit horizontal displacements at the supports. If the truss is placed directly on ground supports, the slope of the roof should preferably exceed 30° ($f/l \geq 1/3$), mainly to increase the free space under the roof. The span should be in the range of 15 – 50 m.

For larger spans, more than 50 m, the rafters of the three-hinged truss can be reinforced by a system of struts (usually made of timber) and steel ties, *see figure 9.2 a)*, or by creating a secondary underlying trussed structure, *see figure 9.2 b)*.

In the following sections, only three-hinged trusses with tension tie will be discussed.

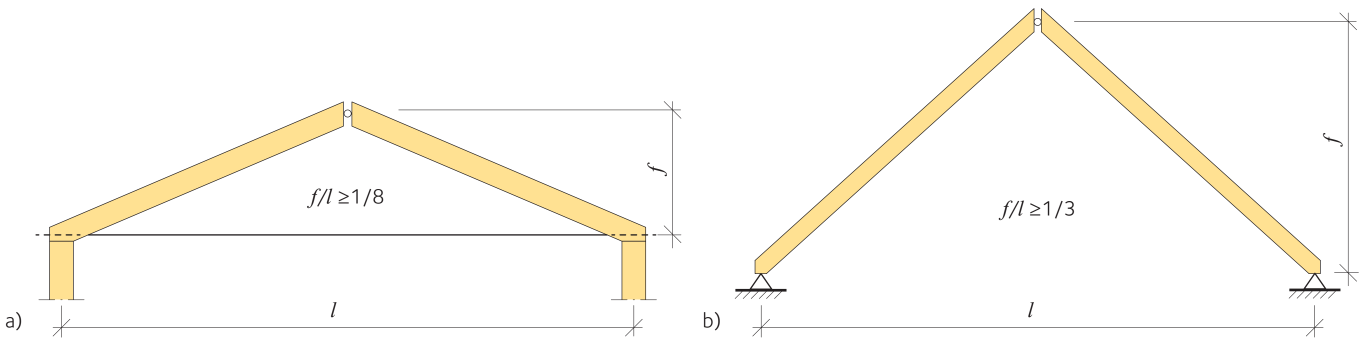


Figure 9.1 **Three-hinged trusses**
a) on columns,
b) on ground supports.

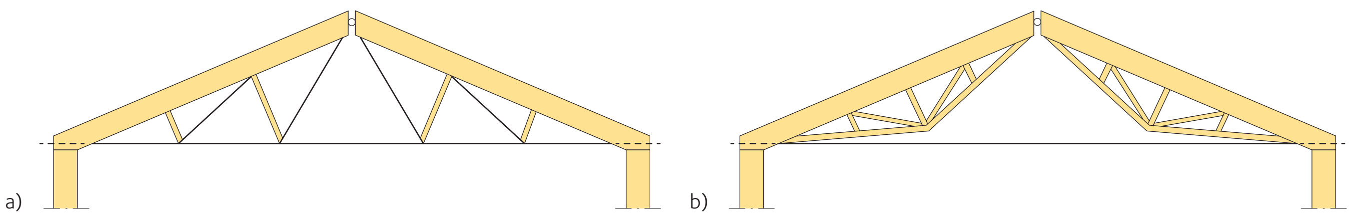


Figure 9.2 **Three-hinged trusses with reinforced ties**
a) with timber struts and steel ties,
b) with underlying timber truss (trussed rafter).

Riding school, Gävle, Sweden.

9.1 Three-hinged truss with simple (unreinforced) rafters

The glulam rafters are designed as beams subjected to compression and bending in accordance with *Chapter 4, page 53*, thus taking into account the risk of buckling. In general, out-of-plane buckling of the rafters is sufficiently restrained by purlins, corrugated steel plate or other secondary structures, which in this case must also be designed for bracing forces, *see Chapter 13, page 170*. In buckling perpendicular to the plane of the roof, the buckling length l_c is:

9.1 $$l_c = \frac{l}{2 \times \cos\alpha}$$

where l is the span of the truss and α is the slope of the roof. At the ridge and supports, shear stresses and local compression must be checked, taking into account that the forces act at an angle to the grain.

At spans up to 30 m the tension tie can be made of glulam. However, for such a case, the anchorage between glulam tie and rafters may become rather cumbersome; in particular, when the horizontal thrust is large, a great number of mechanical connectors (normally dowels or bolts) are needed to transmit the force between the rafters and the glulam tie, *see also section 14.8, page 226*. Moreover, fasteners can significantly reduce the effective timber cross-section (and thus the load carrying capacity) at critical points. Therefore, steel ties are in general preferred, due to the fact that the anchorage between the tie and the rafter is relatively easy to install. In heated spaces (over +5 °C) high strength steel can be used. Often, however, the benefit of using high strength steel is limited by the fact that it is the horizontal deformation of the tension tie that governs the design of the three-hinged truss.

When designing the connection with the glulam rafters, account must be taken of the fact that the forces act at an angle to the grain.

In most cases the ridge joint is in most cases designed as a hinge, i.e. free of moments, and it is dimensioned for maximum horizontal compression, which has the same magnitude (and obviously opposite sign) as the tension force in the tension tie. In unbalanced load cases, i.e. where the external load differs between the two halves of the roof, a shear force in the ridge will also occur.

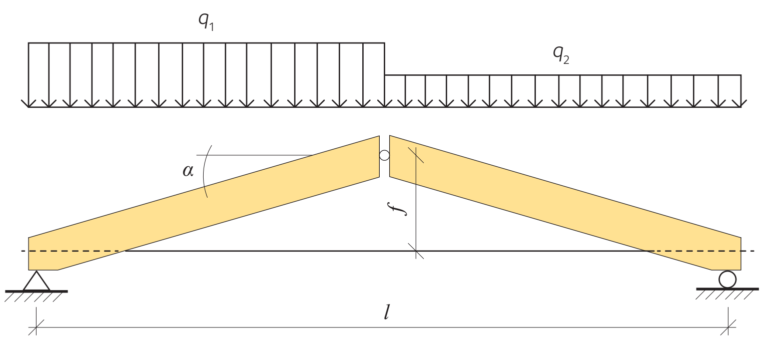


Figure 9.3 **Three-hinged truss with two rafters and tension member (tie) subjected to gravity loads**

9.1.1 Internal moments and forces and support reactions

For uniformly distributed, gravity loading and with symbols as in *figure 9.3, page 132*, the reactions at supports, and the internal moments and forces, can be calculated using the following expressions:

Vertical support reactions:

$$R_1 = \frac{(3 \times q_1 + q_2) \times l}{8} \tag{9.2}$$

$$R_2 = \frac{(q_1 + 3 \times q_2) \times l}{8} \tag{9.3}$$

Force in the tension tie:

$$H = \frac{(q_1 + q_2) \times l^2}{16 \times f} \tag{9.4}$$

Maximum moment M and corresponding normal force N in rafter:

$$M = \frac{q_1 \times l^2}{32} \tag{9.5}$$

$$N = \frac{(q_1 + q_2) \times l}{8 \times \sin \alpha} \tag{9.6}$$

Maximum shear force in rafter:

$$V_1 = \frac{q_1 \times l}{4} \times \cos \alpha \tag{9.7}$$

Shear force in ridge (vertical):

$$V_2 = \frac{(q_1 - q_2) \times l}{8} \tag{9.8}$$

9.1.2 Deformation

Under gravity loads, with load condition as in *figure 9.3, page 132*, the vertical deflection of the ridge can be calculated using the expression:

$$w = \frac{(q_1 + q_2) \times l^2}{16 \times \tan^2\alpha \times (E \times A)_{\text{rafter}}} \times \left(\frac{1}{\cos^3\alpha} + \frac{(E \times A)_{\text{rafter}}}{(E \times A)_{\text{tension tie}}} \right) \tag{9.9}$$

where:

$(E \times A)_{\text{rafter}}$ is the modulus of elasticity and area of the rafter.

and:

$(E \times A)_{\text{tension tie}}$ is the modulus of elasticity and area of the tension tie.

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

Mountain station, Idre, Sweden.

Sahlgrenska Hospital, Gothenburg, Sweden.

9.1.3 Safety against lifting

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

9.10
$$\sum_i H_i \geq 0$$

where H_i are the axial forces in the tension tie generated by different loads, e.g. self-weight and wind, taken with positive sign in the case of tension and vice versa for the case of compression.

By developing the condition of *equation 9.10*, a general expression can be derived for the required self-weight, g, of three-hinged trusses with tension tie and typical wind load distribution, *see figure 9.4*. Under uniformly distributed wind suction loads of different magnitude at the two roof halves, $q_{w,1}$ and $q_{w,2}$, and uniformly distributed self-weight, g, as in *figure 9.4*, *equation 9.10* can be rewritten as follows:

9.11
$$-\left[\frac{q_{w,1}+q_{w,2}}{2} \times \frac{l^2}{8 \times f} \times \left(1-\tan^2\alpha\right)\right] + \left[\frac{g \times l^2}{8 \times f}\right] \geq 0$$

where the square brackets contain the axial forces in the tension tie generated by the wind load (left bracket) and by the self-weight of the roof (right bracket).

If the condition of *equation 9.11* is not fulfilled, the inward horizontal thrust at the support must be dealt with, e.g. by means of a "tension tie" which can also take compression forces.

Note that columns clamped at the base would in general not be suitable to take such a horizontal reaction, due to their limited bending stiffness.

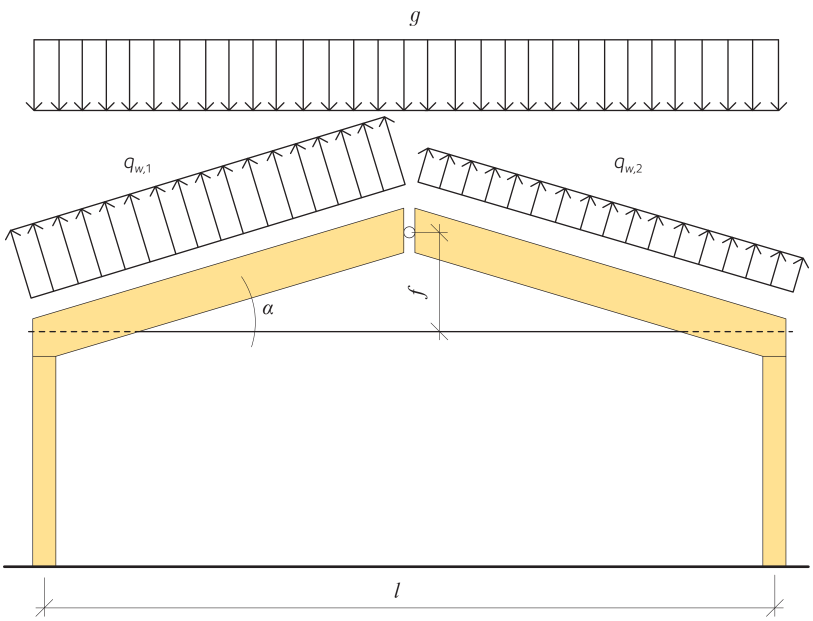


Figure 9.4 Three-hinged truss subjected to self-weight, g, and wind suction, $q_{w,1}$ and $q_{w,2}$

9.2 Preliminary design of three-hinged frame with steel tension tie subjected to uniformly distributed load

General indications concerning the preliminary design of a three-hinged truss with steel tension tie will be given in this section. A typical structure of such a type, with indication of the principal geometric parameters is shown in *figure 9.5*.

During the preliminary dimensioning stage, the following variables are generally known:

- Uniformly distributed design load q_d (symmetric). With snow load, the safest approach is to take the leeward load as being evenly distributed over the whole roof.
- Roof slope α (normally ≥ 14°).
- Span l (normally ≤ 50 m).

Furthermore, all the material design strengths are known, since they depend on the choice of glulam strength class, e.g. GL30c.

The preliminary design will comprise estimation of the cross-sectional dimensions of the rafter and of the tension tie.

Riding school, Sätra, Sweden.

Rafter breadth, *b*

In order to reduce problems with lateral instability, especially during the erection stage, the breadth of the beam should be chosen in the range:

$$b \approx \frac{l}{200} - \frac{l}{170} \qquad \textbf{9.12}$$

Rafter depth, *h*

In general, the distance i between purlins is chosen so that out-of-plane buckling of the rafter will not occur (normally: $c \leq 2.4$ m). In such a case, and for structures subject to an evenly distributed design load, q_d in service class 1 or 2 and with shortest load duration in the medium term (i.e. $k_{mod} = 0.8$), the depth of the beam can be estimated by means of the following equation (dimensions in mm and forces in N):

$$\begin{cases} h \approx \frac{k_1}{2} + \frac{1}{2} \times \sqrt{k_1^{\,2} + 4 \times k_2} \\ \text{where:} \\ k_1 = \dfrac{q_d \times l}{54 \times b \times \sin\alpha} \\ k_2 = \dfrac{q_d \times l^2}{102 \times b} \end{cases} \qquad \textbf{9.13}$$

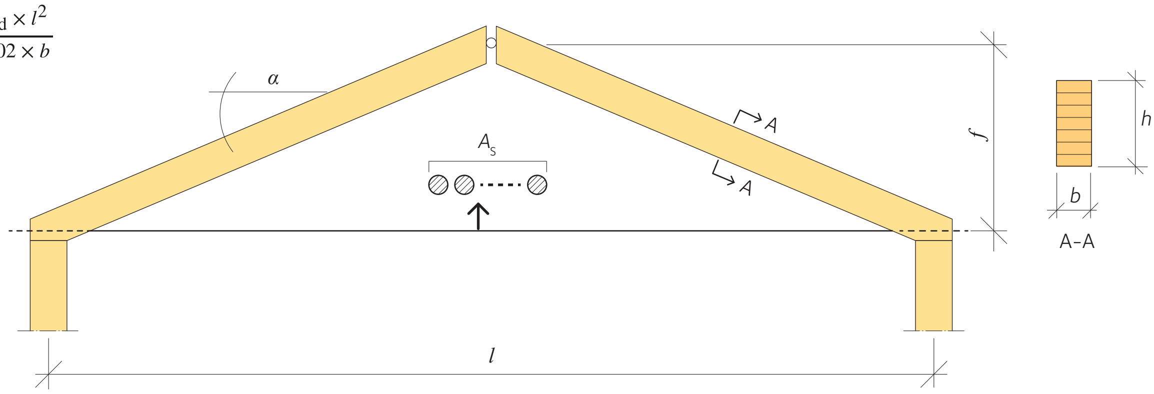


Figure 9.5 Typical three-hinged frame with steel tension tie

Wood for production of glulam.

Tension tie cross-sectional area, A_s
Generally, in order to limit horizontal deformations, steel grades with very high strength should also not be used. Steel types with too low tensile strength should also be avoided due to the difficulty of manufacturing threaded ends of good quality. It is preferable therefore to use steel types with yield strength, f_y, in the range 355 to 600 MPa. The required effective area (or nominal area) of the threaded rods can be estimated by means of the following expression:

9.14 $$A_s \approx \frac{q_d \times l^2}{8 \times f} \times \frac{1.4}{f_{ub}}$$

where f_{ub} is the ultimate tensile strength of the steel rod.

9.3 Serviceability check

For larger spans ($l \geq 25$ m), it is often the vertical displacement of the ridge at the SLS that governs the design. In such cases, therefore, the cross-section of the tension tie often becomes oversized from the point of view of its ultimate strength. As an indication, the vertical displacement of the ridge should be within the range $l/500 - l/700$ for serviceability loads with instantaneous duration.

9.4 Details

9.4.1 Anchorage of tension tie to rafter

The tension tie anchorage only transfers the horizontal tension force of the tie to the rafter. Usually, the tie is made of steel or glulam, *cf. section 14.8, page 226*. The anchorage is normally arranged so that the tension force is acting as near as possible to the intersection of the system lines of the beam and the column.

Steel ties are suitable both for small and large tension forces. For moderate loads, a single steel tension tie pulled through a central hole in the rafter and fixed against the end face of the rafter by means of an anchor plate can be used, *see figure 9.6 a), page 137*. From a practical point of view, it is easier to only use surface-mounted tension ties. If the tension forces are large, however, two ties on either side, possibly complemented by a third tie, centrally placed, should be used. In this case, it is advisable to use a complementary steel profile, e.g. a UPE-profile, positioned between the anchor plate and the timber in order to distribute the high compression load onto a larger area, *see figure 9.6 b), page 137*. The steel plate acting against the end of the rafter should have nail holes to simplify erection.

For moderate tension forces, glulam ties can also be adopted. If a single tension tie is used, the anchorage of the tie in the rafter can be performed with e.g. a steel plate, which is either folded around the end of the rafter, *see figure 9.7 a), page 137*, or ends a little way in from the beam end. Alternatively, slotted-in plates and dowels may be used to connect the rafter and the tension tie, *see figure 9.7 b), page 137*. If a double tension tie is used, the anchorage can be carried out by means of dowels, bolts or screws, *see figure 9.7 c), page 137. See also section 14.8, page 226.*

Note that an unprotected steel tie does not as a rule meet the requirements for fire resistance, e.g. R30, *see Chapter 15, page 236*. Protection is most easily provided by applying preformed mineral wool modules. If visual standards are high, the mineral wool can be enclosed in plastic tubes. Note also that the increase in length due to the rise in temperature is considerable, despite the insulation, and that the supports must be designed to take this into account.

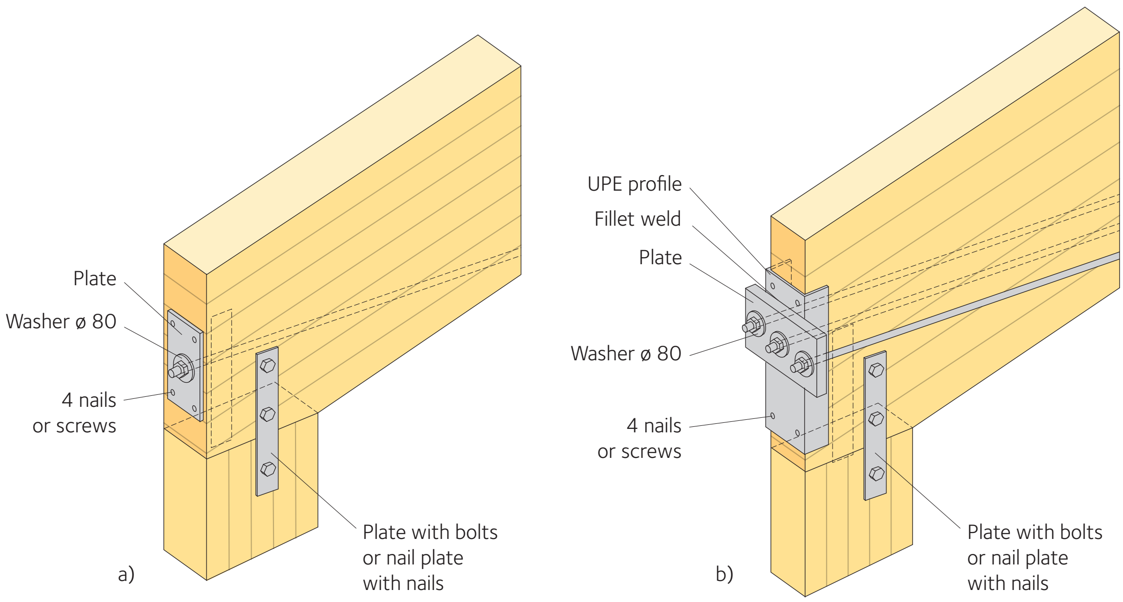


Figure 9.6 **Anchoring of steel tension tie to the rafter**
a) Detail suitable for moderate tension forces,
b) Detail suitable for large tension forces. *Cf. Section 14.8, page 226.*

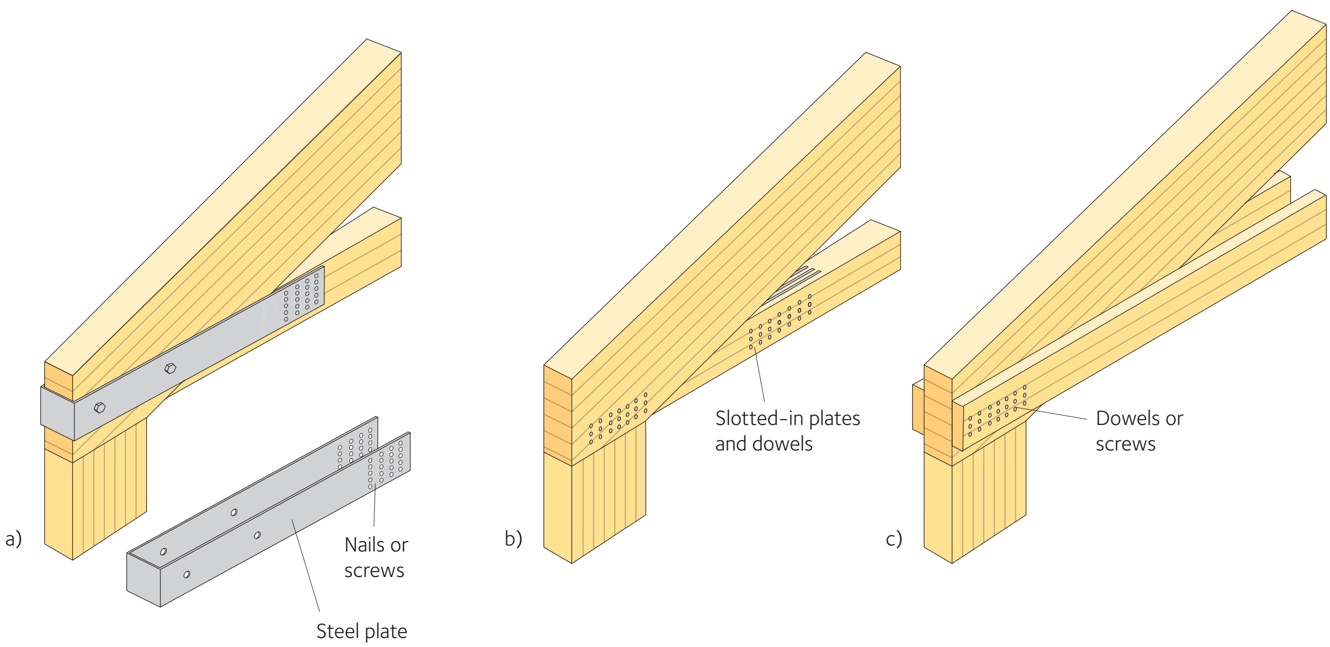


Figure 9.7 **Anchoring of glulam tension tie to the rafter**
a) Connection with single tension tie and steel plate which is taken around the end of the rafter,
b) Connection with slotted-in plates and dowels,
c) Connection with double tension tie.

9.4.2 Hinged ridge joint

Hinged ridge joints transfer horizontal and vertical forces, *see section 14.7, page 224*. 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 such a movement cannot take place, additional stresses will arise, which can lead to unforeseen damage to the structure, usually splits in the wood.

The connection normally consists of external fishplates made of nail plate, *see figure 9.8 a)*. These can be combined with a slotted-in cruciform steel connector, when shear forces at the ridge are large, *see figure 9.8 b)*.

For large shear forces at the ridge, steel profiles, e.g. IPE-type, can also be used for the hinged ridge connection, *see figure 9.9*. Typically, the width of the steel profile, *a*, is chosen as large as the width of the rafter *b* or slightly smaller. The length of the steel profile, *c*, depends upon the shear force that has to be transmitted, and will normally be within the range 200 mm to 350 mm, *see figure 9.9*.

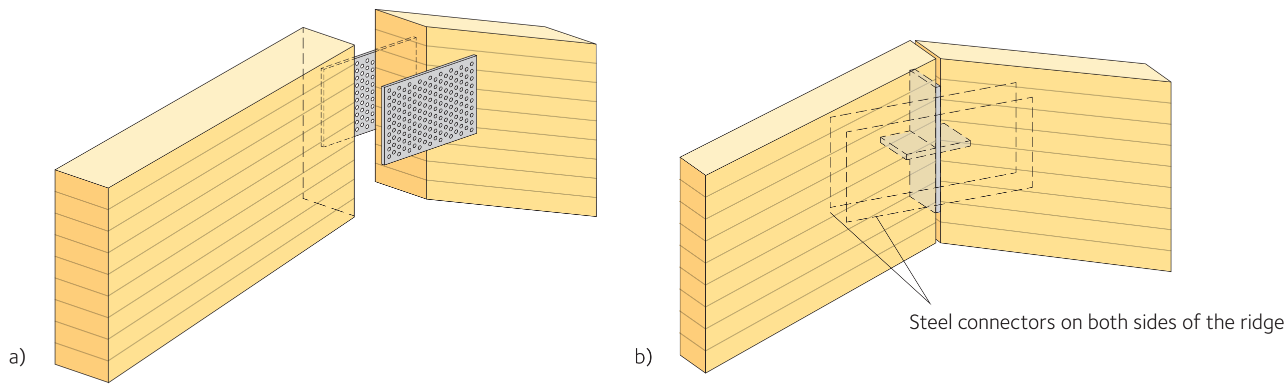


Figure 9.8 Hinged ridge joint
a) for moderate shear force at the ridge,
b) for large shear force at the ridge.

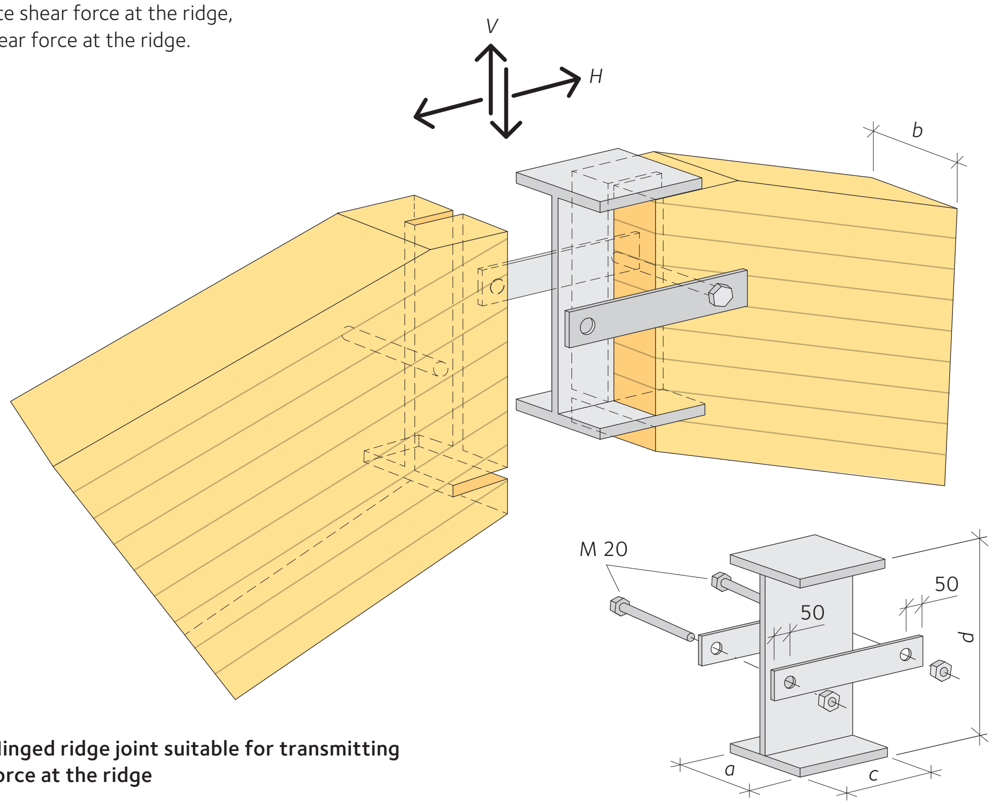


Figure 9.9 Hinged ridge joint suitable for transmitting large shear force at the ridge

9.4.3 Connection between rafter and steel tension tie

Tension ties can be relatively long. In order to limit excessive deflection and bending moments due to the self-weight of the steel rod, suspenders are normally used. Suspenders typically consist of a steel rod with a very small diameter, typically d = 5 mm, placed with spacing of approximately 10 m. A typical set-up for connection between the suspender and the rafter and tension tie is shown in *figure 9.10*.

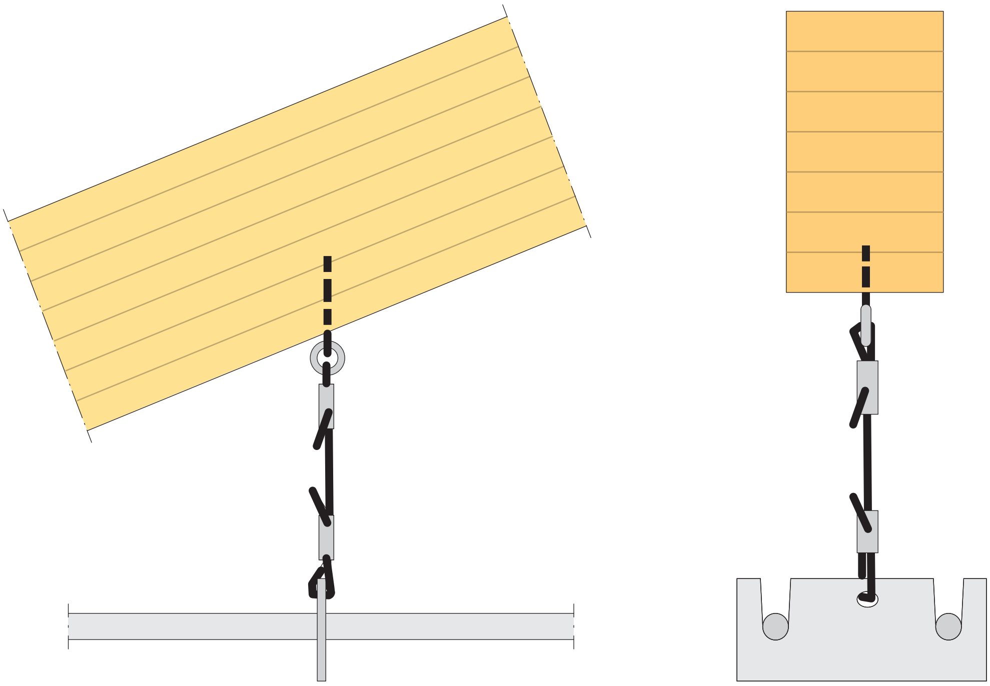


Figure 9.10 Typical set-up for connection between the suspender and the rafter and tension tie

Portal frames

Nowadays timber frame structures are almost always executed in glulam. The haunch is typically made curved with continuous lamellas, finger-jointed, or built-up, *see figure 10.1*. The form of the frame should follow the funicular of the main load combination, as far as functional and aesthetic considerations permit. Curved or built-up haunches fulfil this requirement most easily and are therefore the forms best suited for large spans.

The roof slope should be no less than 14° due to, among other reasons, the wish to reduce the deflection of the ridge.

The three-hinged portal frame is suitable for spans up to 30 – 40 m. For transportability reasons, the connecting line between ridge and foot should 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 3.7 m.

Although the two-hinged portal frame provides a stiffer structure, this type of frame must generally be manufactured and transported in three or more parts, which are jointed with rigid joints on the site. Joints should be placed at positions in the structure with small moments. Rigid joints demand more complicated workmanship than hinged joints and therefore will have a higher price. They are also often highly visible in an undesirable way. On the other hand, the elements of the frame are smaller than those in a corresponding three-hinged frame and therefore easier to transport.

Hinge-less frames are not usually used for timber structures.

The three-hinged portal frame is by far the most common type. Therefore, only indications concerning the design of such types of frames will be given in the next sections.

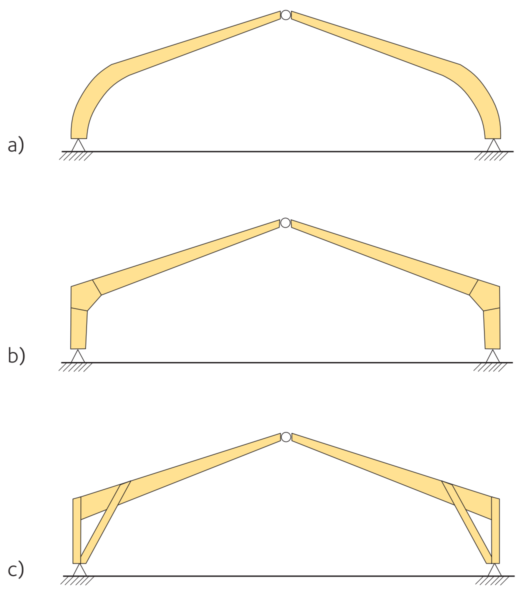


Figure 10.1 Examples of three-hinged portal frames
a) Frame with curved haunches,
b) Frame with finger-jointed haunches,
c) Built-up frame (knee-braced frame).

10.1 Design of three-hinged portal frame

The three-hinged portal frame is stable against horizontal forces in its own plane and statically determinate, which means that the moment distribution is not affected by uneven settlement of the foundations or by unforeseen deformations in joints and connections. Furthermore, the three-hinged frame is hinged at the foundations, which simplifies its basic construction. For poor soil conditions, the horizontal reactions at the supports can be taken up by tension members between the foundations (within or under the slab).

In normal cases, with roof slopes around 15°, the governing load combination consists of self-weight and snow plus possible concentrated loads from overhead cranes and similar. However, in connection with steep roof slopes, e.g. in churches or certain types of storage buildings, load combinations with wind as the main load can be critical.

10.2 Internal forces and support reactions

The three-hinged 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 be carried out with the aid of a computer programme.

Under uniformly distributed, unsymmetrical loading, *cf. figure 10.2*, the reaction and internal forces acting in a three-hinged frame can be calculated using the following expressions.

Note that *equations 10.4* and *10.5* apply for a pitch angle α = 14°. For larger pitch angles, these equations give results on the safe side.

Vertical support reactions:

$$R_A = \frac{(3 \times q_1 + q_2) \times l}{8} \quad (10.1)$$

$$R_C = \frac{(q_1 + 3 \times q_2) \times l}{8} \quad (10.2)$$

Horizontal thrust:

$$H = \frac{(q_1 + q_2) \times l^2}{16 \times f} \quad (10.3)$$

Maximum normal force at the haunch:

$$N = 0.79 \times (R_c - 0.38 q_2 r) + 0.62 \times H \quad (10.4)$$

Maximum moment at haunch:

$$M = (a + 0.62r) \times H - 0.21r \times (R_c - 0.38 q_2 \times r) \quad (10.5)$$

Shear force at ridge (vertical):

$$V_B = \frac{(q_1 - q_2) \times l}{8} \quad (10.6)$$

Note that in the case of three-hinged frames with vertical member at the support, e.g. frames with finger-jointed haunches, the bending moment at the haunch, *see equation 10.5 and figure 10.2*, simply becomes $M = H \times h'$.

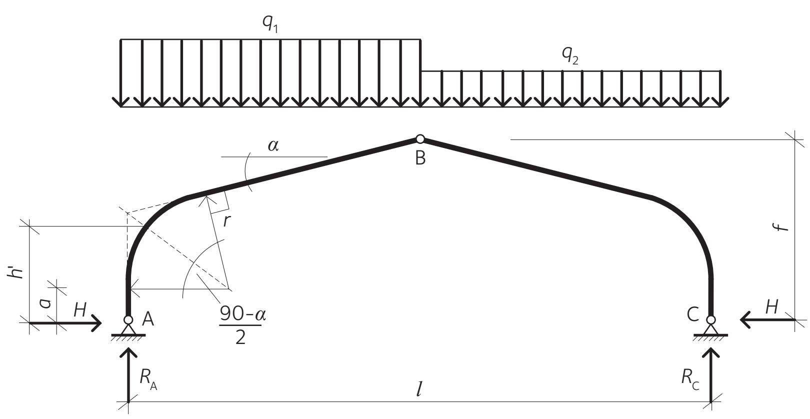


Figure 10.2 Three-hinged frame with curved haunches, symbols

Cow shed, Lyrestad, Sweden.

10.3 Design procedure

The design can be carried out as follows:

1. Determine the main sizes and the design values for the relevant loads and climatic conditions.
2. Sketch the approximate centre lines of the frame based on the following experience-based figure:

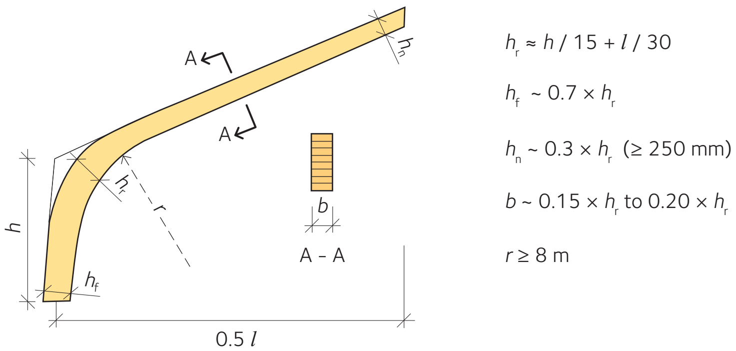


Figure 10.3 **Preliminary sizing of a glulam portal frame**

3. Calculate the support reactions and the 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 maximum normal force or the maximum 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 a) stability and b) detail design according to the instructions given in *section 10.4, page 143, and section 10.6, page 150*.
8. Check the frame rafter for compression and simultaneous bending in accordance with *section 10.4, page 143*. 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 positive moment occurs. It is generally sufficient to check at a couple of randomly selected cross-sections.
9. Check the Serviceability Limit State according to *section 10.5, page 148*.

10.4 Stability control

In frames, as in columns and beams, the buckling resistance about both the major axis and the minor axis (of the glulam cross-section) must be verified by stability checks. Because of the deformability of the connections and the generally non-ideal support conditions, only approximate assessments can be made of the buckling length.

10.4.1 Lateral buckling (buckling around the minor axis)

In portal frames, lateral buckling is checked in the same way as any beam-column between lateral restraints provided by bracings. **Note** that the buckling length depends on the chosen bracing system and also, to some extent, on the roof system. For example, if sufficiently rigid roof sheeting is used, e.g. adequate corrugated plate directly screwed on the top of the portal frames, no out-of-plane buckling will occur. If stiff sheeting is screwed on the top of the purlins, lateral buckling will occur between the points where purlins are fixed to the frame rafters (buckling length: a_1, *see figure 10.4*). However, if the sheeting is not sufficiently stiff, as is the case of, for example, sheets made of fibre cement or similar, the buckling length to be adopted in design should be the distance between the nodes of the wind truss (buckling length: a, *see figure 10.4*).

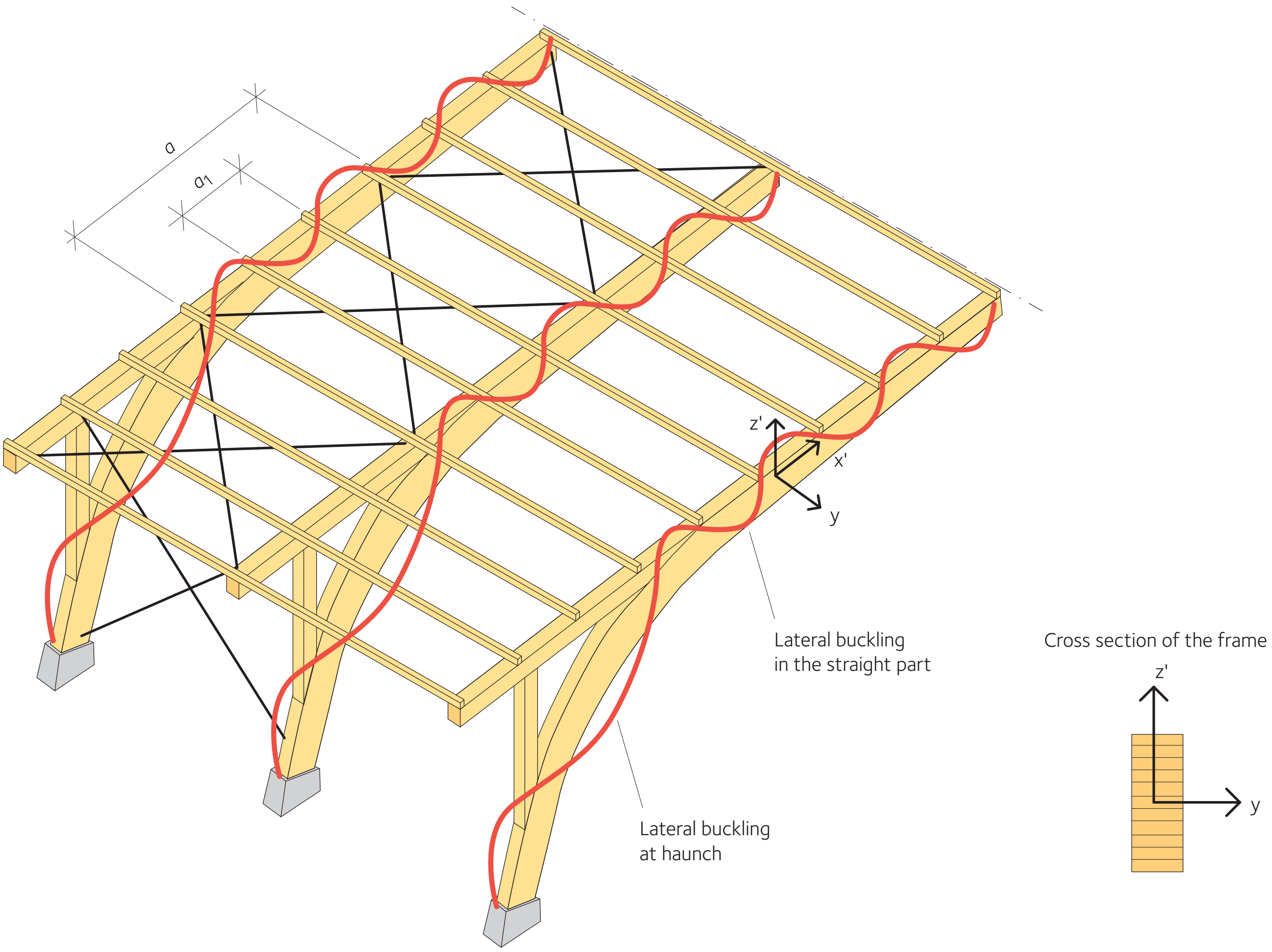


Figure 10.4 Lateral buckling of frame and of the haunch. The roof system consists of a rigid sheeting, e.g. corrugated metal plate.

Swimming facility, Västervik, Sweden.

Out-of-plane buckling of straight parts

The out-of-plane buckling should be checked in the unbraced zones as for beam columns, according to the model shown in *figure 10.5*.

The design criterion is written as:

10.7
$$\begin{cases} \left(\dfrac{\sigma_{m,y,d}}{k_{crit} \times f_{m,y,d}}\right)^2 + \dfrac{\sigma_{c,0,d}}{k_{c,z'} \times f_{c,0,d}} \leq 1 \\ \dfrac{\sigma_{c,0,d}}{k_{c,z'} \times f_{c,0,d}} + k_m \times \dfrac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \end{cases}$$

where:

$\sigma_{c,0,d}$	is the design compression strength parallel to the grain.
$\sigma_{m,y,d}$	is the design bending stress about y.
$f_{c,0,d}$	is the design compression strength parallel to the grain.
$f_{m,y,d}$	is the design bending strength about y.
$k_{c,z'}$	is the reduction factor for out-of-plane buckling (i.e. buckling about z'-axis, *see figure 10.4, page 143*).
k_{crit}	is the reduction factor for lateral-torsional buckling, *see section 4.1.3, page 55*.
k_m	is a factor that accounts for the redistribution of stresses and the effect of inhomogeneities in the material's cross-section. It assumes a value of 0.7 for a rectangular cross-section.

For more detailed information concerning the symbols used, *also see Chapter 4, page 53*.

Out-of-plane buckling of curved parts

Normally, portal frames are not braced in the haunch zone. Due to gravity load, negative bending moments occur at the haunch, which cause compression at the underside of the cross-section. Moreover, compression stresses due to normal force are also present in this zone.

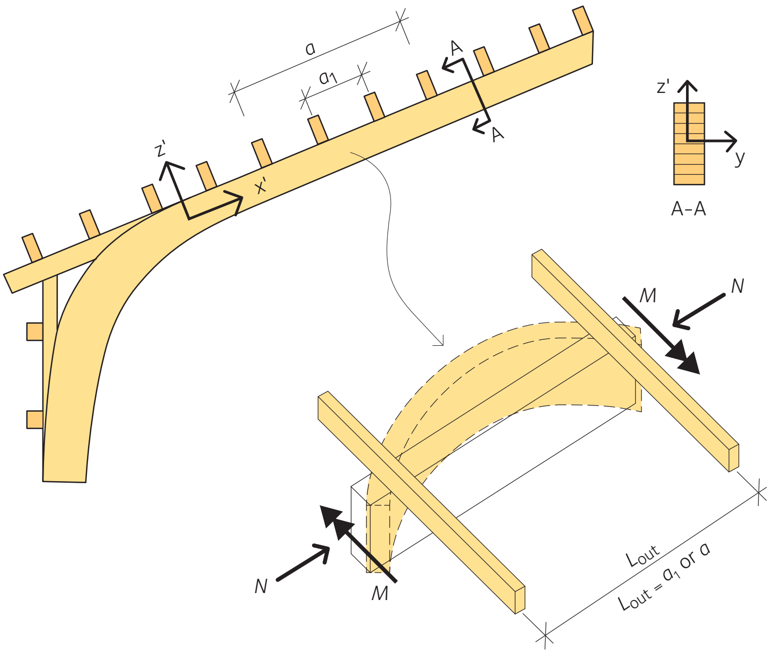


Figure 10.5 Model for out-of-plane buckling check of the straight part of the frame. If the roof material is sufficiently stiff, the buckling length can be assumed to be equal to a_1. Otherwise, the buckling length should be assumed equal to a, *see also figure 10.4, page 143.*

Therefore, the haunch should be checked for combined action of compression and bending. The design conditions to be fulfilled are similar to those of the straight part, *see equation 10.7*, with the exception of the introduction of the coefficient k_r, *see Chapter 7, page 102*, which takes into account the strength reduction due to bending of the lamellas during production.

$$\begin{cases} \left(\dfrac{\sigma_{m,y,d}}{k_{crit} \times k_r \times f_{m,y,d}}\right)^2 + \dfrac{\sigma_{c,0,d}}{k_{c,z'} \times f_{c,0,d}} \leq 1 \\ \dfrac{\sigma_{c,0,d}}{k_{c,z'} \times f_{c,0,d}} + k_m \times \dfrac{\sigma_{m,y,d}}{k_r \times f_{m,y,d}} \leq 1 \end{cases} \tag{10.8}$$

For the calculation of the critical bending moment M_{crit} to be used for the determination of the corresponding slenderness ratio and finally the reduction factor k_{crit}, *cf. section 4.1.3, page 55*, the following formula should be used:

$$M_{crit} = \frac{\pi}{s_0} \times \sqrt{(E \times I_z) \times (G \times k_v)} + \frac{(E \times I_z) + (G \times k_v)}{2 \times r} \tag{10.9}$$

where:

$$I_z = \frac{b^3 \times h}{12}$$

and:

$$k_v = \frac{b^3 \times h}{3} \times \left(1 - 0.63 \times \frac{b}{h}\right)$$

where k_v is the cross-sectional factor of the torsional stiffness.

Note that for $r \rightarrow \infty$, *equation 10.9* becomes identical to the equation for the determination of the critical moment of straight members subjected to constant bending moment, *see Chapter 4, page 53*.

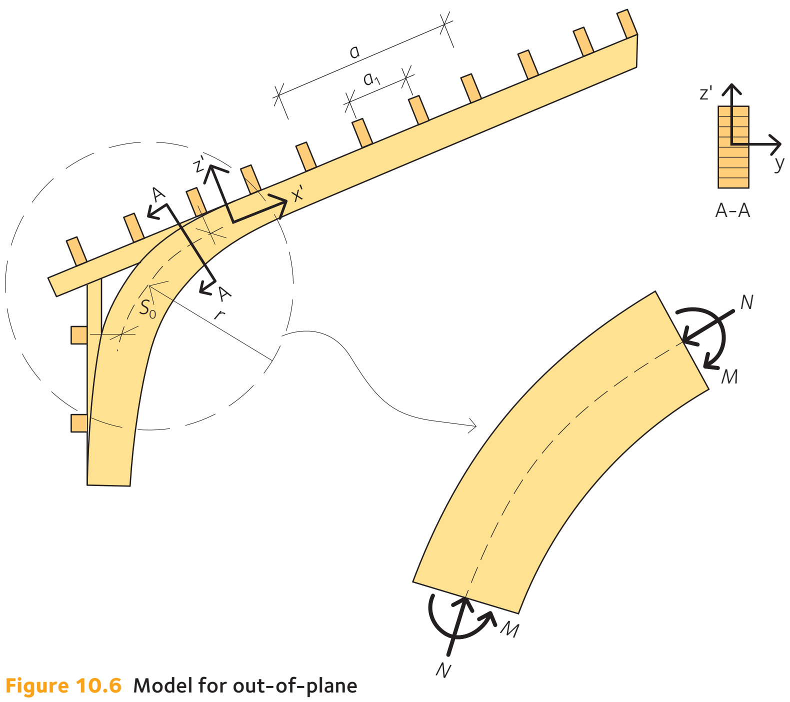


Figure 10.6 Model for out-of-plane buckling check of the haunch

10.4.2 In-plane buckling (buckling around the major axis)

Buckling in the plane of the frame is in general more complicated than for normal beam-column members. The common method of analysis for frames and arches is performed in two steps:
1. Linear buckling analysis.
2. Second order analysis.

Linear buckling analysis

If linear buckling analysis is adopted, frames and arches can be verified in the same manner as beam-columns, i.e. members subjected to simultaneous action of bending and compression. The calculation of stresses due to external loading is based on linear elastic theory considering the equilibrium of the undeformed static system. Stresses caused by in-plane and lateral imperfections and induced deflections are taken into account by multiplying the compression and bending strength values by buckling reduction factors k_c and k_{crit}. Here k_c is the reduction factor for buckling under axial compression. For common rise-span ratios the first buckling mode of a portal frame is normally asymmetric and has a shape similar to that shown in *figure 10.7*.

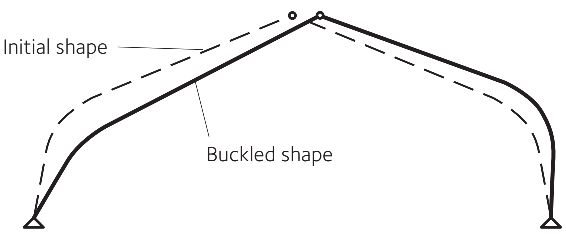


Figure 10.7 In-plane buckling of a three-hinged portal frame

For the determination of the buckling load according to the simplified analysis, the buckling length L_e is required. Such a length can be determined either a) numerically, e.g. by means of a linear buckling analysis performed by means of a finite element programme or b) by empirical formulas.

As a first approximation, the buckling length of the frame can be assumed as:

10.10 $$L_e \approx 1.25 \times a_{hf}$$

where a_{hf} is the length of the half-portal, *see figure 10.8*.

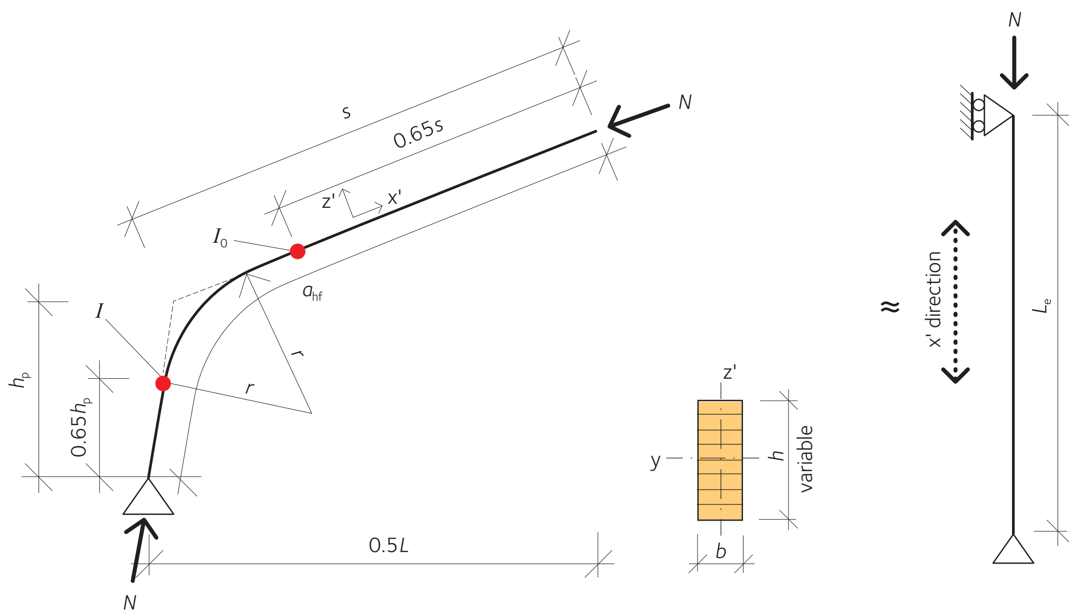


Figure 10.8 Model for the in-plane buckling length of a three-hinged portal frame

For a more refined analysis, the following formula offered by the German building code for timber structures, DIN EN 1995-1-1/NA, can be adopted:

$$L_e \approx h_p \times \sqrt{4 + \frac{I_0}{I} \times \frac{s}{h} \times \left(\pi^2 + \frac{s}{h_p}\right)} \quad 10.11$$

The moments of inertia I and I_o are determined at the distances $0.65 \times h$ and $0.65 \times s$ from the foot and the ridge respectively, *see figure 10.8, page 146*.

The design criterion is written as:

$$\frac{\sigma_{m,y,d}}{k_r \times f_{m,y,d}} + \frac{\sigma_{c,0,d}}{k_{c,y} \times f_{c,0,d}} \leq 1 \quad 10.12$$

where $k_{c,y}$ is the reduction factor for in-plane buckling (i.e. buckling about the y-axis, *see figure 10.8, page 146*) and with the other symbols explained in *equation 10.7, page 144* and in *Chapter 4, page 53* and *Chapter 7, page 102*.

Machine shed, Söderköping, Sweden.

Second order analysis

As the load increases in a compressed structure, the deformations also increase, and these generate a larger bending moment, which in turn produces larger deformations. If the load continues to increase, this process will eventually lead to the stress exceeding the material's strength somewhere in the structure, thus causing a failure.
A geometric non-linear calculation takes account of the way the moment (caused by the additional deformations) increases with the rise in the axial load. If the non-linear calculation also takes account of the structure's initial imperfections, the result will give the "correct" sectional forces. These sectional forces can be used directly in the design process, without the need to adjust them with any reduction factors for buckling (i.e. k factors). The design values are thus achieved by checking the cross-section for simultaneous compression and buckling without reference to the risk of buckling (namely by setting the reduction factor for buckling at $k_c = 1.0$).

This way of designing a timber structure should not be considered without the use of suitable finite element software, which is now widely available. The way that the geometric imperfections should be dealt with depends, to some extent, on the modelling options offered by the software in question. The lowest buckling mode is usually a good approximation for the form of the initial imperfection, *see figure 10.9*. An alternative is to use the deformations obtained from a linear elastic analysis using the relevant load case as input data for the initial imperfections in the structure. A typical value for the maximum amplitude of the imperfection when calculating glulam structures is around $L/400$ according to Eurocode 5 (*see figure 5.3 in SS-EN 1995-1-1*).

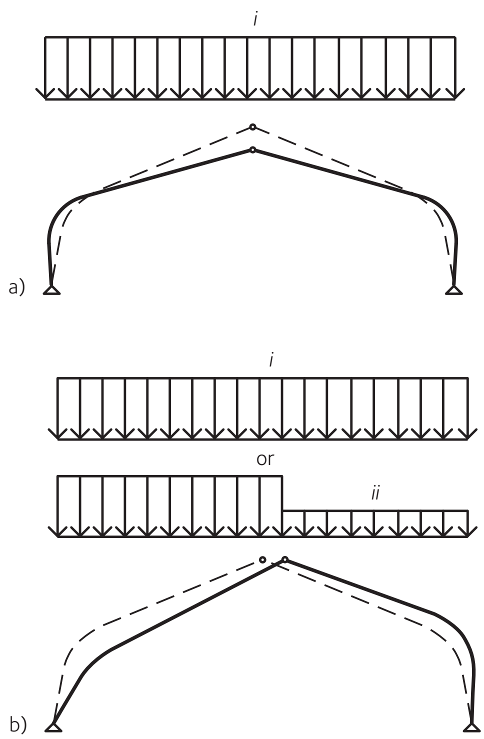


Figure 10.9 **Initial deviation in the geometry and corresponding loads to be adopted for second-order analyses**
a) Symmetric buckling,
b) asymmetric (or sway mode) buckling.

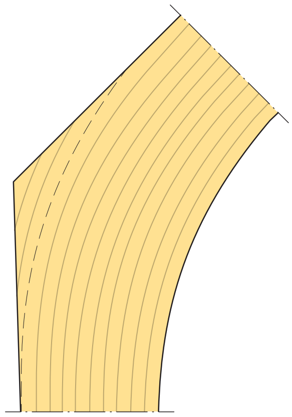


Figure 10.10 Curved haunch with fully glued corner piece

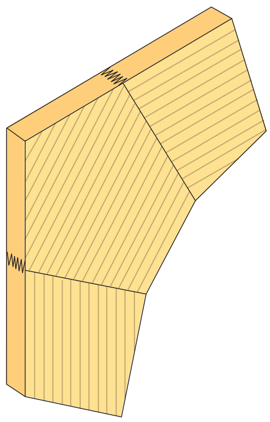


Figure 10.11 Finger-jointed haunch with jointing piece

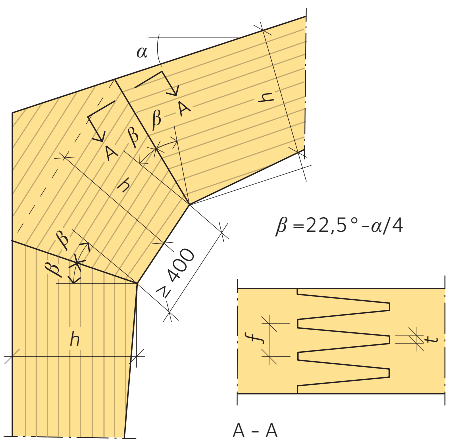


Figure 10.12 Finger-jointed haunch with jointing piece, symbols and geometry

Table 10.1 Reduction factor k_α as a function of the angle β.

Angle β	Reduction factor k_α
$\beta \leq 11.25°$	$k_\alpha = 0.33$
$11.25° < \beta \leq 18.75°$	$k_\alpha = 0.533\text{-}0.0178 \times \beta$
$18.75° < \beta \leq 22.5°$	$k_\alpha = 0.20$

10.5 Design of haunches

10.5.1 Curved haunches

Curved haunches are usually made with a constant cross-section and sometimes with the addition of an outer corner wedge as in *figure 10.10*. In order to allow for easy manufacturing, and also to avoid strength reduction, the ratio between the radius of curvature and the lamination thickness is normally relatively large, $r_{in}/t \approx 240$. Often, a lamination thickness t = 33 mm along with a radius of curvature r = 8 m are adopted for the manufacturing of portal frames with curved haunches. The corner wedge (outside the broken line in the figure) can then either co-act with the cross-section (composite action if the wedge is glued) or be simply nailed or screwed on. Radial tension stresses (tension perpendicular to the grain) must be checked if load combinations give positive moments (inner edge in tension) at the haunch – usually when wind load is the leading load in a load combination. The design process for glulam haunches is similar to the process for a pitched cambered beam, *see Chapter 7, page 102*.

10.5.2 Finger-jointed haunches

Finger-jointed haunches are usually manufactured with a jointing piece, *see figure 10.11*. The main function of the jointing piece is to reduce the angle between the axial force and the grain at the corner zone, which is favourable for the load-bearing capacity of the haunch. In normal cases, i.e. when the governing load combination generates negative moment at the haunch, the laminations of both frame rafter and frame leg should be chosen parallel with the upper side of the frame, *see figure 10.11*. The laminations of the jointing piece of the haunch are commonly always parallel with the underside of the frame.

The two joints at the haunch are checked according to the following semi-empirical method:

- The bending moment M_{joint} and the longitudinal force N_{joint} perpendicular to the joint is calculated at the centre of each joint.
- The effective cross-sectional area and effective section modulus at the joints are calculated. With symbols as in *figure 10.12*, the following expressions apply:

$$A_{joint} = \frac{b \times h}{\cos\beta} \times \left(1 - \frac{t}{f}\right) \qquad (10.13)$$

$$W_{joint} = \frac{b \times h^2}{6 \times \cos^2\beta} \times \left(1 - \frac{t}{f}\right) \qquad (10.14)$$

where:

t is the width of the finger tip.
f is the spacing of fingers centre to centre at base.

The ratio t/f is normally within the range 0.1 to 0.2. Therefore, if the geometry of the finger joint is not known a priori, the terms within the brackets in *equation 10.13* and *10.14* can be replaced by 0.8.

- The design value of the compressive strength $f_{c,\alpha}$ is determined from the angle β between longitudinal forces and the grain, *see Chapter 4, page 53*.
- For load combinations that produce compression at the inside edge of the haunch, the following critical condition applies:

$$\frac{N_{\text{joint}}}{A_{\text{joint}}} + \frac{M_{\text{joint}}}{W_{\text{joint}}} = f_{c,\alpha} \tag{10.15}$$

- For load combinations that produce tension at the inside edge of the haunch, usually in combinations with wind as leading load, the following design condition applies:

$$\frac{N_{\text{joint}}}{A_{\text{joint}}} + \frac{M_{\text{joint}}}{W_{\text{joint}}} = k_{\alpha} f_{c,\alpha} \tag{10.16}$$

where the coefficient k_{α} depends on the angle β between the longitudinal force and the direction of grain in the jointing piece, *see table 10.1*.

10.5.3 Haunches in built-up frames

Built-up haunches can be manufactured in many different ways. In knee-braced frames, as in *figure 10.13*, section A shall be checked for compression and simultaneous bending as in *section 10.5.2, page 148*, possibly with reduced design values of compression and bending strength due to lateral-torsional buckling and buckling in the plane of the frame. Further, shear stresses at section B are checked.

The outer frame leg is normally designed for axial loading, possibly with simultaneous moments due to wind load. In the case of predominant gravity loads, large tension forces in the outer frame leg will take place. These can be taken down to the foundations with the aid of a steel tension member as in *figure 10.13*, while moment and compression forces are taken up by a simple timber strut. The tension member should be fixed at the top of the frame rafter, rather than at its bottom, in order to minimize the risk of splitting.

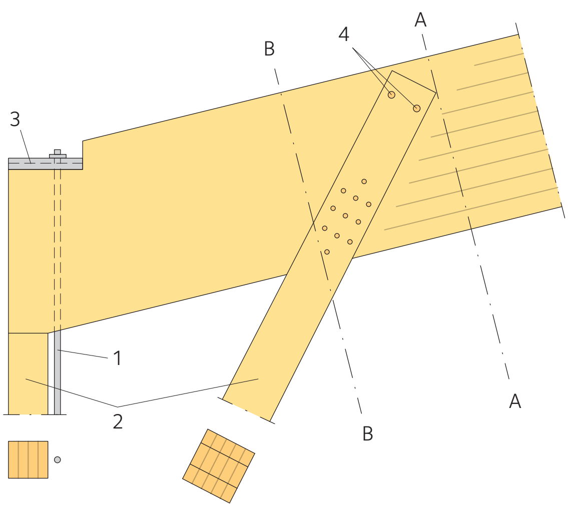
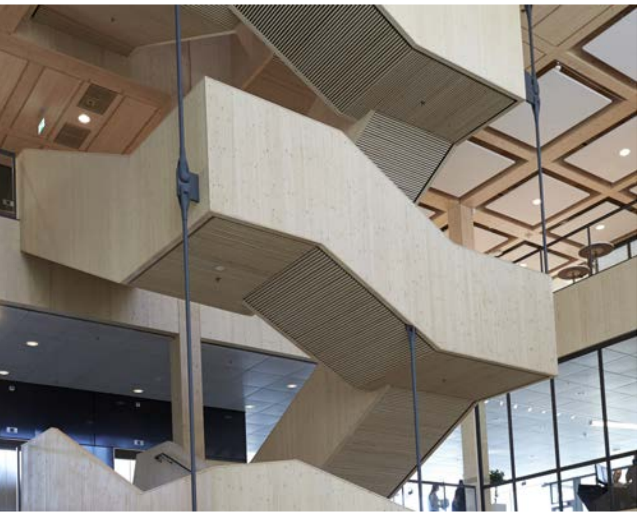


Figure 10.13 **Built-up haunch with bolted connection between rafter and inner leg**
1) Steel tension member,
2) Timber compression member,
3) Steel "U-profile",
4) Elongated bolt holes.



Ulls hus, Uppsala, Sweden.

The inner frame leg is designed as a column subjected to compression and possible bending, if the connection between the leg and the rafter is such that it causes eccentric loading. The connection between the frame rafter and the frame legs is often designed in such a way that the transfer of compression forces occurs mainly at the lower edge of the frame rafter, preferably through contact pressure. This can be achieved by means of, for example, a timber cleat glued to the underside of the rafter, as in *figure 10.14, page 151*. At the contact area there is, however, a risk of penetration of the contact surfaces, in roughly the same way as when two scrubbing brushes are pressed together. In order to prevent this, it can be suitable to manufacture the connection with an insert of, for example, a steel sheet or similar. One or more full-threaded screws can also be inserted perpendicularly to the joint line between the cleat and the rafter to take possible tension forces, due to, for example, some probable misfit of the contact surfaces.

The connection can be checked as follows:

- Check the contact pressure $\sigma_{c,\alpha,d}$ between the frame leg and the glulam cleat:

10.17 $$\sigma_{c,\alpha,d} = \frac{N}{b \times a} \times \cos\beta \leq f_{c,\alpha,d}$$

where $f_{c,\alpha,d}$ is the compression strength at an angle α to the grain, determined as in *Chapter 4, section 4.1.5, page 61*, considering $\alpha = \beta$ as the angle between the direction of the force and the grain direction. b is the width of the frame leg, in the out-of-plane direction.

- Check the shear stress between the cleat and the frame rafter:

10.18 $$\tau_d = \frac{N}{b \times s} \times \cos\beta \leq 0.5 \times f_{v,d}$$

where $f_{v,d}$ is the shear strength of glulam (in *equation 10.18* $f_{v,d}$ is reduced by a factor 0.5 in accordance with experimental results). Further, it must be checked that the length of the cleat s shall be at least 200 mm and the ratio between the cleat length and its depth shall be at least 6 (i.e. $s/a \geq 6$). Length in excess of $s = 8 \times a$ cannot be utilised. This means that in case $s \geq 8 \times a$, the input value for s in *equation 10.18* shall be $8 \times a$.

10.6 Deformation control

Normally, members of three-hinged portal frames are not precambered. However, in some circumstances, e.g. where deflections due to variations in moisture content are expected, pre-camber may be necessary.

Normally, the serviceability check is performed by calculating the deflection at the ridge. That calculation may be carried out most simply by means of some of the reliable finite-element programmes for frame analysis, that exist on the market.

There are no recommendations given in the building code concerning deflection limits, because such limits are related to the intended use of the structure. However, the final deflection due to self-weight and relevant variable load including creep effect should not, under any circumstances, exceed 1/200 of the span.

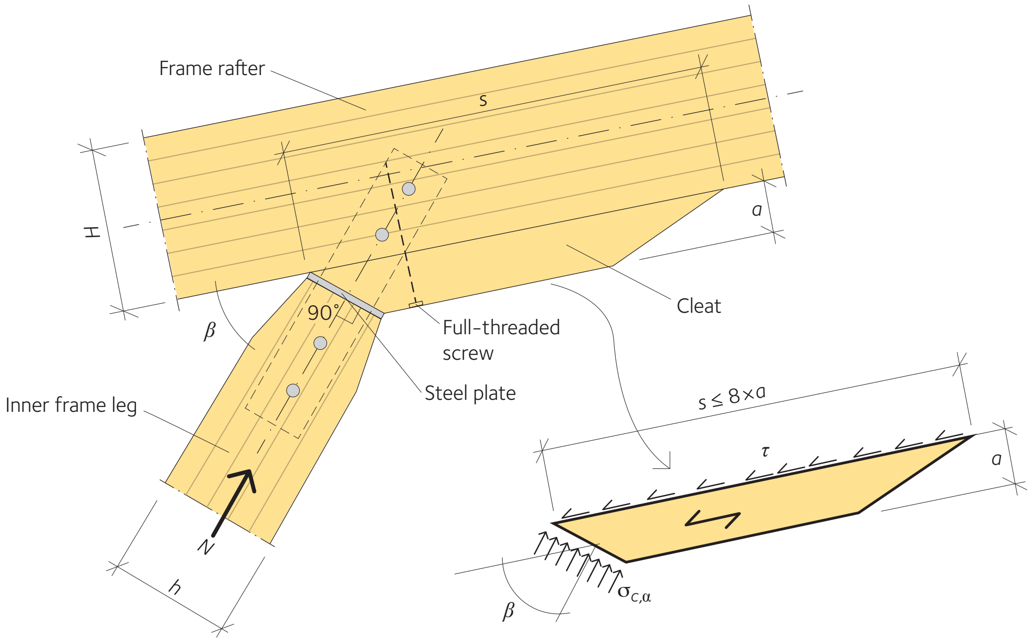


Figure 10.14 Framed joint: force distribution and symbols. The angle β between the rafter and the frame leg is normally within the range of 45° to 60°.

10.7 Details

Hinged base detail of frames with curved and finger-jointed haunches
The connections at the supports of portal frames are designed as nearly moment-free hinges. Depending on the choice of the connection, relatively small moments are likely to be transferred. However, they are usually of such limited magnitude that they do not need to be taken into account when designing the frame, although they will be of importance when designing the connection itself and its anchorage.

Besides allowing for rotations around the strong axis, the connection must also be able to transmit both vertical and horizontal forces, *see figure 10.15*.

It should be remembered that the need for constructional tolerances when constructing concrete bases with cast-in steel hardware is essential, *see e.g. figure 10.15 c)*. A moisture barrier at the bottom of the timber member is always recommended, even in service classes 1 and 2, to avoid moisture transfer into the end grain.

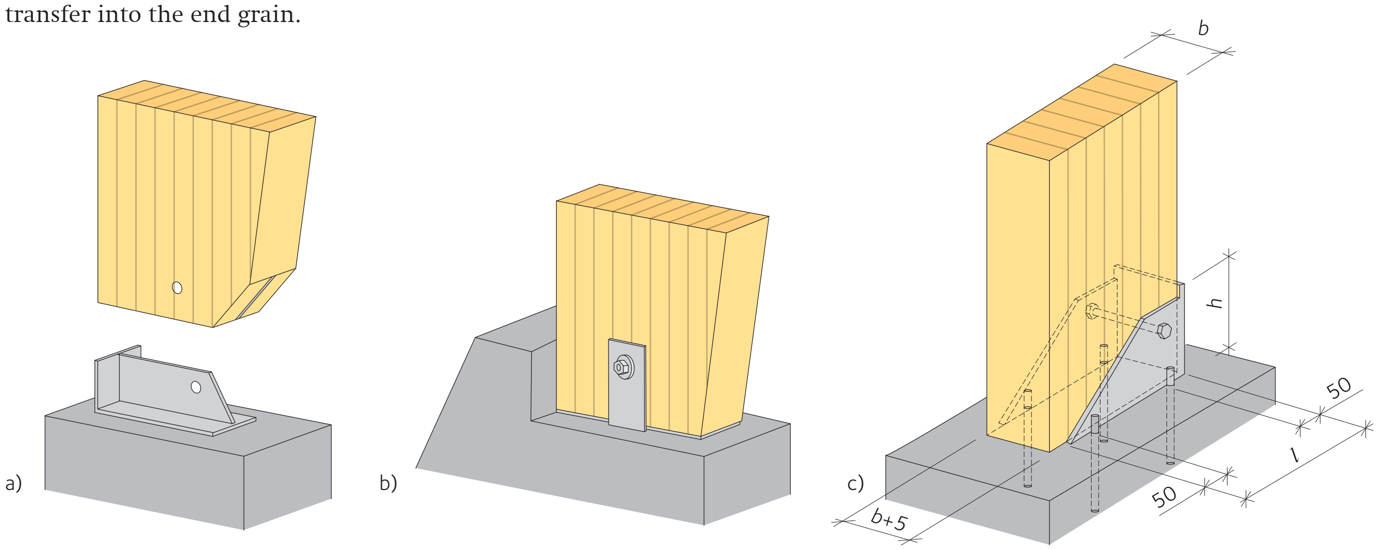


Figure 10.15 Possible frame base details of frames with curved and finger jointed haunches:
a) with slotted-in steel plate and with back plate,
b) with lateral steel plates and concrete abutment,
c) with lateral plates and back plate.

Hinged ridge joint

Hinged 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 connection should not restrict changes of angle in the beams. If this movement cannot take place, additional stresses will arise, which can lead to unforeseen damage to the structure – usually splits in the wood.

The connection can be made in the same way as the ridge connection of three-hinged frames shown in *Chapter 9, page 131*. Some other possible solutions are shown in *figure 10.16*.

Some details of built-up frames

Figure 10.17 shows some possible solutions for the base support and for the connection between the steel tie and the frame rafter.

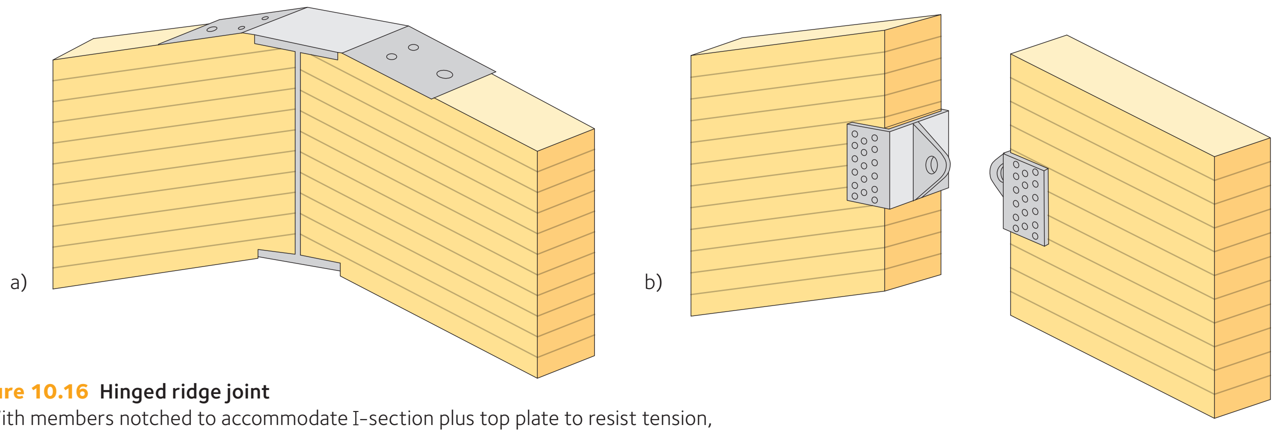


Figure 10.16 **Hinged ridge joint**
a) With members notched to accommodate I-section plus top plate to resist tension,
b) with hinge pin.

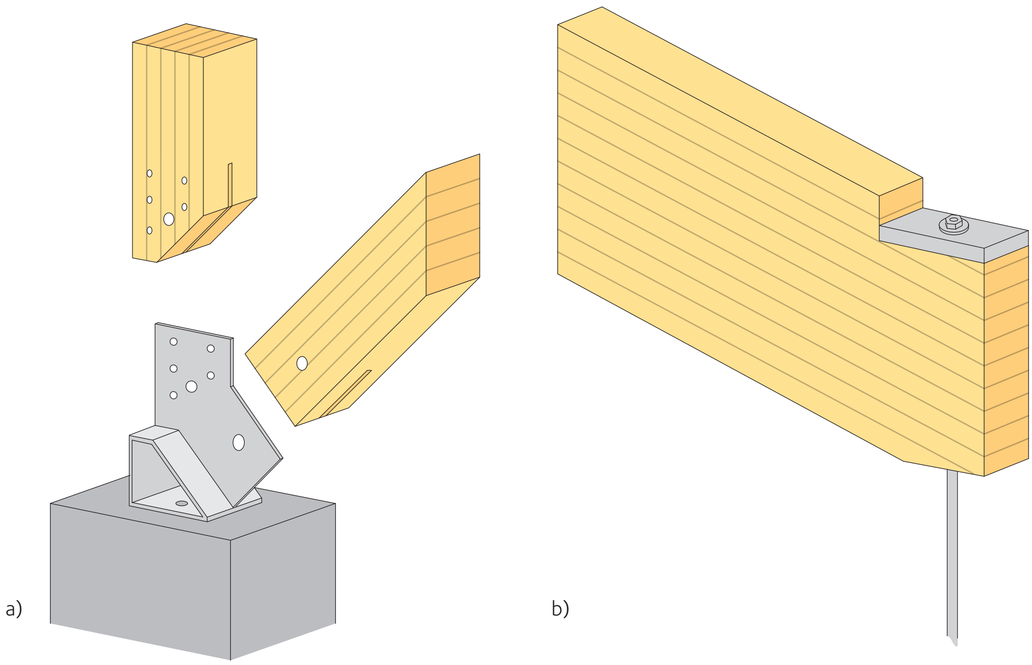


Figure 10.17 **Example of details in built-up frames**
a) Base support,
b) connection between steel tension member and frame rafter.

Arches

Arches are very suitable structures for execution in glulam — a material that can be produced in curved forms and with varying depth at no great extra cost. Most glulam arches have solid sections of constant depth, but composite sections of I- or box-shape are also available, especially for large spans.

The form of the arch should be chosen so that the bending moments are as small as possible. This means that the arch geometry should follow the thrust line of the dominating loading combination. The influence of moments cannot be avoided completely, however, since several load combinations must be taken into account, each with its own thrust line. As a compromise a parabola is often chosen, or a circle with a rise-to-span ratio f/l of approximately 0.14 to 0.15. For functional reasons, e.g. in order to increase the headroom near the supports, an elliptical or other arch form may be preferable.

Note that in such a case, the dividing line between the concepts of frames and arches is not well defined. The same result can be achieved by placing the arch on columns, *see figure 11.1*. The horizontal support reactions caused by the arch must in this case be taken care of by a tension tie between the springing points of the arch.

When the arch rests directly on the ground floor concrete slab or on an abutment, e.g. as in *figure 11.2 a)*, the horizontal forces can be taken up by the foundations if ground conditions permit, *see figure 11.2 b)*, or by tie rods under or within the floor, *see figure 11.2 c)*. In order to limit the size of the horizontal reactions the rise of the arch should be equal to or greater than 0.14 to 0.15 of its span. For a parabola or a circle this corresponds to an angle of spring of approximately 30°. In practice, arches are normally designed with a rise-to-span ratio $0.14 \le f/l \le 0.30$.

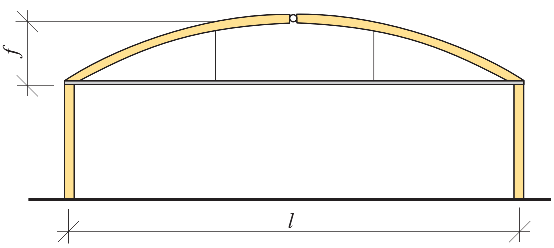


Figure 11.1 Arch with tension tie, on columns

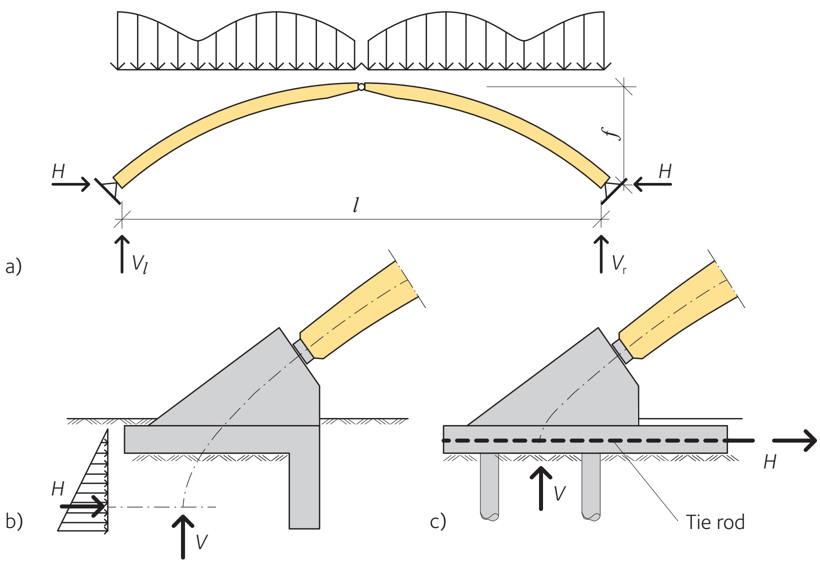


Figure 11.2 a): Arch springing from foundations,
b) the horizontal thrust is taken directly by the abutment and foundation,
c) the horizontal force is taken by a tie rod in the slab

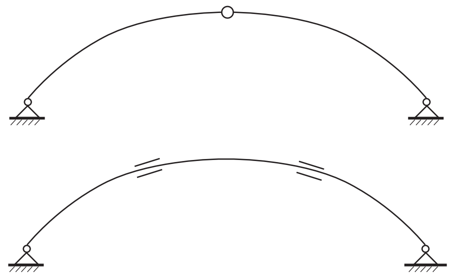


Figure 11.3 **Suitable placing of joints in arch structures**
a) Three-hinged arch,
b) two-hinged arch.

The choice between two- and three-hinged arches is made after similar considerations to those for frames, *see Chapter 10, page 140*. Three-hinged arches are thus preferable for 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 site. In such a case, a system with hinges placed only at the abutments is chosen (two-hinged arch). Hinges and rigid joints should be placed as in *figure 11.3*. The two-hinged arch has the inconvenience of being statically indeterminate, which means that it is sensitive to e.g. support settlements or/and moisture changes.

Hingeless arches are in practice never employed for load-bearing timber structures.

It should be noted that arches are not generally recommended where there are large concentrated loads. In fact, such isolated loads will increase the distance between the thrust line and the geometrical axis of the arch, which in turn will increase the magnitude of bending moments in the structure. Large 'point loads' thus dramatically reduce the "arch-like behaviour" of the structure. The effect of such loads is less negative in the case of roofs with heavy permanent, uniformly distributed loads.

According to some building codes, such as Eurocode 1-3, drifted snow load arrangements, with triangular load distributions on each half of the arch should be considered during design, see figure 11.4. In some countries, such as Norway and Finland, a drifted snow load case with only one triangular load distributed over only one half of the span may also be considered. Such a load condition gives rise to relatively large bending moments in the arch, especially when the span is large. One method of reducing the effect of bending moments could be to increase the "internal lever arm" of the arch, for example by choosing an arch structure, where each half consists of a lenticular truss, *see figure 11.4 b)*.

The three-hinged arch is the most common type of glulam arch, and is therefore the subject of the following design information. *Cf. also Chapter 9, page 140*.

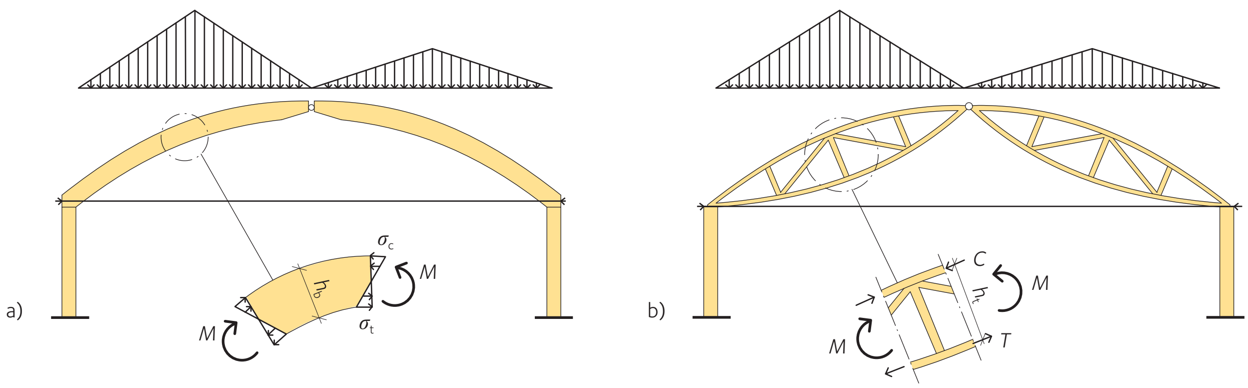
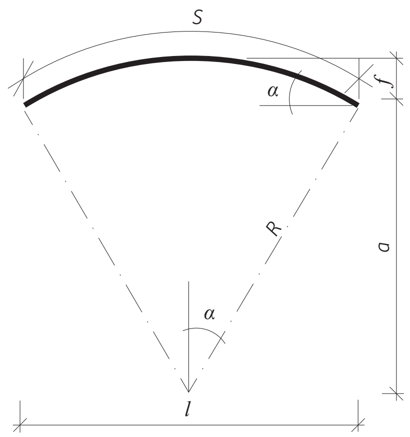


Figure 11.4 **Arch structures subjected to triangular load distribution**
a) Common arch structure,
b) Arch structure consisting of two lenticular trusses connected at the ridge.
The effect of relatively large local bending moments M (case a) can significantly be reduced by choosing a structure with larger internal lever arm (case b). (Comment: case b is a three-hinged truss, not an arch.)

11.1 Some useful geometrical properties of low rise arches

For circular shaped arches of any rise-to-span ratio and for arches of parabolic shape with rise-to-span ratio $f/l \approx 0.14$ to 0.15, relevant geometrical properties can be obtained from *figure 11.5*.

Note that a parabola does not have a constant radius; however, for small f/l ratios, parabolas and circular arches almost coincide, which means that the formulae given in *figure 11.5* can be used with sufficient accuracy for both types.



Radius of curvature $R = \dfrac{l^2 + 4 \times f^2}{8 \times f}$

Angle of spring (degrees) $\alpha = \arcsin\left(\dfrac{l}{2 \times R}\right)$

Arch length $s = 2 \times \alpha \times \dfrac{\pi}{180} \times R$

Figure 11.5 Geometrical properties of low rise arches

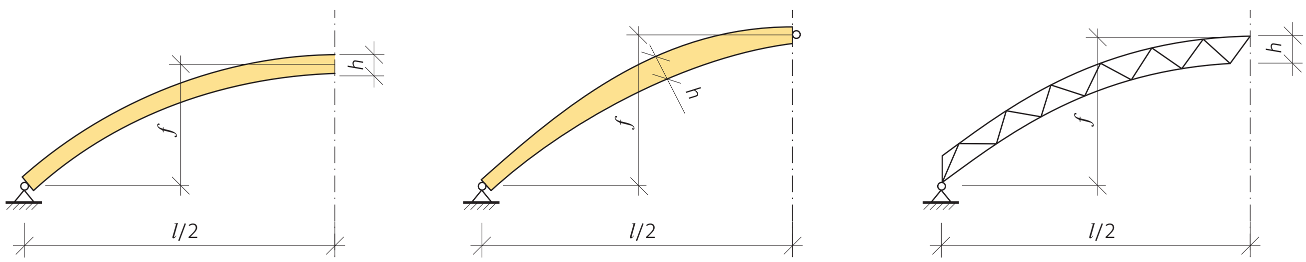
11.2 Conceptual design

Generally, architectural considerations determine the shape and possibly the rise of the arch. However, for economic reasons and also in order to limit the horizontal thrust, some “rules of thumb” concerning depth-to-span ratios, maximum span etc. should be followed. *Figure 11.6* gives indications for the preliminary design of three typical arch profiles. Deformations are rarely an issue with arches and can be controlled by adjusting the arch radius in the design stage.

11.3 Design of three-hinged arches

The three-hinged arch is stable against horizontal forces in its own plane and statically determinate, which means that the moment distribution is not affected by uneven settlement of the foundations or by unforeseen deformations in joints and connections, *cf. Chapter 10, page 140*. Furthermore, the three-hinged arch has hinges at the springing points, which simplifies the construction of the foundations. In poor soil conditions, the horizontal reactions at the supports can be taken by tension members between the foundations (located within or under the slab).

Calculation of the deformations is normally carried out with the aid of finite-element based computer programmes.



a) l = 30–100 m ; $h = l/50–l/30$; $f \geq l/10$ b) l = 30–100 m ; $h = l/50–l/30$; $f \geq l/7$ c) l = 50–120 m ; $h = l/40–l/20$; $f \geq l/8$

Figure 11.6 Preliminary design guidance for three different arch types
a) Two-hinged arch,
b) three-hinged arch,
c) trussed arch.

Bridge, Gislaved, Sweden.

11.3.1 Internal forces and support reactions

Normal forces and shear forces at any point of the arch depend upon the angle α between the tangent at the considered point and a horizontal line, *see figure 11.7*.

The normal force N, at any point of the arch can be calculated using the following formula:

11.1 $$N = H \times \cos\alpha + V_b \times \sin\alpha$$

The shear force V, at any point of the arch can be calculated using the following formula:

11.2 $$V = -H \times \sin\alpha + V_b \times \cos\alpha$$

where V_b is the shear force in a simply supported beam with the same span and the same vertical loading as the considered arch and H is the horizontal thrust, *see figure 11.7*.

Typically, the critical cross-section for arches is at approximately ¼ of the span. At that location the angle α can be calculated using the following expressions:

11.3a $$\alpha = \arcsin\left(\frac{l}{4 \times R}\right) \quad \text{for circular arches}$$

11.3b $$\alpha = \frac{180}{\pi} \times \arctan\left(\frac{2 \times f}{l}\right) \quad \text{for parabolic arches}$$

At ¼ of the span, the ordinate y_0 of the centre line of the arch can be calculated using the following expressions:

11.4a $$y_0 = \frac{f}{2} - \frac{l^2}{8 \times f} + \sqrt{\frac{l^4}{64 \times f^2} + \frac{f^2}{4} + \frac{l^2}{16}} \quad \text{for circular arches}$$

11.4b $$y_0 = \frac{3}{4} \times f \quad \text{for parabolic arches}$$

The governing load combinations, which are normally considered during the design of three-hinged arches are:

- Permanent loads + uniformly distributed snow load.
- Permanent load + triangularly distributed snow load – with different magnitudes – on each half arch.

In the rest of this section, indications are given concerning the determination of internal actions (normal forces, shear forces and bending moments) and support reactions.

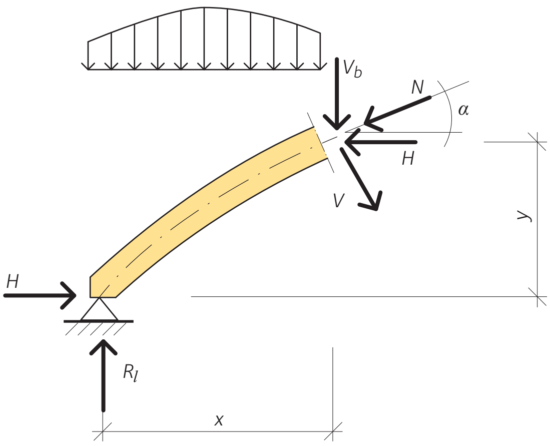


Figure 11.7 Internal force in an arbitrary cross section of the arch

The following considerations apply only to circular arches with rise-to-span ratio $f/l \approx 0.14$ to 0.15 and to parabolic arches obeying the following equation:

$$y = \frac{4 \times f}{l} \times \left(x - \frac{x^2}{l} \right) \quad (11.5)$$

where x is the abscissa with origin at the springing of the arch and y is the ordinate of the arch axis.

Permanent loads + uniformly distributed snow load
Under loading according to *figure 11.8*, the reaction and internal forces acting in a three-hinged arch can be calculated using the following expressions:

Vertical support reactions (at left abutment: R_l and at right abutment: R_r):

$$R_l = R_r = \frac{(q_r + g) \times l}{2} \quad (11.6)$$

Horizontal thrust:

$$H = \frac{(q_r + g) \times l^2}{8 \times f} \quad (11.7)$$

Maximum normal force at ¼ of the span:

$$N = \frac{(q_r + g) \times l^2}{8 \times f} \times \cos\alpha + \frac{(q_r + g) \times l}{4} \times \sin\alpha \quad (11.8)$$

Bending moment at ¼ of the span:

$$M \approx 0 \quad (11.9)$$

Shear force at springing ($x = 0$):

$$V_{sp} = -\frac{(q_r + g) \times l^2}{8 \times f} \times \sin\alpha_{spr} + \frac{(q_r + g) \times l}{2} \times \cos\alpha_{spr} \quad (11.10)$$

The shear force at the ridge is nil.

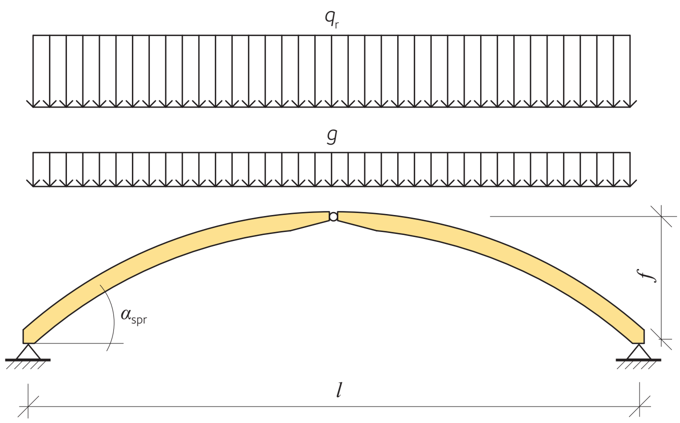
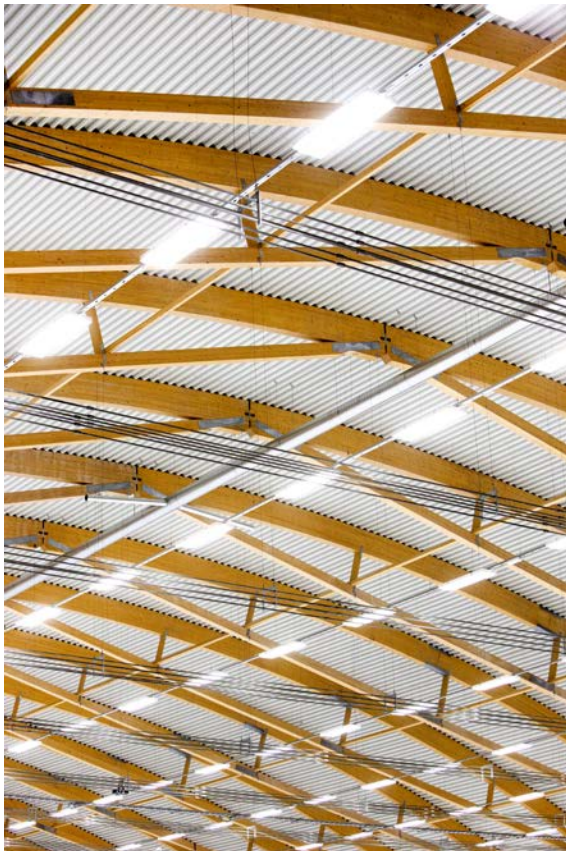


Figure 11.8 Three-hinged arch subjected to permanent load + uniformly distributed snow load



Bandy hall, Nässjö, Sweden.

Stockholm Central Railway Station, Sweden.

Permanent loads + triangularly distributed snow load
Under loading according to *figure 11.9*, the reaction and internal forces acting in a three-hinged arch can be calculated using the following expressions:

Vertical support reaction, left abutment:

11.11 $$R_l = \frac{7}{32} \times q_t \times l + \frac{1}{2} \times g \times l$$

Vertical support reaction, right abutment:

11.12 $$R_r = \frac{5}{32} \times q_t \times l + \frac{1}{2} \times g \times l$$

Horizontal thrust:

11.13 $$H = \frac{3}{64}\frac{(q_t) \times l^2}{f} + \frac{(g) \times l^2}{8 \times f}$$

Maximum normal force at ¼ of the span:

11.14 $$N = \frac{l^2}{8 \times f} \times \left(g + \frac{3 \times q_t}{8}\right) \times \cos\alpha + \frac{l}{4} \times \left(g + \frac{3 \times q_t}{8}\right) \times \sin\alpha$$

Bending moment at ¼ of the span (y_0 according to *equation 11.4*):

11.15 $$M = \frac{q_t \times l^2}{22.6} - \left(\frac{3}{64}\frac{(q_t) \times l^2}{f} + \frac{(g) \times l^2}{8 \times f}\right) \times y_0$$

Shear force at springing ($x = 0$):

11.16 $$V_{spring} = -\frac{l^2}{8 \times f} \times \left(g + \frac{3 \times q_t}{8}\right) \times \sin\alpha_{spr} + \frac{l}{2} \times \left(g + \frac{5 \times q_t}{16}\right) \times \cos\alpha_{spr}$$

Shear force at the ridge:

11.17 $$V_{ridge} = \frac{q_t \times l}{32}$$

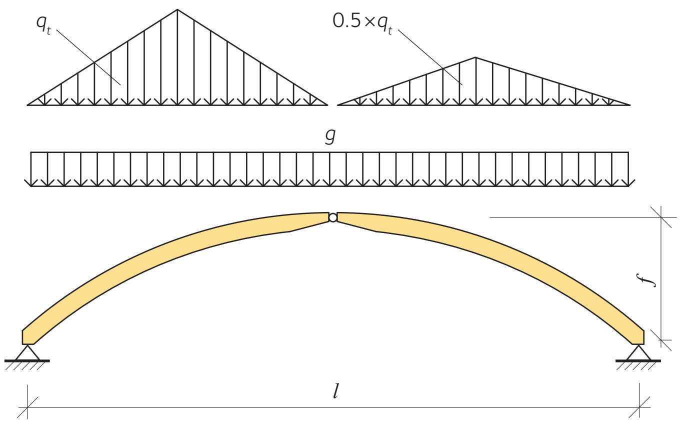


Figure 11.9 Three-hinged arch subjected to permanent load + triangularly distributed snow load according to Eurocode 1–3

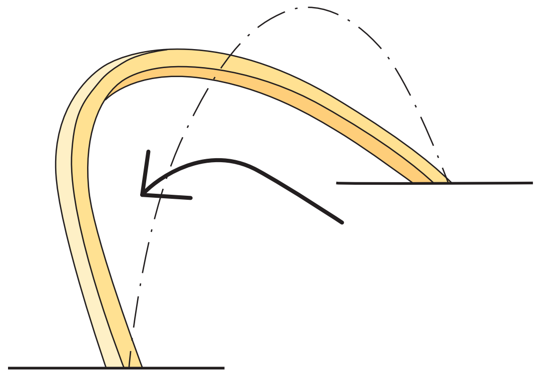
11.4 Stability control

As a rule, arches are slender structures and the design must therefore – even to a larger extent than for frames – be carried out by taking the risk of buckling into account, i.e. both in-plane and out-of-plane buckling.

11.4.1 Out-of-plane buckling (buckling around the minor axis)

An arch, which lies in one vertical plane, must be prevented from toppling over sideways, *see figure 11.10 a)*. This phenomenon is particularly relevant during erection. Two methods may be adopted to prevent this. One has fixed connections at the base, which is rather cumbersome to achieve and which also requires – especially in the case of large structures – a massive foundation to prevent overturning. Another, more commonly used method for achieving lateral stability during construction, is to erect two adjacent arches simultaneously. The arches are in this case provided with a temporary or permanent bracing, which prevents the structure from collapsing, *see figure 11.10 b)*.

The second major problem with respect to the behaviour of frames and arches in the lateral direction is that of lateral-torsional buckling (or out-of-plane buckling). Since timber elements can be fairly slender, out-of-plane buckling of the type illustrated in *figure 11.11* may occur.



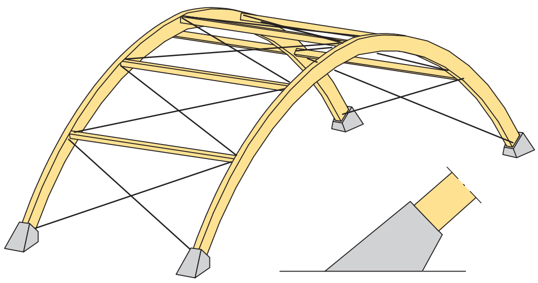


Figure 11.10 Considerations concerning the lateral behaviour of arches during erection
a) Possible toppling over of an arch sideways (overturning).
b) The lateral stability can be achieved by lateral bracing of arches with other elements, such as purlins from the roof structure in combination with cross-bracing.

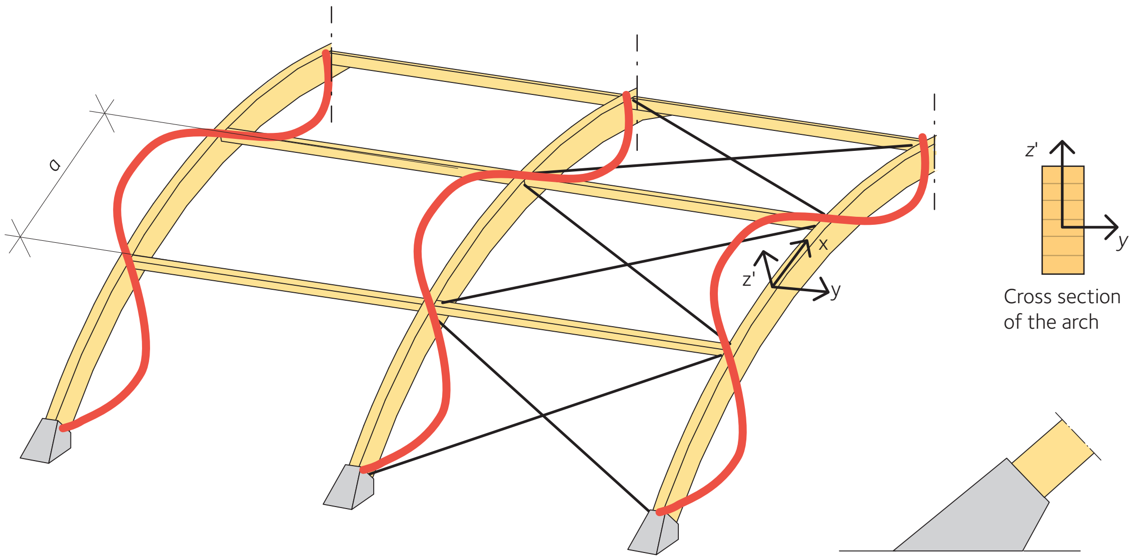


Figure 11.11 Lateral buckling of braced arches; bracings with spacing *a*

One method of increasing the stability with respect to out-of-plane buckling is to increase the stiffness of the frame or the arch in the lateral direction by increased lateral dimension. Another method is to reduce the centre-to-centre distance between purlins or to use stiff sheeting at the top of the purlins. Obviously the purlins must be connected to the arches with proper joints suitable for the transmission of bracing forces. In normal arches, out-of-plane buckling is checked in the same way as any other beam-column between lateral restraints. These bracings make the effective length of each element easily identifiable.

The design criterion is written as:

11.18

$$\begin{cases} \left(\dfrac{\sigma_{m,y,d}}{k_{crit} \times k_r \times f_{m,y,d}}\right)^2 + \dfrac{\sigma_{c,0,d}}{k_{c,z'} \times f_{c,0,d}} \leq 1 \\ \dfrac{\sigma_{c,0,d}}{k_{c,z'} \times f_{c,0,d}} + k_m \times \dfrac{\sigma_{m,y,d}}{k_r \times f_{m,y,d}} \leq 1 \end{cases}$$

where:

$\sigma_{c,0,d}$ is the design compression stress parallel to the grain.
$\sigma_{m,y,d}$ is the design bending stress about y.
$f_{c,0,d}$ is the design compression strength parallel to the grain.
$f_{m,y,d}$ is the design bending strength about y.
$k_{c,z'}$ is the reduction factor for out-of-plane buckling (i.e. buckling about the z'-axis, *see figure 11.11*).
k_{crit} is the reduction factor for lateral-torsional buckling, *see section 4.1.3, page 55.*
k_r is the reduction factor for taking into account the strength reduction due to bending of the laminates during production, *see Chapter 7, page 102.*
k_m is a factor that accounts for the redistribution of stresses and the effect of inhomogeneities in the material's cross section. It assumes a value of 0.7 for a rectangular cross section.

For more detailed information about the used symbols, *see also Chapter 4, page 53.*

Note that if the arch is continuously braced at its topside (e.g. by steel corrugated plate) lateral-torsional buckling should be checked only at the zones of the arch with negative bending moment (i.e. those zones where the arch is subjected to compression at its bottom side). Factor k_r is usually 1.0 for regular arch structures, i.e. where the curve radius is relatively large in relation to the lamella thickness.

11.4.2 In-plane buckling (buckling around the major axis)

Buckling in the plane of the arch is in general more complicated than for normal beam-column members. The common methods of analysis for arches are the following:

1. Linear buckling analysis.
2. Second order analysis.

Viewing tower, Härnösand, Sweden.

Linear buckling analysis
If linear buckling analysis is adopted, arches can be verified in the same manner as beam-columns, i.e. members subjected to simultaneous action of bending and compression. The calculation of stresses due to external loading is based on linear elastic theory considering the equilibrium of the undeformed static system. Stresses caused by in-plane and lateral imperfections and induced deflections are taken into account by multiplying the compression and bending strength values by buckling reduction factors k_c and k_{crit}.

Two-hinged arches always buckle in antisymmetrical configurations, regardless of whether the load is symmetrical or non-symmetrical, *cf. figure 11.15 b)*. This means that the crown moves horizontally and becomes a point of contraflexure. In the case of three-hinged arches, however, for low rise-to-span ratios (i.e. $f/l \leq 0.3$), the buckling is symmetrical if the loading is symmetrical, *see figure 11.12 a)*.

For the determination of the buckling load according to the simplified analysis, the buckling length l_e is required. Such a length can be determined either a) numerically, e.g. by means of a linear buckling analysis performed by a finite element computer programme or b) by empirical formulas.

According to Timoshenko et al. (1963) the critical value of the intensity of the load for a parabolic arch of uniform cross-section subjected to uniformly distributed load with two or three hinges, can be expressed by the following equation:

$$q_{cr} = \gamma \times \frac{E \times I}{l^3} \qquad (11.19)$$

The numerical factor γ is expressed graphically as a function of f/l in *figure 11.13*.

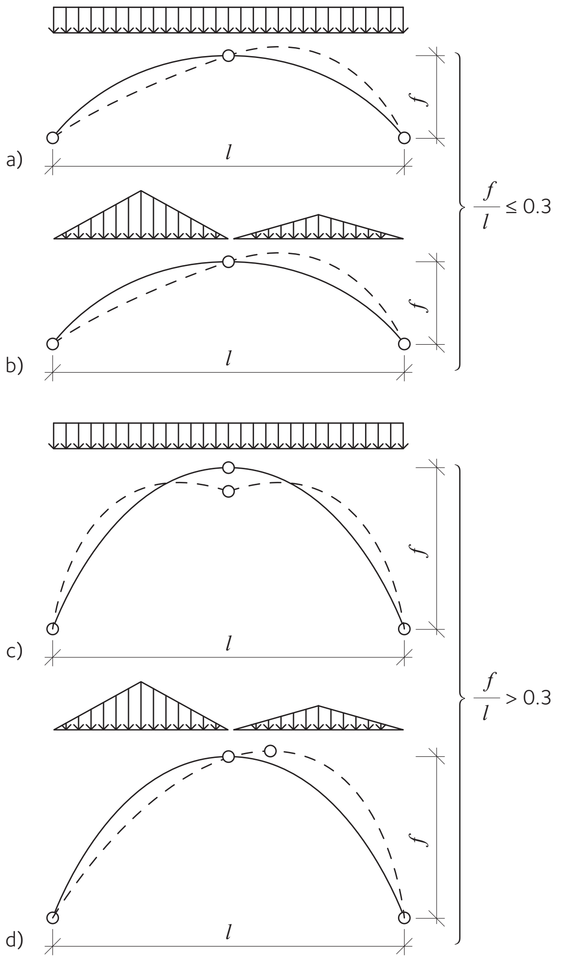


Figure 11.12 In-plane buckling of three-hinged arches
a) and b) apply for $f/l \leq 0.3$.
c) and d) apply for $f/l > 0.3$.
With load case d) either a symmetrical or asymmetrical buckling mode may occur.

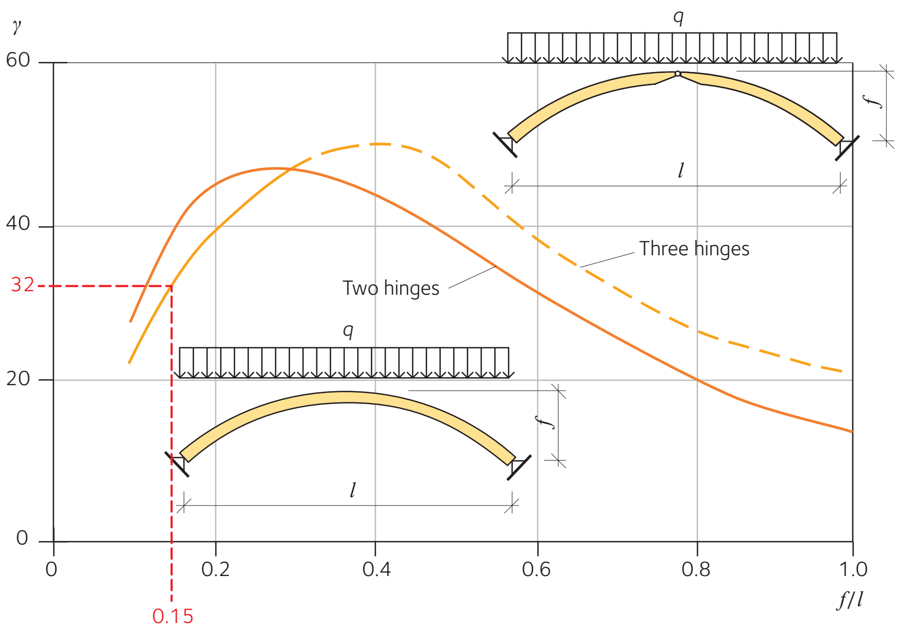


Figure 11.13 **γ-values as a function of *f/l*.** The portion of the curves indicated by dashed lines corresponds to symmetrical forms of buckling.

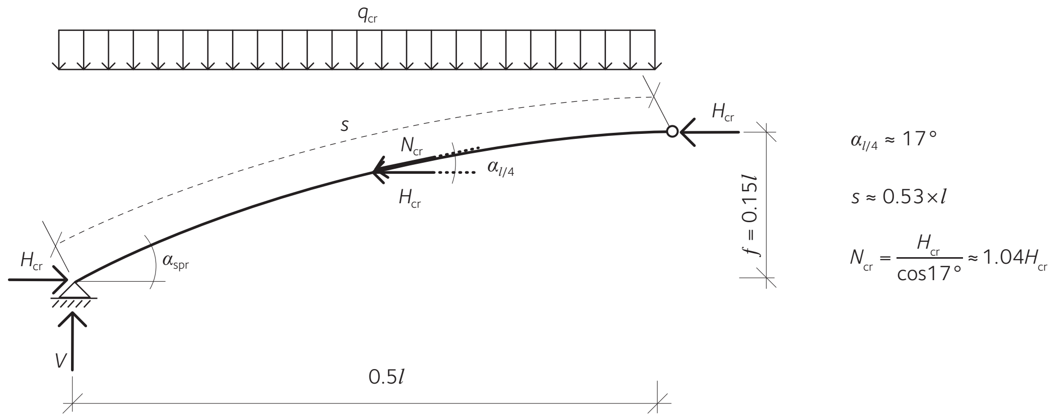


Figure 11.14 Forces acting on one half of a three-hinged arch. $\alpha_{l/4}$ is the angle between the axis of the arch and a horizontal line at the quarter point. s is the length of the semi-arch. N_{cr} is the Euler buckling load calculated at the quarter point of the arch.

A circular or parabolic arch with constant cross-section and uniformly distributed load with $f/l \approx 0.15$, which has the semi-arch length s, the angle between the axis of the arch and a horizontal line at the quarter point $\alpha_{1/4}$, and the Euler buckling load N_{cr} calculated at the quarter point of the arch is shown in *figure 11.14*.
(**Note** that the geometrical values shown in *figure 11.14* are based on the expressions given in *figure 11.5*).

The critical value of the normal force at the quarter point of the arch is:

11.20 $$N_{cr,1/4} \approx 1.04 \times \frac{q_{cr} \times l^2}{8 \times f} = 1.04 \times \gamma \times \frac{E \times I}{8 \times f \times l}$$

Comparing with the critical load (Euler) for an axially loaded column with length s (= 0.53 × l):

11.21 $$N_{cr} = \pi^2 \times \frac{E \times I}{(\beta \times s)^2}$$

and observing that $\gamma \approx 32$ for $f/l = 0.15$ and, *see figure 11.13*, the buckling length factor β becomes:

11.22 $$\beta \approx 1.17$$

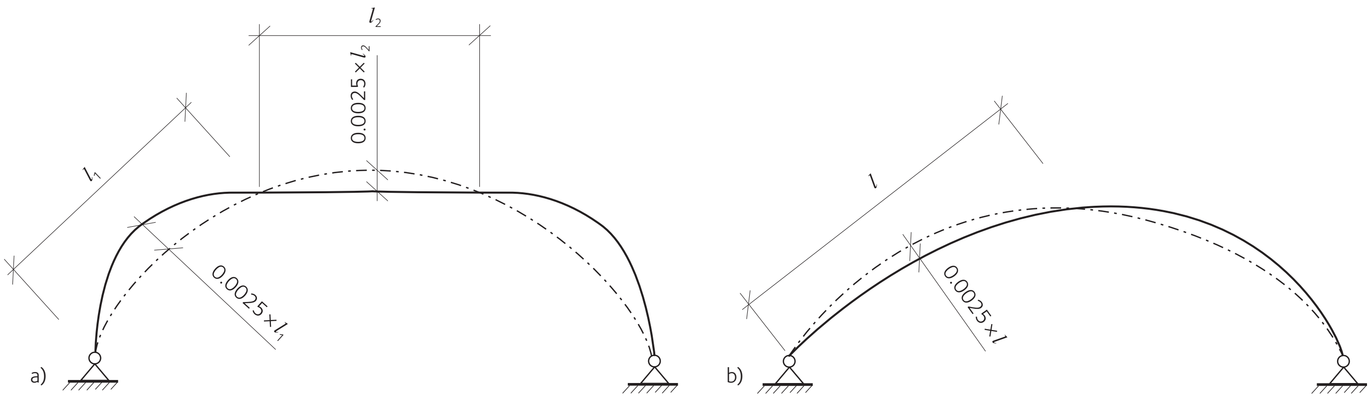


Figure 11.15 Recommended assumptions for initial deviation in the geometry of arches according to EC5
a) Symmetric (assumed) initial imperfection shape, affine with the second buckling mode of a two-hinged arch,
b) asymmetric (or sway mode) imperfection shape affine with the first buckling mode.

For practical design, a more conservative value of the critical length l_e is often assumed for the analysis of in-plane buckling of arches:

$$l_e = \beta \times s = 1.25 \times s \tag{11.23}$$

Obviously, non-symmetric load cases, such as those shown in *figure 11.4*, will give critical buckling lengths, which will differ from symmetrical load cases. However, in the phase of preliminary design it is also possible to use a critical length $l_e = 1.25 \times s$ also for non-symmetrical load case, which in most cases will lead to conservative results.

The design criterion takes into consideration the simultaneous action of bending moment and axial force and is written as:

$$\frac{\sigma_{m,y,d}}{k_r \times f_{m,y,d}} + \frac{\sigma_{c,0,d}}{k_{c,y} \times f_{c,0,d}} \leq 1 \tag{11.24}$$

where:

$k_{c,y}$ is the reduction factor for in-plane buckling (i.e. buckling about the y-axis, *see figure 11.11, page 159*) and with the other symbols explained in *equation 11.18* and *Chapter 4, page 53.*

Second order analysis

As the load increases in a compressed structure, the deformations also increase, and these generate a larger bending moment, which in turn produces larger deformations. If the load continues to increase, this process will eventually lead to the stress exceeding the material's strength somewhere in the structure, thus causing a failure.

A geometric non-linear calculation takes account of the way the moment (caused by the additional deformations) increases with the rise in the axial load. If the non-linear calculation also takes account of the structure's initial imperfections, the result will give the "correct" sectional forces. These sectional forces can be used directly in the design process, without the need to adjust them with any reduction factors for buckling (i.e. k factors). The design values are thus achieved by checking the cross-section for simultaneous compression and buckling without reference to the risk of buckling (namely by setting the reduction factor for buckling at $k_c = 1.0$).

This way of designing a timber structure requires the use of suitable finite element software, which is now widely available. The way that the geometric imperfections should be dealt with depends, to some extent, on the modelling options offered by the software in question. The first or second buckling mode is usually a good approximation for the form of the initial imperfection, *see figure 11.15, page 162*. An alternative is to use the deformations obtained from a linear elastic analysis using the relevant load case as input data for the initial imperfections in the structure. A typical value for the maximum amplitude of the imperfection when calculating glulam structures is around $L/400$ according to Eurocode 5 (*see figure 5.3 in SS-EN 1995-1-1*).

Malmö Central Railway Station, Sweden.

Kolmården zoo, Sweden.

Simplified second order deflection and moment computation
For arches having initial imperfections with a shape affine with one of the lower buckling modes, the following simple, approximate formula can be used to give a good estimate of the enlargement factor for the displacement amplitude δ_0, i.e. the increased displacements due to second order effects, *cf. Chapter 4, equation 4.19, page 65*:

11.25
$$\delta_{tot} = \delta_0 + \delta_p = \frac{\delta_0}{1 - H/H_{cr}}$$

where H is the actual governing arch force component (e.g. the horizontal force) and H_{cr} is the critical value of H for the associated buckling mode.

The bending moment M_0 according to the linear theory increases with the same enlargement factor as above to:

11.26
$$M^{II} = \frac{M_0}{1 - H/H_{cr}}$$

where M^{II} is the bending moment according to the second order theory. When using *equation 11.26*, the best approximation is obtained when the moment M_0 is affine with the moment curve at buckling.

11.5 Tension perpendicular to grain and shear

For combined tension perpendicular to grain and shear the following expression shall be satisfied:

11.27
$$\frac{\tau_d}{f_{v,d}} + \frac{\sigma_{t,90,d}}{k_{dis} \times k_{vol} \times f_{t,90,d}} \leq 1$$

τ_d	is the design shear stress.
$\sigma_{t,90,d}$	is the design tensile stress perpendicular to the grain.
$f_{t,90,d}$	is the design tensile strength perpendicular to the grain.
$f_{v,d}$	is the design shear strength.
k_{dis}, k_{vol}	are explained in *Chapter 7, page 102.*

For certain load conditions a large part of an arch will be subjected to positive bending moment, which generates tension perpendicular to the grain. However, in large parts of the arch such stresses will be relatively low. It is recommended therefore, in accordance with the Australian building code AS 1720.1—1997, that only the volume of timber stressed above 80 percent of the maximum value in tension perpendicular to the grain is taken into account for the determination of k_{vol}.

11.6 Details

Hinged base detail of arches
The connections at the supports of portal arches are designed as nearly moment-free hinges. Depending on the choice of the connection type, relatively small moments are likely to be transferred. However, they are usually of such limited magnitude that they do not need to be taken into account when designing the arch, although they will be of importance when designing the connection itself and its anchorage.

Besides allowing for rotations around the strong axis, the connection must also be able to transmit both vertical and horizontal forces, *see figure 11.16*.

A moisture barrier at the bottom of the timber member is always recommended, even in service classes 1 and 2, in order to avoid moisture transfer into the end grain.

Arch with slotted-in steel plates and dowels, Kolmården zoo, Sweden.

Hinged ridge joint
Hinged ridge joints transfer horizontal and vertical forces. Moments are transferred only to a limited extent and are generally not taken into account in the design. The connection should not restrict changes of angle in the beams. If this movement cannot take place, extra stresses will arise that can lead to unforeseen damage to the structure, usually splits in the wood.

The joint can be executed in the same way as the ridge joint for three-hinged trusses, as shown in *Chapter 9, page 131*. Some other possible solutions for hinged ridge joints are shown in *figure 11.17*.

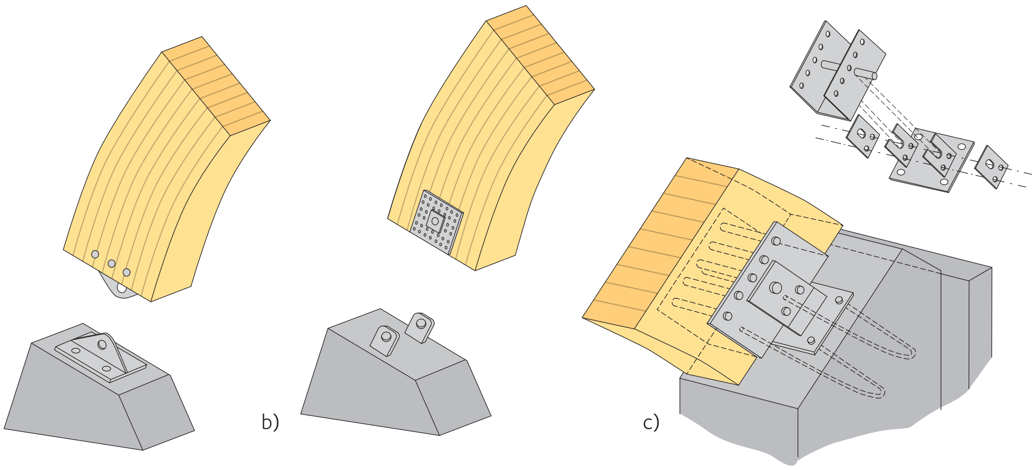


Figure 11.16 **Possible arch base details of arches**
a) with end plate and hinge pin,
b) with nail plate and hinge pin,
c) with end U-shaped plate, hinge pin and side lugs.

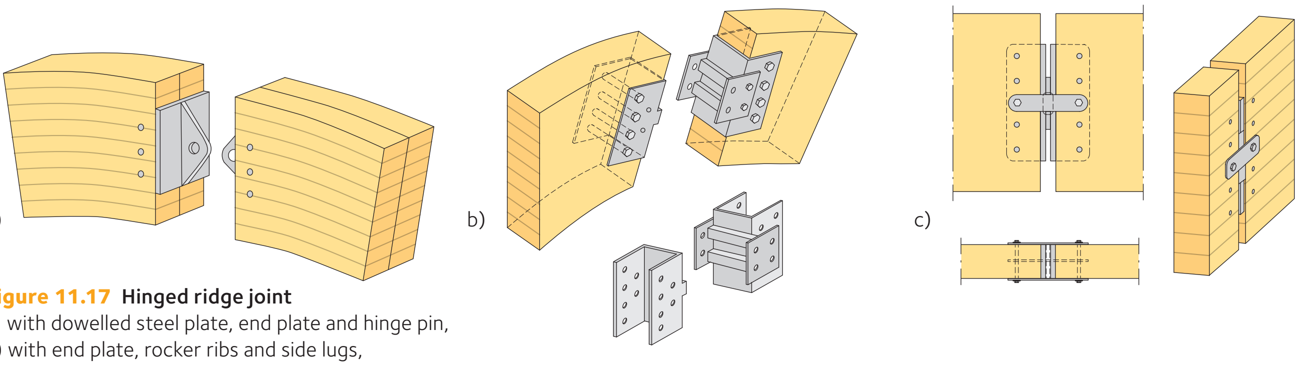


Figure 11.17 **Hinged ridge joint**
a) with dowelled steel plate, end plate and hinge pin,
b) with end plate, rocker ribs and side lugs,
c) with dowelled end plate, rocker ribs and side lugs.

Purlins

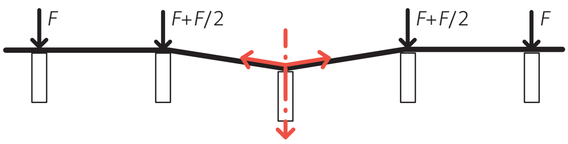


Figure 12.1 Example of purlin system acting as a tie member in the case of failing main member (to be avoided). Figure taken from Dietsch and Winter 2010.

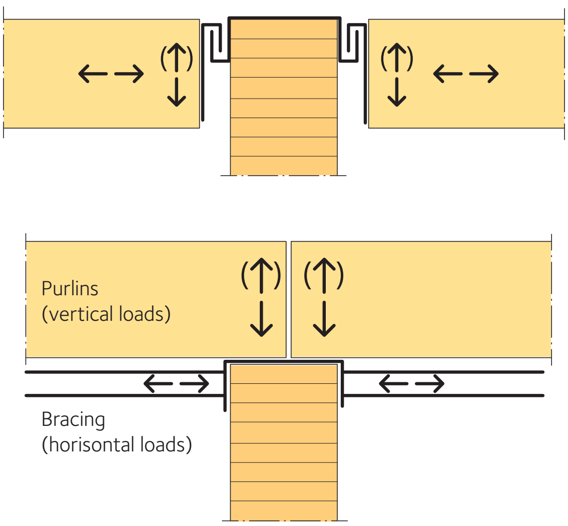


Figure 12.2
a) Connection to transfer horizontal and vertical loads, potentially enabling detachment in case of failure,
b) Separation of load-bearing structure for horizontal and vertical loads, enabling detachment in case of failure.
Figure after Dietsch and Winter 2010.

Glulam purlins normally consist of straight glulam beams of constant cross-section. They can be simply supported (on two supports) and hung in between the primary beams, or alternatively they are continuous beams on 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 purlins overlapping over supports, or with hinged joints as in a 'Gerber System'.

Continuous purlins or Gerber-jointed purlins are advantageous because their cross-sections are smaller than simply supported purlins. However, with regard to robustness and risk of progressive collapse, simply supported purlins are preferable. Purlins usually serve double tasks – to transfer vertical loads to the main beams, and to brace the main beams against wind loads and instability. However, when a main member fails, continuous purlins will act as a tie beam and redistribute the load from the failing member to the neighbouring members, *see figure 12.1*. As most failures are not caused by local defects (e.g. bad material quality or decay), but rather by global (repetitive) mistakes made by designers and builders, the neighbouring main members are usually not able to take up the extra load, thus resulting in progressive failure. Thus, for robustness reasons and prevention of progressive collapse, simply supported purlins, not acting as tie beams, are preferable. For robustness reasons, purlins should not serve double tasks, but either transfer vertical loads or horizontal loads. Thus the connections between purlins and main beams also have to be specially designed to take either vertical loads or horizontal loads. Joints have to enable a detachment of the purlin system in the case of failure, *see figure 12.2*.

12.1 Overlapping purlins

Overlapping purlins have the advantage that their load carrying capacity is doubled over supports, where the bending moments are greatest, *see figure 12.3 a), page 167*. The deflection of the purlin is also reduced in the same way as with haunches. The overlap should be made sufficiently long to reduce the bending moment to half, *see figure 12.3 c), page 167*; the field bending moment then becomes critical. Bending moments, support reactions and maximum deflection can be calculated for a continuous beam with constant moment of inertia using tabulated values. Although the variation in stiffness affects the distribution of bending moments positively, this is counteracted by unavoidable deformations in the joints. If there are at least two similar bays, with overlaps as in *figure 12.3 b), page 167*, the results in *table 12.1* apply for design bending moments M_d, design joint forces F_d, necessary overlaps x and maximum deflections w.

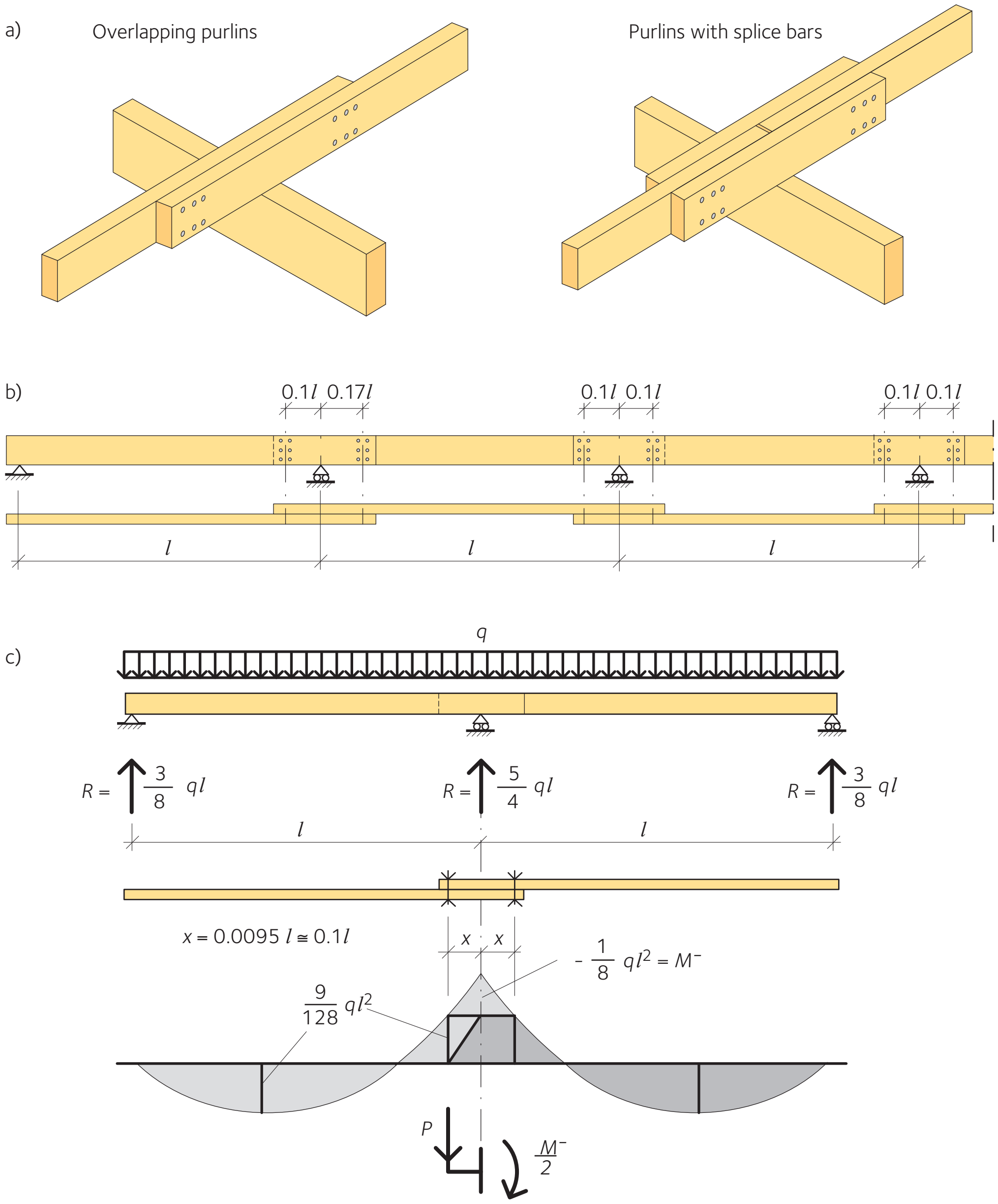


Figure 12.3
a) Overlapping purlins: 2 purlins overlapping (left) or 2 purlins meeting, with splice boards (right).
b) Continuous purlin system with overlaps.
c) Overlaps should be made long enough to allow the bending moment to decrease to half the maximum value (example for 2-span beam).

Table 12.1 Design bending moments, design joint forces, necessary overlaps and maximum deflections for overlapping purlins. *See figure 12.3.* q_d is the uniform design load and l the bay length. Compare also with *figure 12.3 c)* for 2 bays.

	2 bays	3 or more bays	
		End bay	Intermediate bay
Design bending moment M_d	$0.07 \times q_d \times l^2 = \frac{9 \times q_d \times l^2}{128}$	$0.08 \times q_d \times l^2$	$0.046 \times q_d \times l^2$
Design joint force F_d	$0.0625 \times q_d \times l = \frac{5}{8} \times q_d \times l$	$0.42 \times q_d \times l$	$0.42 \times q_d \times l$
Overlap necessary x	$0.1\,l$	$0.1\,l$ *	$0.1\,l$ *
Maximum deflection w	$\frac{0.54 \times q_d \times l^4}{100EI}$	$\frac{0.69 \times q_d \times l^4}{100EI}$	$\frac{0.32 \times q_d \times l^4}{100EI}$

* At the inner side of the first intermediate support (e.g. in the second bay), an overlap of 0.17 l is used, *see figure 12.3 b)*.

It is common to use double purlins in end bays in order to use the same dimensions in all bays, despite the higher bending moments in end bays.

Unjointed purlins in two or more bays are designed with regard to the fact that the roof beams form a deformable support. The support bending moment given in *table 12.1* can therefore be reduced by 10 percent. 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, which can be designed for 1.1 $q_d l$ (instead of 1.25 $q_d l$).

12.2 Purlins with Gerber system

A typical Gerber system is designed so that bending moments in the bay and above the supports are approximately equal. In order to reduce the risk of progressive collapse, the system should be designed so that every second bay is free from hinges if one bay should collapse.

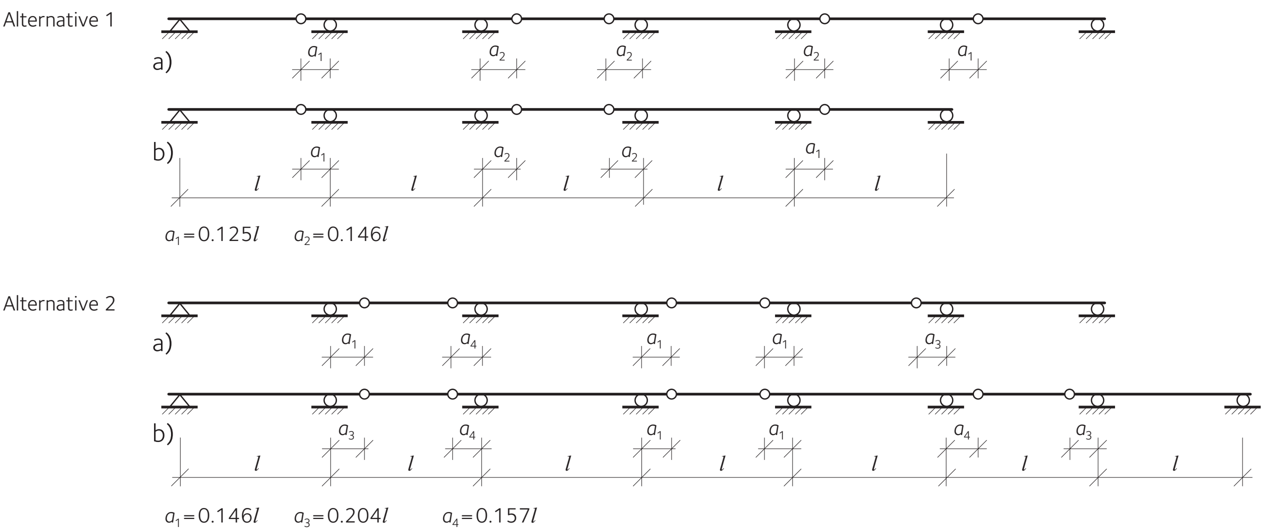


Figure 12.4 Various types of Gerber systems. Alternative 1: Joint in end bay. a) Even number of bays, b) odd number of bays. Alternative 2: End bays without joints. a) Even number of bays, b) odd number of bays.

Table 12.2 Design bending moments, design shear forces and joint forces, location of hinges and deflections for purlins with Gerber system. *See also figure 12.4.* q_d is the uniform design load and l the bay length.

	More than 3 bays, joint in end bays (alternative 1 in *figure 12.4*)		More than 3 bays, no joint in end bays (alternative 2 in *figure 12.4*)	
	End bays	Intermediate bays	End bays	Intermediate bays
Design bending moment M_d	$0.096 \times q_d \times l^2$	$0.063 \times q_d \times l^2$	$0.086 \times q_d \times l^2$	$0.063 \times q_d \times l^2$
Design shear force V_d	$0.44 \times q_d \times l$	$0.56 \times q_d \times l$	$0.59 \times q_d \times l$	$0.56 \times q_d \times l$
Design joint force F_d	$0.44 \times q_d \times l$	$0.35 \times q_d \times l$	–	$0.35 \times q_d \times l$
Location of hinges *a*	$0.125 \times l$	$0.146 \times l$	According to *figure 12.4*	According to *figure 12.4*
Deflection *w*	$\frac{0.72 \times q_d \times l^4}{100EI}$	$\frac{0.52 \times q_d \times l^4}{100EI}$	$\frac{0.77 \times q_d \times l^4}{100EI}$	$\frac{0.52 \times q_d \times l^4}{100EI}$

For purlins with at least four bays, if the joints are placed as in alternative 1 or 2 in *figure 12.4, page 168*, bending moments, shear forces, joint forces, location of hinges and size of deflections can be calculated as shown in *table 12.2, page 168*. It is often practical to choose the same depth for purlins in the end bays and intermediate bays and, if required, make wider purlins in the end bays instead to increase their capacity.

12.3 Inclined purlins

When the roof slope exceeds 1:10 (circa 6°), the inclination of the purlins must be taken into account. The vertical loads, i.e. snow load and self-weight, are split into a component perpendicular to the roof ($q \times \cos \alpha$) and a component parallel with the roof ($q \times \sin \alpha$) as in *figure 12.5*.

As a rule, the roof covering is stiff enough to take up the component in the plane of the roof ($q \times \sin \alpha$) by the diaphragm effect. This is usually not checked. In inclined roofs, if the two roof sheathings on either side of the ridge are sufficiently connected across the ridge, the sheathing can take the force component parallel to the roof (diaphragm effect) and the purlins are subjected to bending in the stiff direction only, *see figure 12.6*. If the roof slope exceeds 1:10 (circa 6°), or the diaphragm effect cannot be counted on, the purlins must be checked for simultaneous bending in the stiff and weak directions, e.g. as in *section 4.1, equation 4.4, page 54*.

Note the different bending strengths for bending in weak and strong directions for combined glulam. 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 12.7*. The trusses should then be checked for the extra load placed on them by the suspension rods.

The joints between purlins and roof beams can, for example, be carried out with various types of steel angles or screws. Those joints have to be designed for the load component parallel with the roof ($q \times \sin \alpha$) in order to prevent the purlin from sliding on the roof beams. For purlins located between roof beams (joints with beam hanger), the joint has to be designed for the total force (both components).

If the purlins are used to stiffen the primary beams, or to transfer tension or compression forces to wind bracing, the load-carrying capacity should be checked for simultaneous compression and bending as in *section 4.2, page 62*. Joints and connections to roof beams have also to be checked for these forces, *see Chapter 14, page 198*.

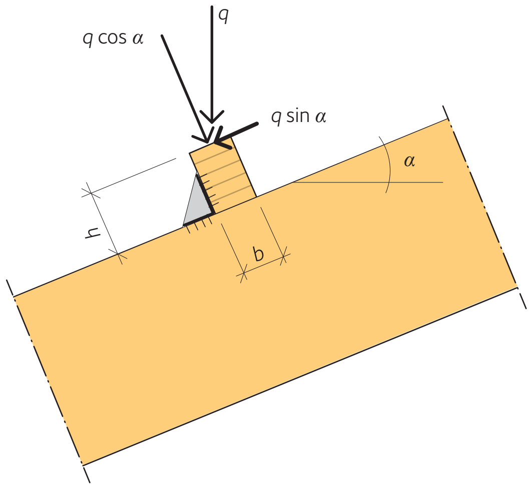


Figure 12.5 **Vertical load on inclined purlin divided into components**

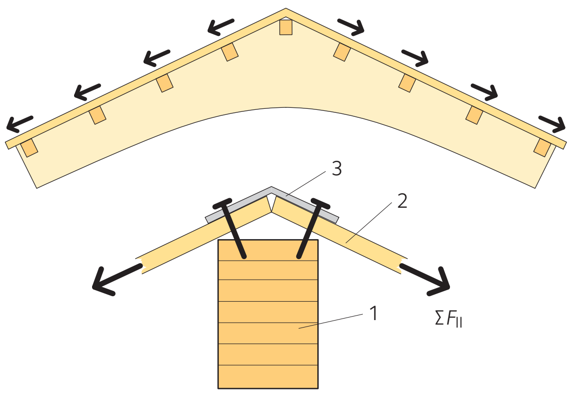


Figure 12.6 **Inclined roof with purlins and roof sheathing**
1) Ridge purlin,
2) roof sheathing,
3) nailing plate.
If the two roof sheathings are sufficiently connected across the ridge, the sheathing can take the force component parallel to the roof ($F_{||}$; diaphragm effect) and the purlins are subjected to bending in the stiff direction only. The ridge purlin has to be designed for the vertical force component.

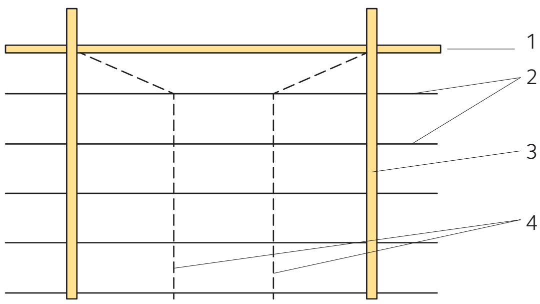


Figure 12.7 **In steep roofs the other purlins can be hung up from the ridge purlin**
1) Ridge purlin,
2) other purlins,
3) roof beam,
4) suspension rod.

Horizontal stabilisation

In the design of structural systems, the way horizontal stability is achieved is of fundamental importance. Horizontal loads may act in any direction parallel to the horizontal plane of the building. They can be generated by wind loads, by seismic events or by impact caused by fork-lift trucks. If the columns are supporting runway girders and crane girders, horizontal forces caused by starting, braking and stopping also occur. Moreover, horizontal forces are also caused by gravity loads, due to the inevitable out-of-straightness of columns and beams.

All structures undergo some changes in shape under horizontal load. In a stable structure, the deformations induced by a lateral load are typically small. Moreover, in a stable structure, internal forces are generated by the action of a lateral load; these forces tend to restore the structure to its original shape after the load has been removed, *see e.g. figures 13.1 b)–e)*. By contrast, in an unstable structure such internal forces that tend to restore the structure to its original configuration are not generated. In an unstable structure, the deformations induced by a lateral load are typically large and tend to continue to increase as long as the load is applied, *see e.g. figures 13.1 a)*.

It is the fundamental responsibility of the structural engineer to assure that a proposed structure forms a stable configuration.

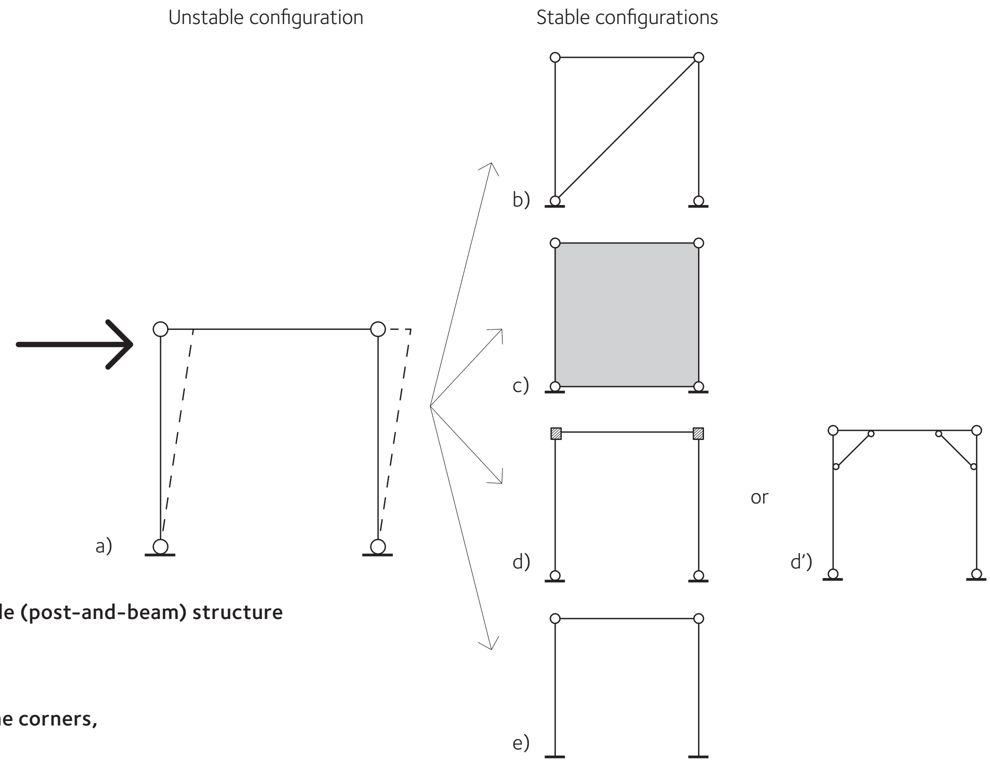


Figure 13.1 a) Conversion of an unstable (post-and-beam) structure into a stable structure by means of:
b) diagonal bracing,
c) shear wall,
d), d') stopping of angular changes at the corners,
e) clamping at column footings

13.1 General considerations

There are just a few fundamental ways to convert a self-standing structure of the general type from an unstable to a stable configuration. These methods are illustrated in *figure 13.1*.

The first is to add a diagonal member to the structure, *see figure 13.1 b), page 170*. In this way the structure cannot undergo the "parallelogramming" indicated in *figure 13.1 a), page 170*, without a very large elongation of the diagonal member – which will not occur if the diagonal is adequately sized. Another method used to assure stability is through shear walls, i.e. rigid planar surface elements that resist changes in shape of the type illustrated in *figure 13.2 c)*. A third method used to achieve stability is by stopping the angular changes that occur between members either at the corners, *figure 13.1 d) and (d'), page 170*, or at the footings, *figure 13.1 e), page 170*, or at both places.

There are, of course, variants on the basic methods of assuring stability. Still, most structures composed of discrete elements rely on one or the other of these basic approaches for stability. More than one approach can be used to brace a structure, e.g. both rigid joints and diagonals. This is normally done when the stiffness of the structure needs to be improved and, in such a case, the structural redundancy is obviously increased.

Glulam columns.

13.2 Global stabilisation

In order to check the overall stability of a structure and to evaluate correctly the buckling phenomena of its structural elements, it is necessary to conceive the structure in the three-dimensional space. This concept also applies when the structure can be considered and analysed as composed of various substructures behaving in one plane. Consider for example a structure consisting of a set of identical frames, *see figure 13.2 a)*. In order to evaluate the stresses caused by the applied loads, such a structure can be analysed as a number of single planar frame substructures. However, the stability of the structure must be analysed in a global manner, i.e. not only in the substructure plane, *figure 13.2 b)*, but also out of the plane of the frame, *figure 13.2 c)*.

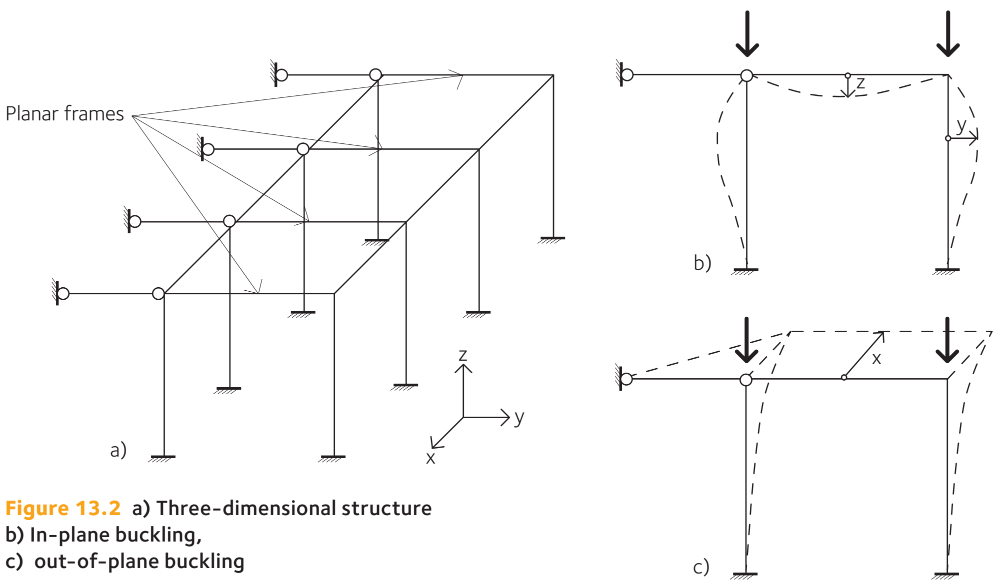


Figure 13.2 **a) Three-dimensional structure**
b) In-plane buckling,
c) out-of-plane buckling

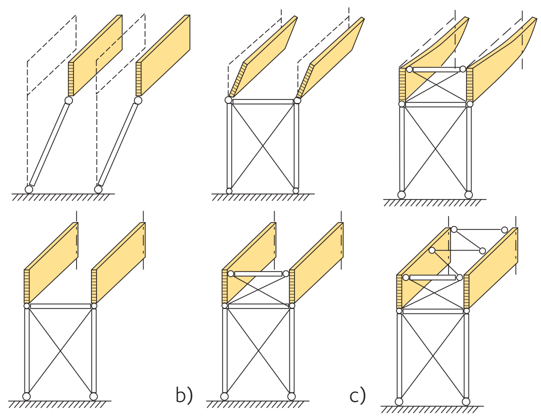


Figure 13.3 Stabilization of a bay of a structure

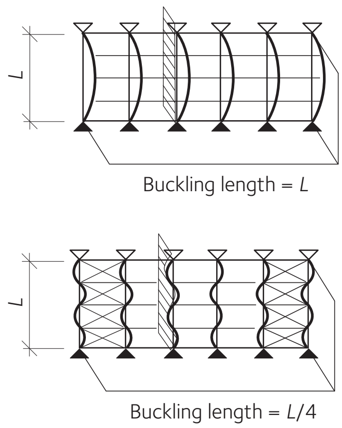


Figure 13.4
a) Unbraced roof structure,
b) roof structure braced with crossing pairs of steel rods.

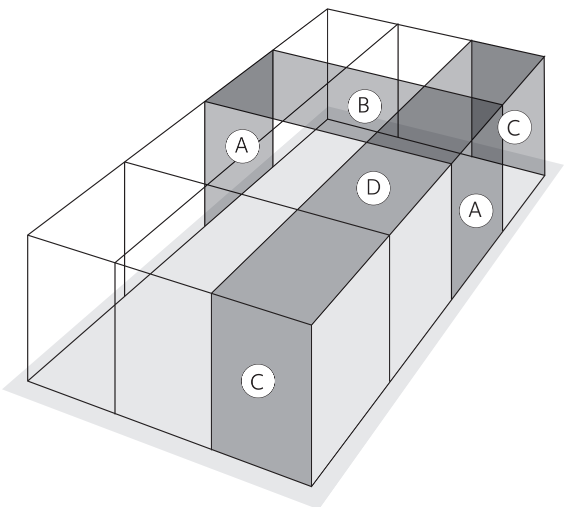


Figure 13.5 Different bracing elements in a simple industrial building

It is important to observe that structures must be able to transmit loads from the roof level down to the foundations. *Figure 13.3* illustrates this concept. If a bay of a structure is to be stabilized against lateral loads, the first step is to prevent lateral movement of the columns, which may be achieved, for example, by introducing diagonal members as shown in *figure 13.3 a)*. However, although the columns are now braced, loads from the roof plane cannot yet be transmitted to the foundation through diagonal members. Therefore, the beams need a bracing system at their support points that permits the transmission of horizontal loads from the roof to the underlying structure, for example as shown in *figure 13.3 b)*. Although the system may look stable now, the beams are still prone to lateral buckling, which may be prevented, for example, by creating a horizontal truss at the roof plane, where the beams act as compression and tension chords, *see figure 13.3 c)*.

Bracing is also necessary to:

- Keep the filler beams (purlins or separate struts) in place so that they can give the proper lateral support to the compression zone of the beam under in-plane bending.
- Prevent lateral buckling of the entire roof plane, *see figure 13.4*.

The engineer often encounters situations where the adequacy of the bracing system against lateral-torsional buckling is uncertain. Typical examples are where the beam in question is not braced at the compression zone but at or near the tension zone, or when the purlins or light-guage decking floor systems are not solidly attached to the beam in question. In such doubtful situations, it is always wise to assume no lateral support at all. Alternatively, it may be possible in some cases to consider the lateral supports as elastic restraints in some cases.

Bracing must not be overlooked; several failures in timber structures that have occurred in the past are the result of inadequate bracing against lateral instability. The engineer should also consider the construction stage carefully, i.e. when all the restraints that may eventually be arranged are not yet in place.

13.3 Bracing system for heavy timber structures

In order to understand the overall stability of a heavy timber structure, it may be useful to consider a simple industrial building. To facilitate the identification of the different bracing elements in the building, it may be appropriate to consider their location both on the "envelope" (longitudinal walls, end walls and roof) and with respect to the main directions of the building (longitudinal and transversal directions). For the considered building, four different bracing elements can be distinguished, *see figure 13.5*:

- Longitudinal wall bracing (A).
- Transversal roof bracing (B).
- End wall bracing (C).
- Longitudinal roof bracing (D).

There are different ways to place the bracing elements in a building; *figure 13.6* shows some of the most common ones. Assuming that the roof is stable in its plane, there are three general conditions concerning the location of the wall bracing elements that must be fulfilled in order to ensure the horizontal stability of a building:

- Wall bracings must be able to resist horizontal forces along three different directions in the plane.
- The three directions should not converge in the same point.
- At least two of the three directions should not be parallel one to another.

It is evident that the principal duty of bracing is the stabilization of the structure. In more specific terms, bracing has three main functions, namely:

- Transmission of horizontal loads, *see section 13.3.1, page 174*.
- Reduction of lateral deformations, *see section 13.3.2, page 176*.
- Enhancing buckling strength, *see section 13.3.3, page 177*.

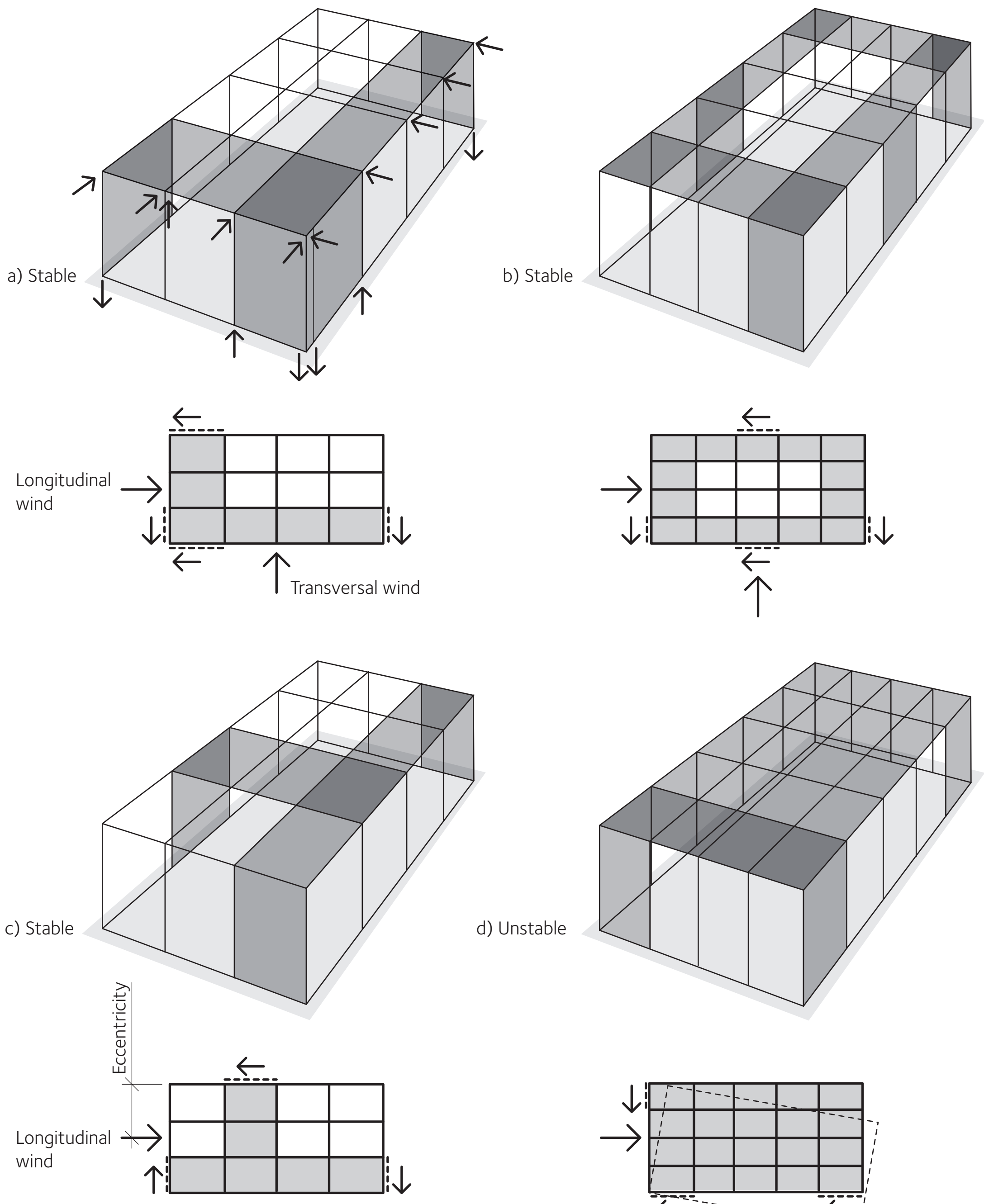
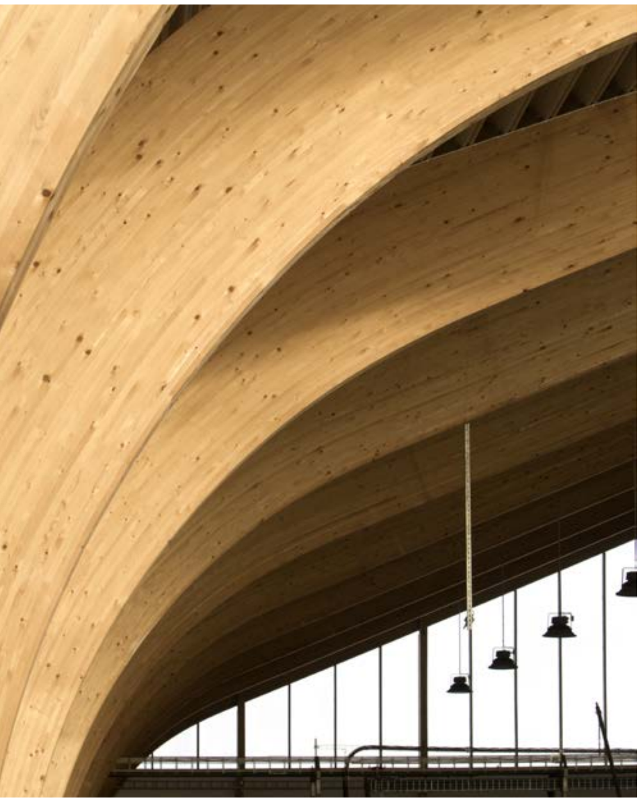


Figure 13.6 **Example of stable and unstable structures**



Cow shed, Lyrestad, Sweden.

13.3.1 Transmission of horizontal loads

Horizontal loads can act in any arbitrary direction of the plane, in contrast to gravity loads. Therefore, it is vital that the adopted bracing system is capable of resisting such loads. Regardless of the chosen structure, it is always necessary that for the bracing system to ensure a proper transmission of the horizontal forces to the foundations. It is also necessary to check that the transmission of horizontal forces between the bracing system and the other parts of the structure is secured by adequate connections between these members.

Horizontal loads perpendicularly to the end walls

In order to better understand the load path, consider a building subjected to wind pressure perpendicular to the end wall, *see figure 13.7*:

- The wind pressure is resisted by the end wall columns, which work as simply supported beams bent in their vertical plane. Two equal horizontal reaction forces are generated: one at the bottom of the column (foundation) and one at the top of the column which is resisted by the purlin.
- The purlins transmit the reaction force at the roof level (by compression) to the transversal roof bracing which works as a horizontal truss.
- The transversal roof bracing is loaded by the forces transmitted by the purlins, and it is supported by the two longitudinal wall bracings.
- The two longitudinal wall bracings work as trussed cantilevers; they transmit the reaction forces from the horizontal truss to the foundations. The forces transmitted to the foundation from each cantilever can be divided in one horizontal force and two opposite vertical forces.

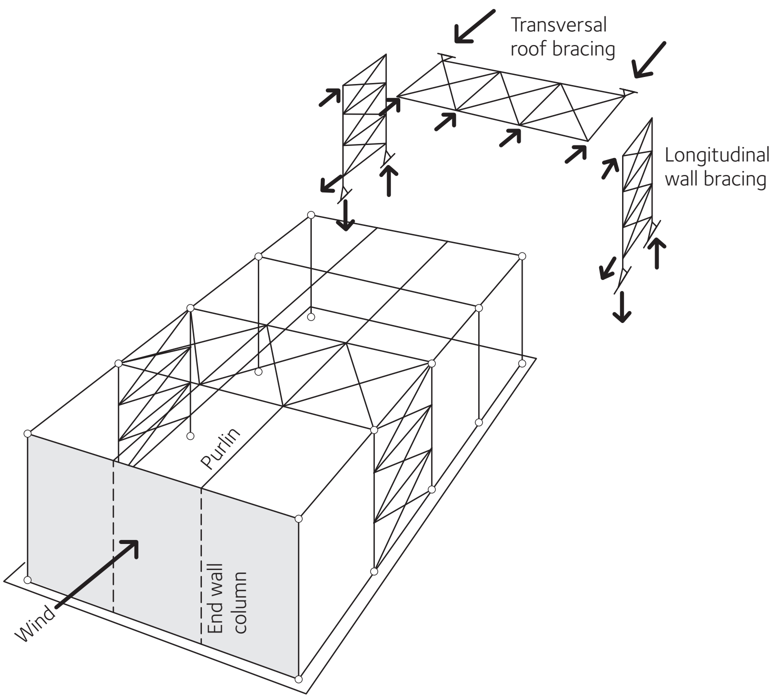


Figure 13.7 **Load resistance mechanism for horizontal load perpendicular to the end wall**

Purlins can act as web post of the horizontal truss; however, it is often preferred to design the truss with separate posts to get all the members in the same plane, thus avoiding eccentricity at the nodes.

Transversal roof bracing, together with longitudinal wall bracing, are normally placed in the same bay. A reduction of the number of braced bays often leads to more complicated erection of the structure, since temporary bracing devices would be required during construction. Braced bays with centre-to-centre distances not exceeding 30 – 40 m are therefore recommended.

Bracings are generally not placed at the bays closest to the end walls of the building. The design of nodes is then not affected by the end wall structure, where the design usually differs from that in the rest of the building.

Horizontal loads perpendicularly to the longitudinal walls

A similar load path as that described above occurs when the wind force is acting perpendicularly to the longitudinal wall of the building, *see figure 13.8*.

Portal frames, *see figure 13.9 b)*, and arches are stable in their own plane. Post-and-beam systems with columns rigidly fixed in the foundations are also stable in their own plane. On the other hand, post-and-beam systems with hinged columns must be stabilised by diaphragm action in the roof or by longitudinal roof bracing, *see figure 13.9 a)*. It should be pointed out that a bracing system according to *figure 13.9 a)* implies a greater complication of structural details. Moreover, such a structure needs temporary spatial bracing at all the columns until the erection of the building is complete.

The roof beams are often used as posts in the truss (longitudinal roof bracing), with some of the purlins as bottom and top chords. In this case, the purlins should not be designed as Gerber beams.

Riding school, Bökeberg, Sweden.

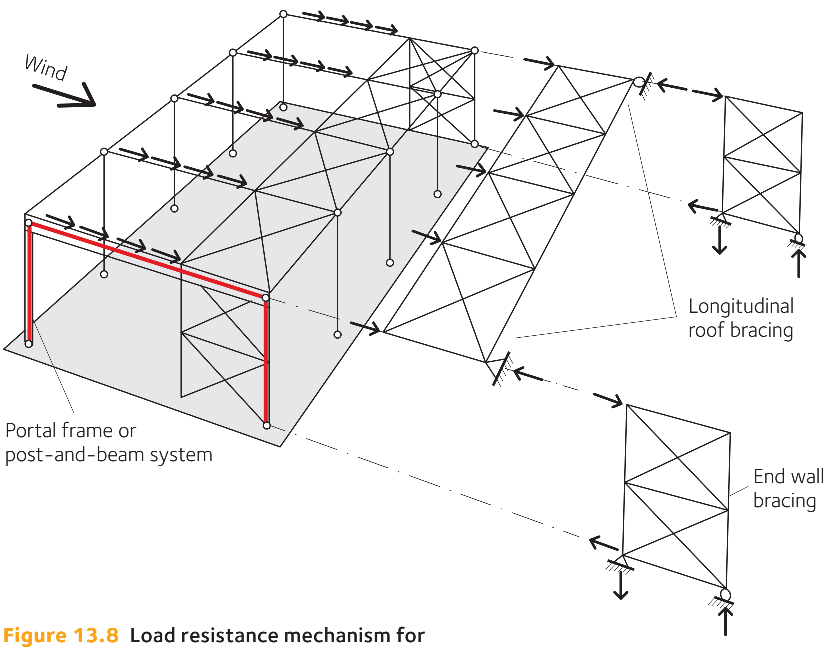


Figure 13.8 Load resistance mechanism for horizontal load perpendicular to the longitudinal wall

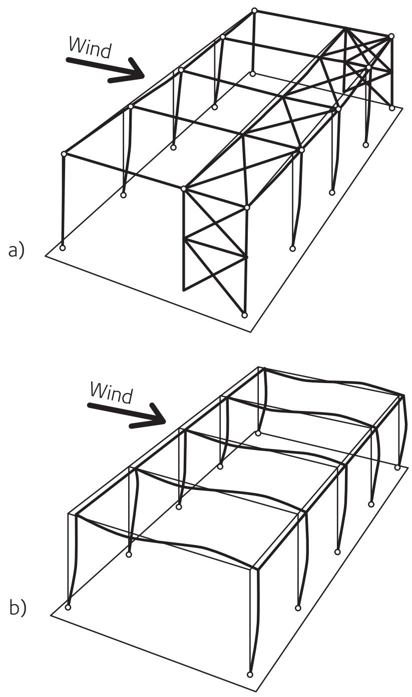


Figure 13.9 Stabilisation of
a) post-and-beam system,
b) portal frame system

Ulls hus, Uppsala, Sweden.

13.3.2 Reduction of lateral deformations

For serviceability reasons, lateral deformations of a structure should be kept below a given value, which depends on the use of the building, for example building height / 300. Moreover, as will be explained in *section 13.3.3, page 177*, over large lateral deformation may have a negative influence on buckling — due to the generation of second order effects.

As previously mentioned, if a post-and-beam system with pinned columns is adopted, the structure must be stabilised by a bracing system that consists of a longitudinal roof bracing (or roof diaphragm) and transversal wall bracings such as end wall bracings, *see e.g. figure 13.9 a) or figure 13.10 a)*. In this case, lateral displacement is resisted solely by the bracing system.

Portal frames or a post-and-beam system with columns rigidly fixed in the foundations, on the other hand, are stable in their own plane. In some cases, however, even though the structure is stable, lateral deformations due to horizontal loads may be too large. In order to reduce such deformations, two methods are normally adopted, *see figure 13.10 b)*:

- Increasing the cross-section of the columns.
- Introducing a longitudinal roof bracing.

If the building is too long and/or if the load-bearing capacity of the foundation soil is modest, it is often beneficial to fix the columns rigidly in the foundations. This will mean the horizontal load is equally taken by all the columns of the building. When a solution according to *figure 13.10 a)* is applied to a long structure, on the other hand, it would result in very large compression and tension forces in the members of the roof bracing (truss or stressed-skin panels), which may require strengthening of purlins and main beams. Moreover, such a solution would induce large uplift and compression forces at

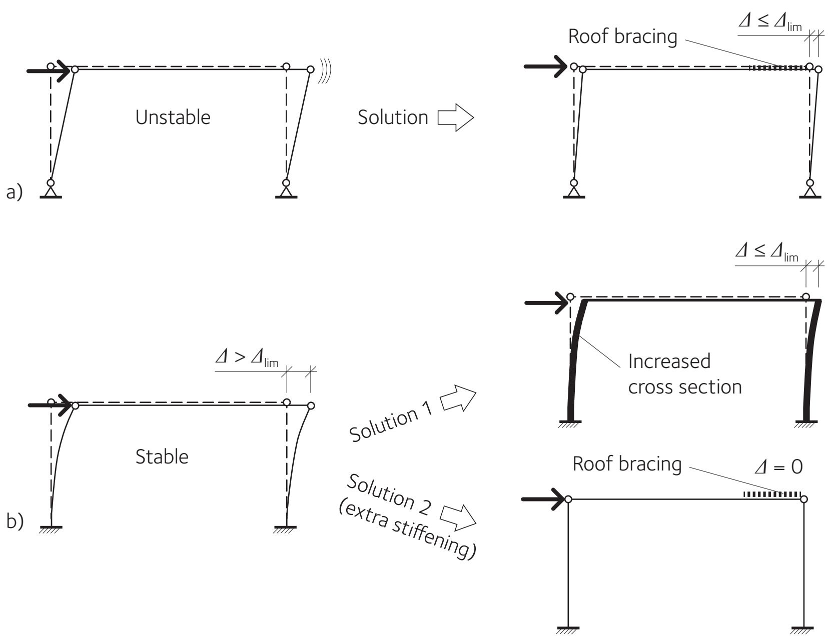


Figure 13.10 Effect of longitudinal roof bracing on the lateral displacement of post-and-beam structures
a) Pinned columns,
b) columns rigidly fixed in the foundations.

the supports of the end wall bracings, *see figure 13.8, page 175*, which will lead to expensive foundation structures. In this case, it may be appropriate to use an inner centre line to channel the bracing forces down from the roof.

Finally, it should be remarked that solutions with portal frames or post-and beam systems with columns rigidly fixed in the foundations are preferable in cases of possible future extension of the building. Such solutions in fact, in contrast to the post-and-beam system with pinned column, do not require reinforcement of the existing bracing system if the building requires extension.

13.3.3 Enhancing buckling strength

Besides its function of transmission of horizontal loads to the foundations and limitation of lateral deformations, bracing — both horizontal and vertical — may have a beneficial effect in reducing the buckling length of those elements of the structure that are subjected to compressive stresses.

In the particular case of a post-and-beam structure with columns rigidly fixed in the foundations, the presence of longitudinal roof bracing considerably reduces the lateral displacements at the top of the columns, thus decreasing their in-plane buckling lengths, *see figure 13.11*. From the point of view of in-plane buckling, the stiffness of the roof bracing is in practice large enough to allow for the assumption that the top of the columns do not move laterally. However, in reality, the bracing works as a series of elastic supports at the top of the columns, *see figure 13.11 b)*.

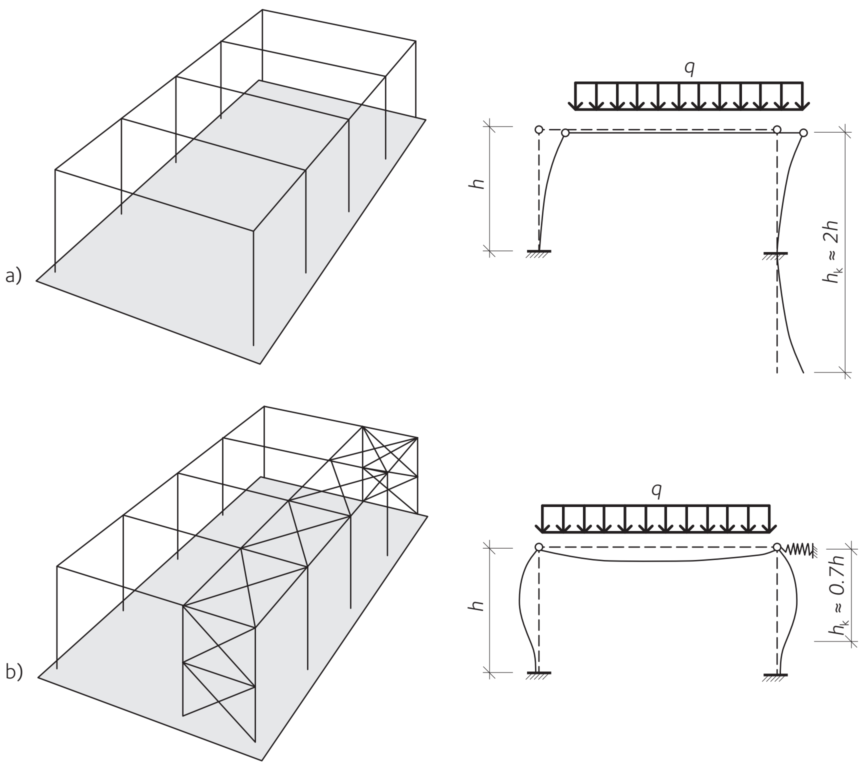


Figure 13.11 Effect of longitudinal roof bracing on the buckling length of post-and-beam structures with columns rigidly fixed in the foundations
a) Without lateral bracing,
b) with lateral bracing.

Transversal bracing systems, i.e. transversal roof bracing and longitudinal wall bracing, also have a positive effect in reducing the buckling length of structural members which are subjected to compressive stresses. *Figure 13.12* shows how the presence of a transversal bracing system influences both the lateral-torsional buckling of main beams and the out-of-plane buckling of columns.

With reference to the main beams of a structure, it should be noted that the points of restraint to lateral buckling are only those connected to the nodes of the truss, which constitutes the transversal roof bracing. Therefore, a bracing system with only a few diagonals (which means also: "with a few truss nodes") may not be suitable for a proper bracing of the main beams against lateral buckling. *Figure 13.13* illustrates how the critical buckling length l_k can be reduced by increasing the number of diagonals in the transversal roof bracing.

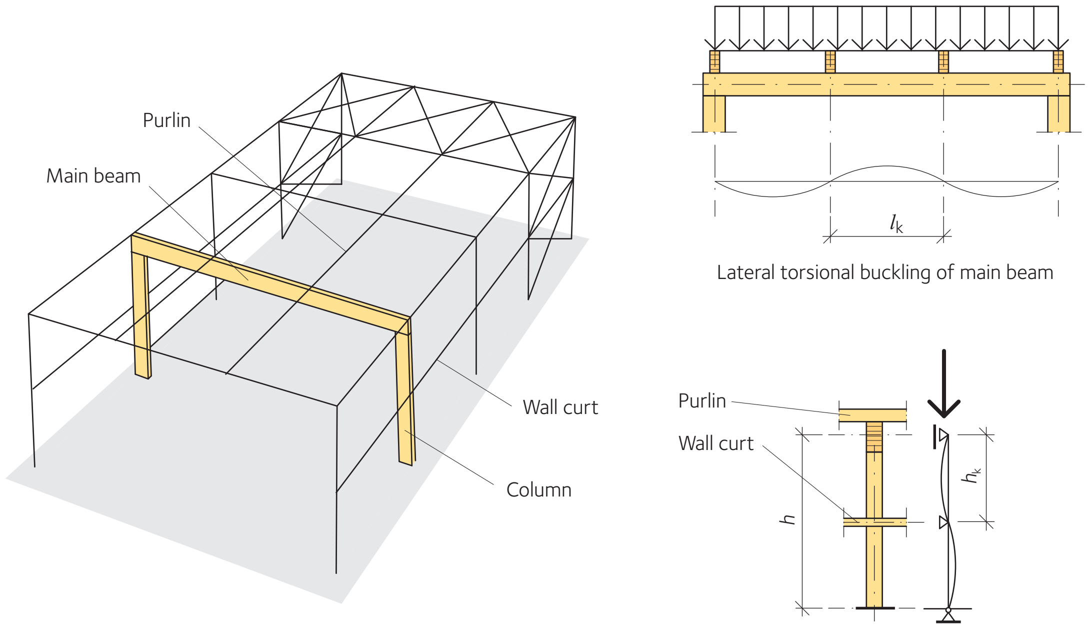


Figure 13.12 Effect of transversal bracing of the roof on the lateral-torsional buckling of a main beam and on the out-of-plane buckling of a column

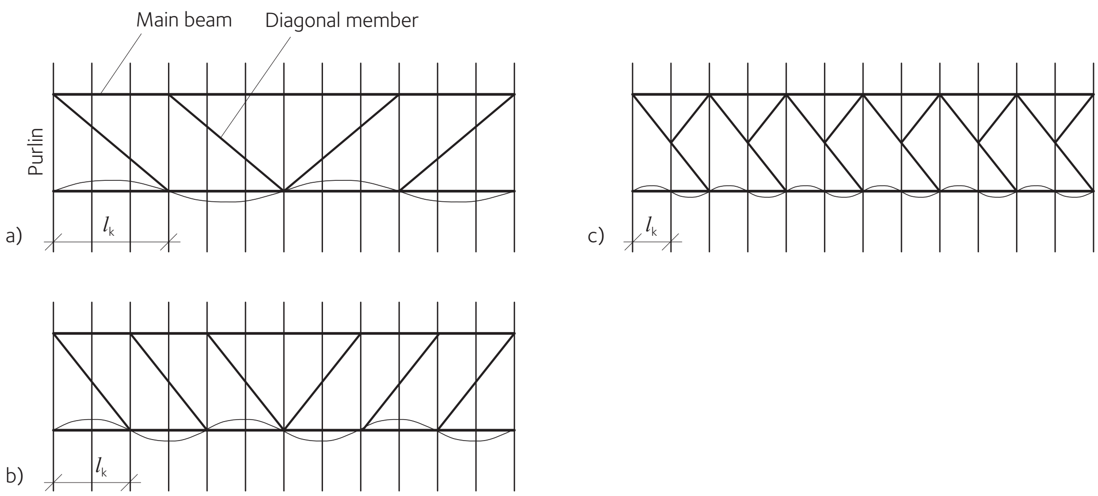


Figure 13.13 Effect of type of transversal roof bracing on the critical buckling length with respect to lateral-torsional buckling of the main beam

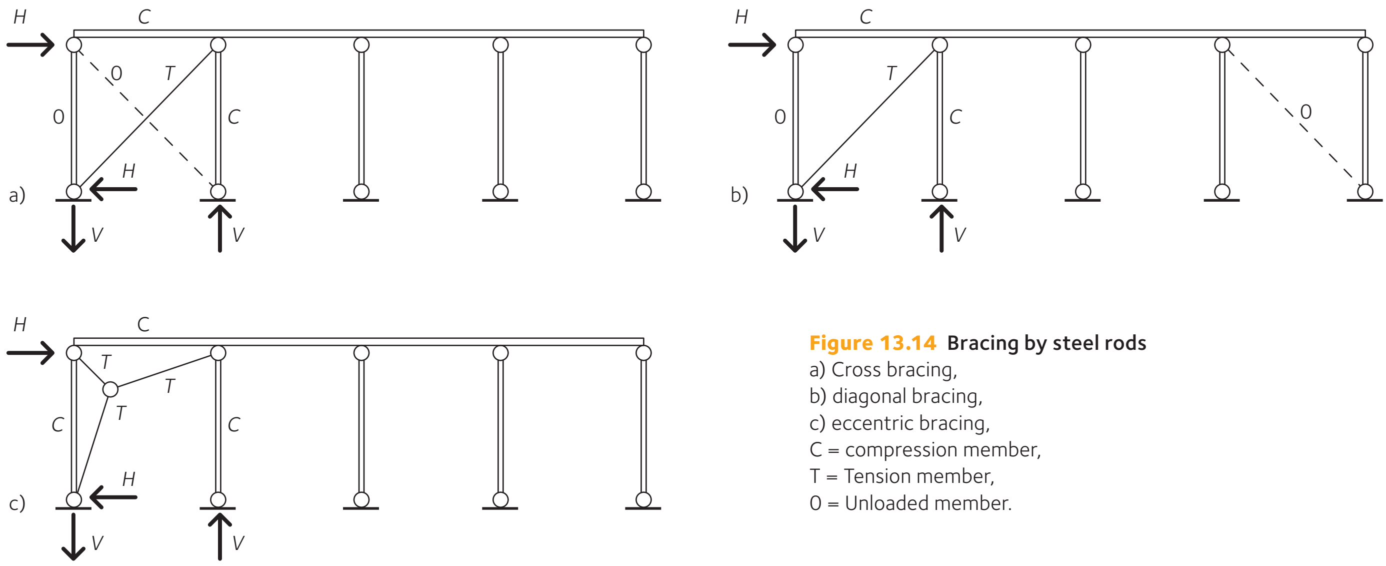


Figure 13.14 Bracing by steel rods
a) Cross bracing,
b) diagonal bracing,
c) eccentric bracing,
C = compression member,
T = Tension member,
0 = Unloaded member.

13.3.4 Wall bracing

Braced bays are located such that they have minimum impact on the structural layout. Moreover, the location of such wall bracings is governed by the manner in which the building is to be erected and the distribution of horizontal forces in the structure.

Wall bracing is normally achieved by means of:

- Threaded steel rods.
- Compression diagonal struts (normally timber members with nearly square cross section).
- Frames.
- Shear walls.

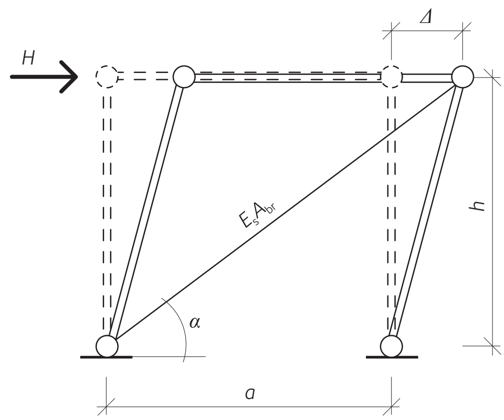


Figure 13.15 Model for estimation of lateral stiffness of a bay braced by diagonals

Bracing by means of steel rods

Braced-bay systems by means of threaded steel rods comprise diagonal, cross and eccentric bracing arrangements, *see figure 13.14*. The advantage of such systems is that the bracing elements are subjected only to tension. Consequently, the members are relatively light, providing a very stiff overall structural response. The tension members can be easily retensioned by means of turnbuckles. This enables the vertical members to be aligned exactly, thus compensating for dimensional tolerances.

The system of eccentric bracing is generally less stiff than an equivalent system using cross-bracing or diagonal bracing.

Disregarding possible slip at the connections and axial deformation of timber members, the stiffness of a braced bay k_{br} (= H/Δ) can be expressed as:

$$k_{br} = \frac{E_s \times A_{br} \times \cos^3\alpha}{a} \qquad 13.1$$

where the terms of the equation are illustrated in *figure 13.15*.

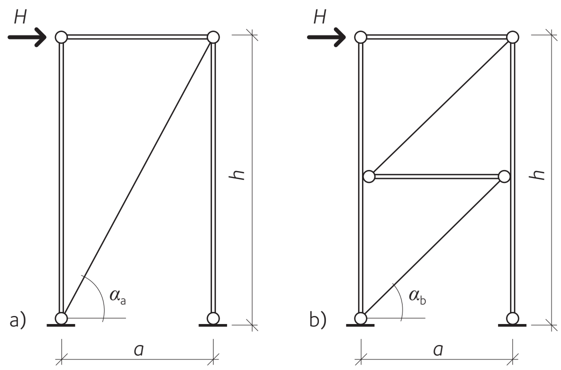


Figure 13.16 **Model for estimation of lateral stiffness of a bay braced by diagonals**

It is evident, from *equation 13.1, page 179*, that when decreasing the angle of inclination of the diagonal brace element, the stiffness and thus the efficiency of the bracing system will improve. Normally, both a and h are specified for a given structure. Therefore, the only way to reduce the angle α is to increase the number of braced levels. The choice of number of braced levels is a compromise between economy and efficiency. For a bracing structure with a given width a and a given height h, for example, two solutions are possible, namely:

- One single diagonal bar, *see figure 13.16 a)*.
- Two or more diagonal bars, placed on two or more levels, *see figure 13.16 b)*.

The choice of one bracing level instead of two (or more) bracing levels would imply a larger angle α, thus a reduced efficiency of the bracing system. Therefore, if a given stiffness of the system, k_{br}, is required, the diameter of the steel diagonal member for the solution of *figure 13.16 a)* must be larger than that used in the solution of *figure 13.16 b)*. On the other hand, solution *b)* requires a larger number of members and connections than solution *a)*. From the point of view of economy, experience has shown that the number of bracing levels should be such that the angle of inclination of the diagonal bars is within the range $45° < \alpha < 60°$.

Bracing by means of timber members

Braced-bay systems by means of timber members comprise diagonal, and 'K' bracing arrangements, *see figure 13.17*. Where a single diagonal brace is used, it must be capable of resisting both tensile and compressive axial forces to allow for the alternating direction of wind load. A shortcoming of such a system is that the diagonal members are in general not adjustable in length and therefore require higher precision during erection than similar bracing systems with steel threaded bars.

Bracing by means of frames

Buildings where large door and window openings are required can be braced by means of frames, which allow for larger clear spaces where otherwise there would be awkward diagonal members, *see figure 13.18*. The frame can be fabricated from timber or, in case of very large horizontal load, from steel.

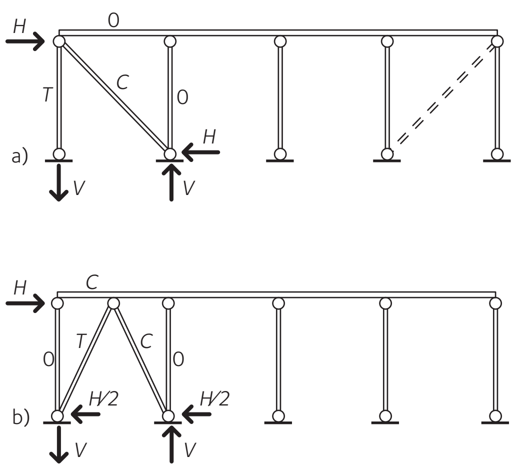


Figure 13.17 **Bracing by timber members**
a) Diagonal bracing,
b) K-bracing.
C = compression member,
T = Tension member,
0 = Unloaded member.

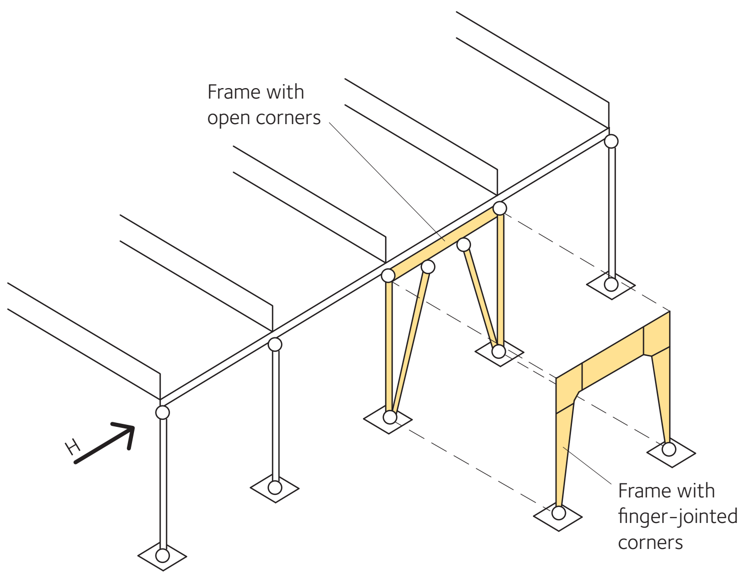


Figure 13.18 **Bracing by frames**

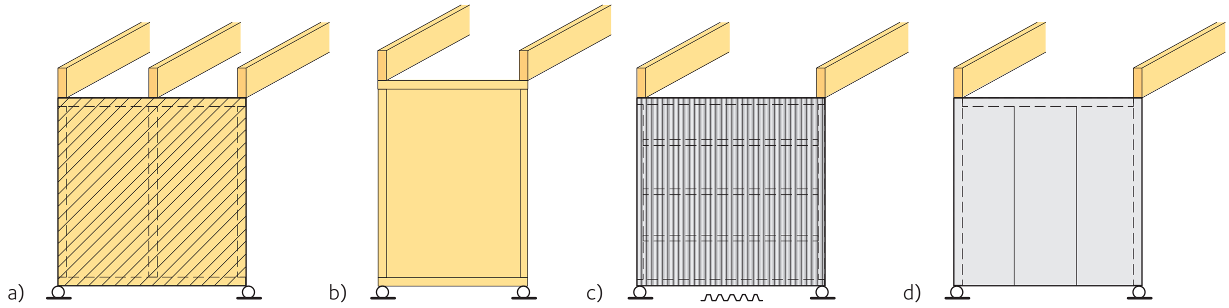


Figure 13.19 **Bracing by shear walls**
a) Diagonal planks,
b) wood-based panels,
c) metal sheeting,
d) concrete panels.

Bracing by means of shear walls

Shear walls can be fabricated from wood-based panels such as cross-laminated timber (CLT), plywood, oriented strand boards (OSB) or cross-glued laminated veneer lumber (LVL). Even diagonally positioned planks or boards can be used. It is also possible to manufacture shear walls made from trapezoidal metal sheeting, concrete panels, etc., *see figure 13.19*. When shear walls are adopted, the possibility of openings in the wall is limited.

Some fixing details for wall bracing

Figure 13.20 illustrates some possible fixing details appropriate for the connection of the bracing diagonal to the concrete foundation.

Note that both systems shown in the examples can be easily tensioned by means of either a turnbuckle (for the steel rod) or tightening nuts (for the timber strut), which enables the vertical members to be correctly aligned.

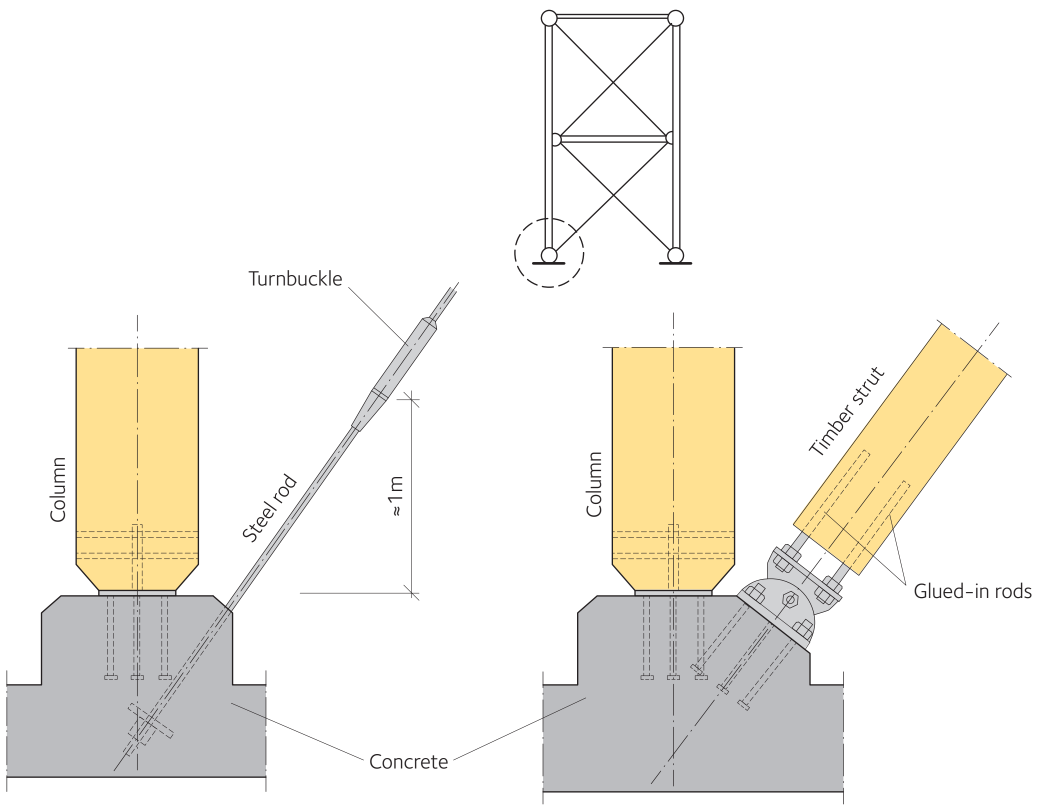


Figure 13.20 **Examples of connections of different bracing diagonals to the concrete foundation**

Figure 13.21 illustrates two examples of fixing details appropriate for the connection of the bracing diagonal to timber a column.

Figure 13.22 illustrates two examples of fixing details appropriate for the connection at the intersection between two steel rods.

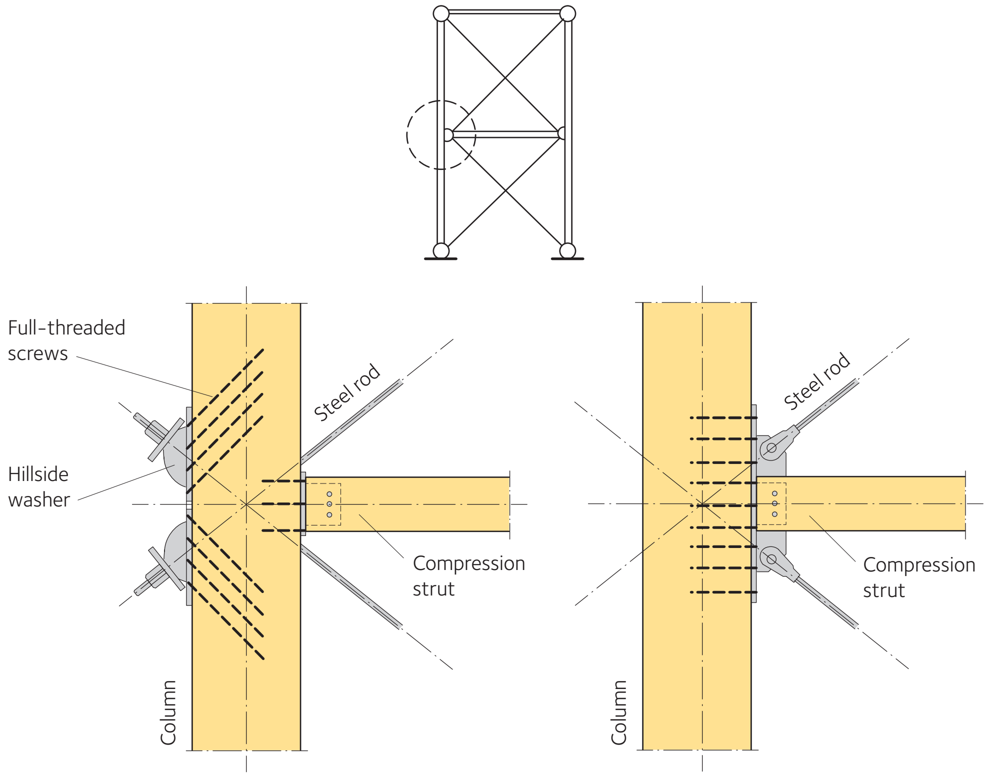


Figure 13.21 Examples of connections of steel diagonals to a timber column

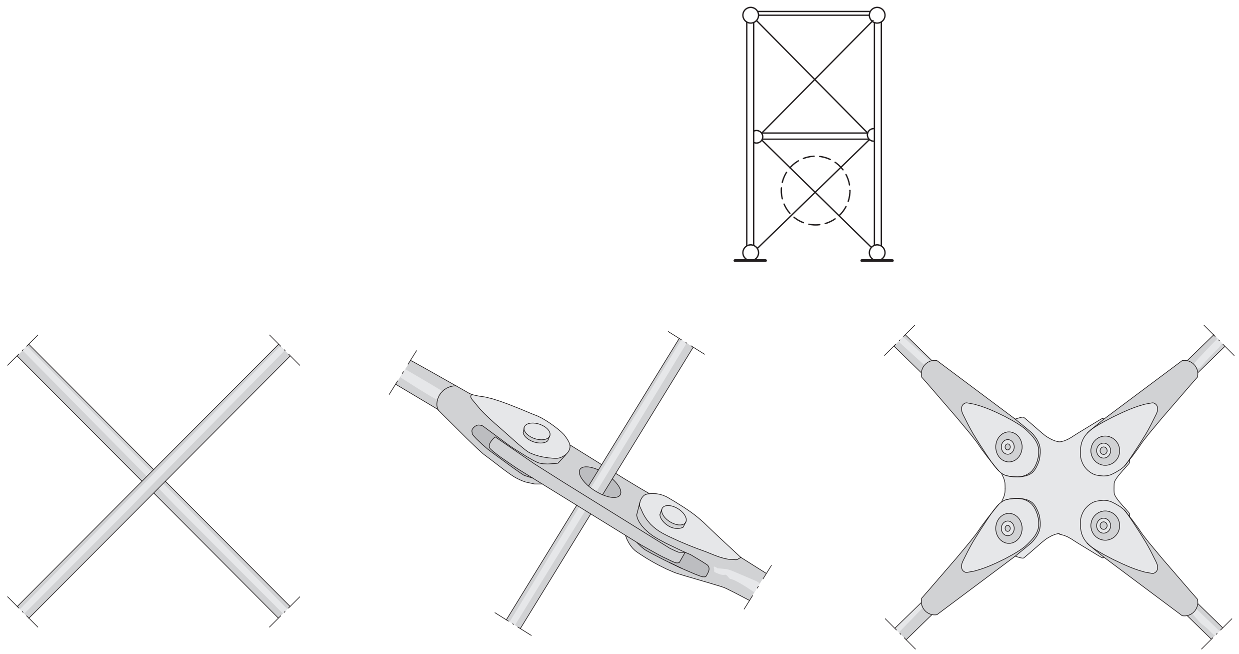


Figure 13.22 Connection at the intersection between steel rods

13.3.5 Roof bracing

To be able to take up horizontal loads, roofs of heavy timber structures are normally stabilised in accordance with one of the following alternatives:

- Horizontal trusses in the roof
- Diaphragm action of the roof by means of wood based panels or corrugated metal sheeting.

Bracing by means of horizontal trusses

In this section, only transversal roof bracing (type "B", *see figure 13.5, page 172*) will be treated. Longitudinal roof bracing (type "D", *see figure 13.5, page 172*) is seldom used in modern large timber structures; it is often preferable to achieve lateral stability of the structure by creating moment-stiff connections at the bases of each columns.

In smaller buildings (say with a longitudinal dimension smaller than 30 – 40 m), only one horizontal truss in the roof can be sufficient. The purlins and also the connections between purlins and main beams must then be able to transfer compression and tension forces from the other end, *see figure 13.23*.

In longer buildings, on the other hand, it can be suitable (not least with regard to stability during erection), to arrange two or more horizontal trusses in different bays within the building. At the ends, the wind truss can be placed in the second bay from the end. The design of nodes is then not affected by the end wall structure (gable), which usually differs from that in the rest of the building.

The diagonals of the horizontal truss are normally made either of steel rods or timber elements. Steel rods are in general preferable, due to the fact that they can be easily retensioned by means of turnbuckles, thus enabling an exact aligning of the main beams.

Depending on the chosen diagonal arrangement, the bracing system can act in one or two directions. For example, bracing systems with only one steel diagonal in each web panel can only work if the load is applied in the direction that causes tension in the diagonals. In such a case, if two braced bays are used, the horizontal load

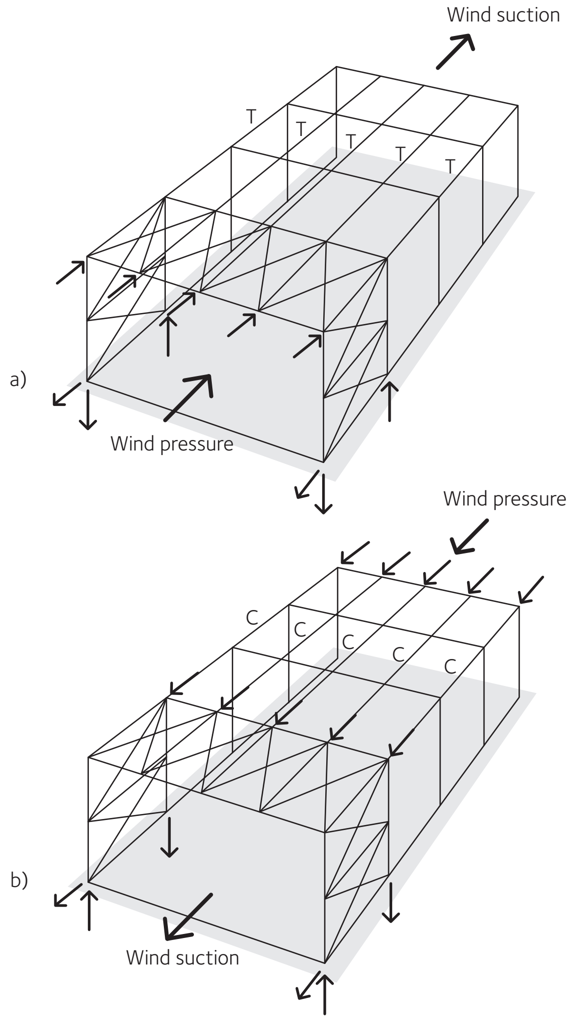


Figure 13.23 Bracing of a small building
T = purlin loaded in tension,
C = purlin loaded in compression.

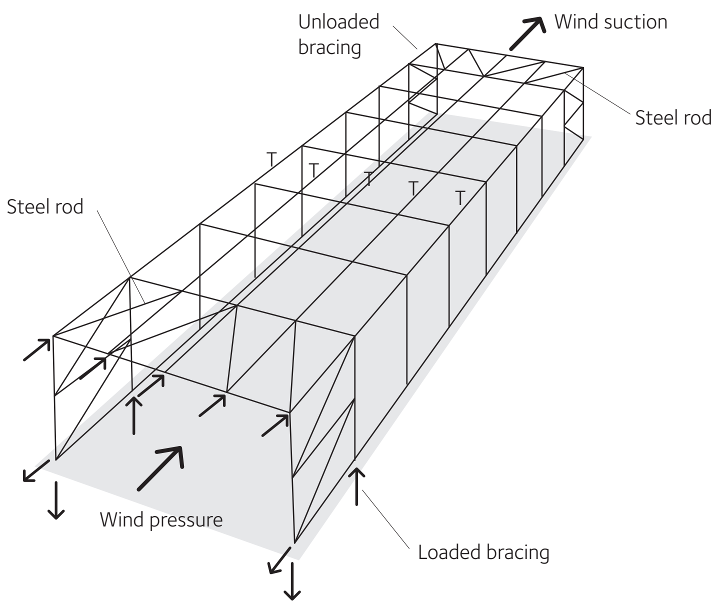


Figure 13.24 Example bracing systems with only one steel diagonal in each web panel. Such a system is "unidirectional", which means that it can only work if the load is applied in the direction that causes tension in the diagonals.

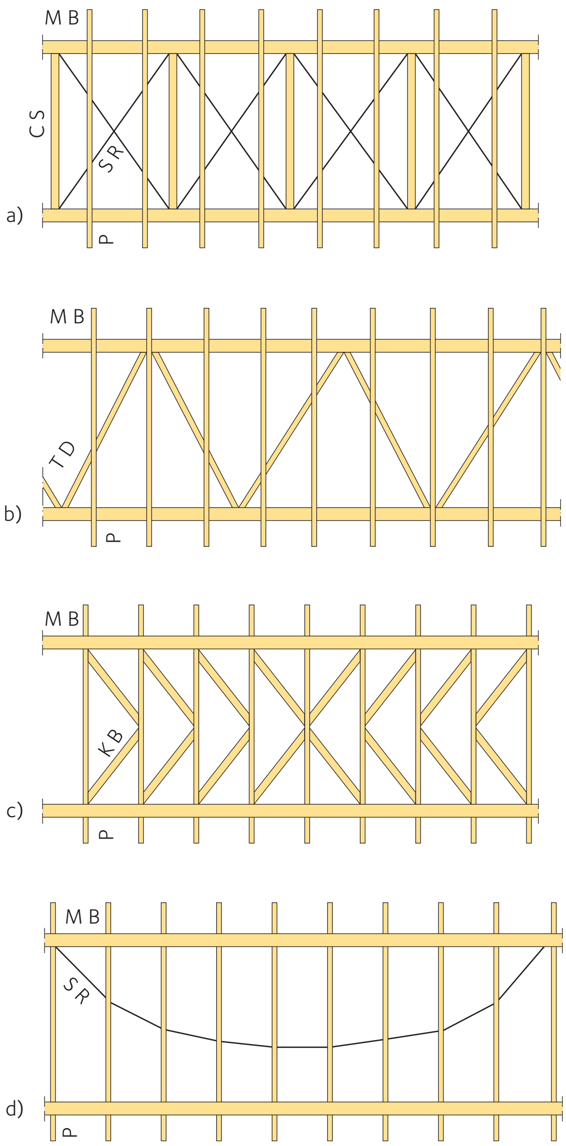


Figure 13.25 **Examples of roof bracing systems by means of different horizontal truss types**
M B = Main Beam
P = Purlin,
S R = Steel Rod,
T D = Timber Diagonal,
C S = Compression Strut (timber),
K B = K-Bracing element (timber).

can only be taken by the outermost horizontal truss that is closest to the point of application of such a load, *see figure 13.24, page 183*. In this case therefore, besides bending action caused by vertical loads, purlins also must be designed to resist tensile forces. On the other hand, bracing systems e.g. with cross diagonals made of steel rods, are able to take load both in tension and compression. *Figure 13.25* illustrates some examples of roof bracing by means of different types of horizontal trusses. During horizontal loading, the two main beams of the bracing systems a), b) and c), will act as upper and lower chord of the horizontal truss. In system d), on the other hand, the main beam, the purlins and the steel rods (or alternatively steel wires) will act as a “fish-belly” beam. In such a case the load-bearing mechanism during horizontal load is:

- Bending (around the main beam weak axis) and compression in the main beam.
- Tension in the steel rods or wires.
- Compression in part of the purlins.

The bracing system of *figure 13.25 a)* includes cross diagonals constructed of steel rods. Due to their great slenderness, and consequent proneness to buckling, only the diagonal members subjected to tension will be active in taking horizontal loads. The “vertical” elements of the truss are normally made of timber and are subjected to compression.

The bracing system of *figure 13.25 b)* includes diagonal timber members positioned in a “V” fashion. The main beams and the diagonals form a Warren-type truss. Each pair of adjacent diagonals is subjected to tension and compression respectively, which can alternate depending on the direction of the horizontal load.

The bracing system of *figure 13.25 c)* includes diagonal timber members arranged in a “K” fashion. This system has the advantage of creating closely spaced brace points, thus increasing the lateral-torsional buckling resistance of the main girders.

The bracing system of *figure 13.25 d)* obviously works only for one direction of the applied horizontal load, i.e. the direction that generates tension in the steel rods. Therefore, two opposite systems are necessary to guarantee the stability of the roof in the longitudinal direction.

Bracing by means of roof diaphragm action

Diaphragm action of the roof can be achieved by plates made from wood-based products or trapezoidal profile metal sheeting. However, it is difficult to achieve the diaphragm action with corrugated steel over time in unheated buildings, since the fasteners tend to lose stiffness in fluctuating temperatures. The main function of a roof diaphragm is similar to that of the web of a plate girder, i.e. to resist shear stresses.

A diaphragm can be visualised as the thin web of a large "plate girder" (or I-girder) that is formed by the roof, primarily resisting shear, while the boundary members — i.e. edge beams or walls perpendicular to the load direction — act as the plate girder flanges by carrying the moment in compression, N_c, and tension, N_T, respectively.

Where $L \leq 2s/3$, the magnitude of the compression and tension forces in the edge beams is:

$$N_C = N_T = \frac{q \times s^2}{8 \times L} \quad 13.2$$

All of the bending moment is assumed to be taken by the edge beams, and they must consequently be continuous or detailed to be able to transfer the tensile or compressive forces to adjacent sections.

The roof skin transfers the shear to the vertical braced frames (shear walls, diagonal bracing or rigid frames). The maximum shear that the roof diaphragm must carry is:

$$V = \frac{q \times s}{2} \quad 13.3$$

All of the shear must be taken by the panel material. The shear stress is higher closer to the edges of the diaphragm. The shear flow, v, (N/mm), which the diaphragm and its connections must be designed for is:

$$v = \frac{V}{L} \quad 13.4$$

This means that the force in individual fasteners connecting the sheeting to the frame must be designed for a force $F_v = v \times s_f$, where s_f is the spacing between the fasteners. Similarly the sheeting panel must be designed for an in-plane shear stress $\tau = v/t$, where t is the thickness of the panel.

Obviously, the mathematical interpretation of the diaphragm behaviour under lateral loads is an oversimplification, since the degree of flexibility or rigidity is highly indeterminate.

Timber warehouse, Sundsvall.

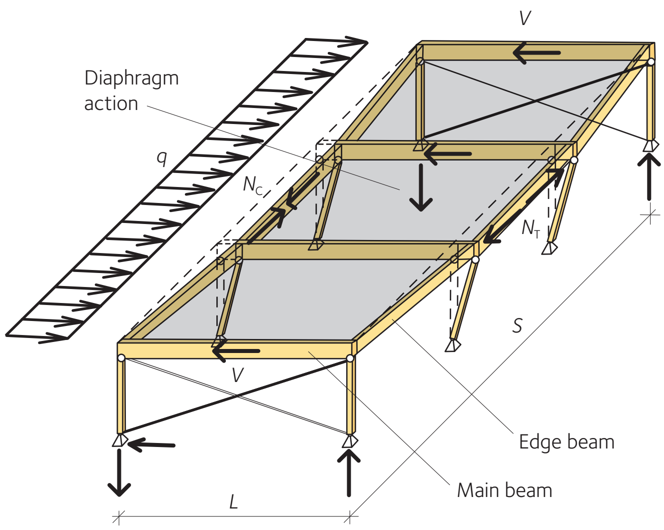


Figure 13.26 **Diaphragm action of the roof**

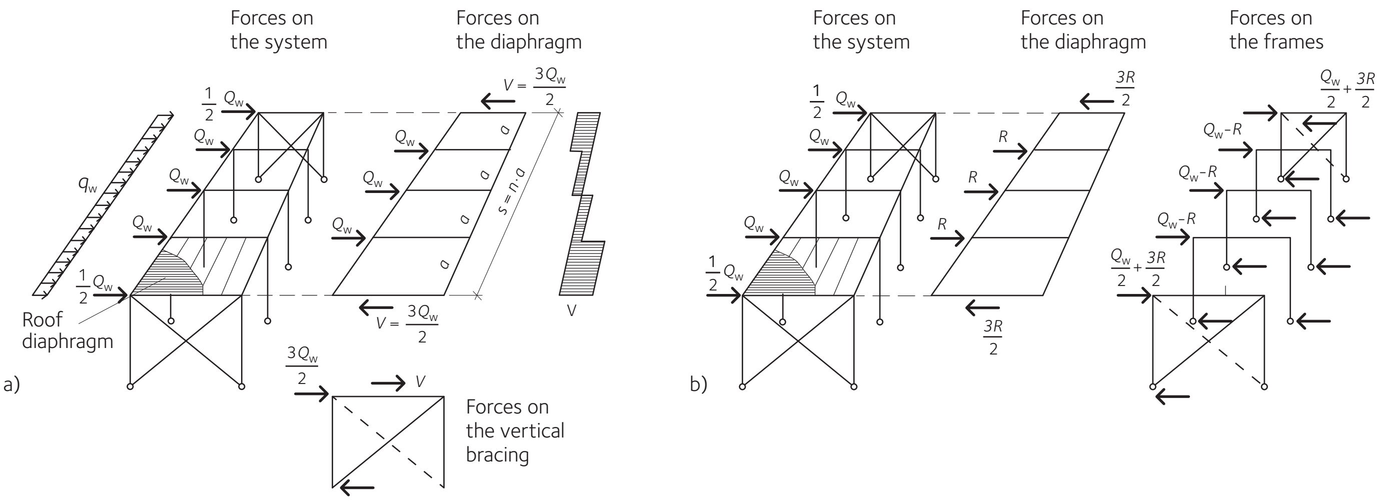


Figure 13.27 Load sharing mechanisms
a): Post and beam system with hinged column bases,
b): frame system.

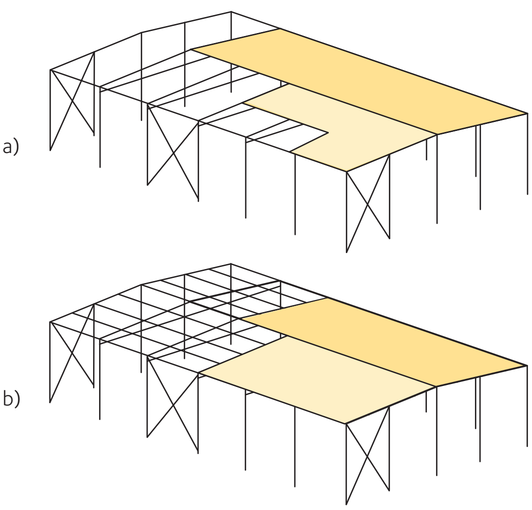


Figure 13.28 Roof diaphragm arrangements
a): Diaphragm directly fastened on the main beams.
b): Diaphragm fastened on the purlins.

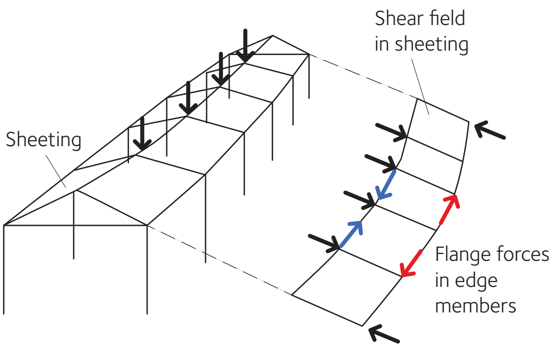


Figure 13.29 "Deep plate girder action" in pitched roof sheeting

There are some general requirements that should be fulfilled when utilizing roof diaphragm action, namely:

1. The end walls must be braced.
2. The roof sheeting must be properly fastened to underlying members with proper connections.
3. The seams (or side laps) between sheets must be fastened with proper connections.
4. Possible roof openings should be less than 3 percent of the roof area unless detailed analyses are made, in which case 15 percent is allowed.

Frames and roof diaphragm will always interact, which significantly affects the overall behaviour of a complete building. If the load-bearing system of a structure consists of a series of posts and beams, with posts hinged at their bases, the horizontal load q_w at the roof level is entirely taken by the roof diaphragm, *see figure 13.27 a)*. On the other hand, if the load-bearing system consists of a series of frames or post and beams with posts fixed at their bases, q_w will be shared between the frames (or the posts) and the diaphragm, *see figure 13.27 b)*.

Typical arrangements for a roof diaphragm are shown in *figure 13.28*. Whenever possible, each panel of sheeting should be fastened to all four edge members to get greater strength and stiffness. Sheeting fastened only to the purlins is also permissible, provided that the end panels of the sheeting are fastened to the end walls by special arrangements.

In a pitched roof building, *see figure 13.29*, under vertical load, frames cannot spread away without mobilising "deep plate girder action" in roof sheeting. This means that the roof sheeting together with the edge beam will absorb part of the applied vertical load. The steeper the roof pitch, the more effective the diaphragms are in resisting vertical load.

Typical roof sheeting systems

Diaphragm action of the roof is commonly achieved by panels made from:

- wood-based products or
- trapezoidal profile metal sheeting.

Wood-based panels include sheeting material such as plywood, OSB (oriented strand board), cross-glued LVL (laminated veneer lumber) or CLT (cross-laminated timber). Since the span (centre-to-centre distance) between main beams/rafters is relatively large — generally greater than 4 m — the wood-based panels often need to be reinforced by means of ribs. The ribs are normally screwed and glued to the panel, either on one side or — for larger spans — on both sides. Typical dimensions of such systems are: depth h = 300 – 800 mm, width b = 1,800 – 2,500 mm, span s = 5 – 18 m, *see figure 13.30*.

Metal sheeting with trapezoidal profile is often used as roof-bracing system in Scandinavian countries such as Sweden and Norway. The sheeting material consists of cold-rolled steel plates with sheet thickness normally between 0.6 – 1.2 mm. For load-bearing sheets, the profile depth varies between 45 mm for short spans to 200 mm for very long spans. The yield strength of the steel material is generally in the range 350 – 500 MPa. In order to increase the bearing capacity for vertical loading, the sheets are sometimes provided with grooves.

Generally, trapezoidal metal sheeting systems are constructed so that they are continuous over three or more supports (i.e. main beams or rafter). Continuous metal sheeting systems can in turn consist of:

- Sheeting with hinged joints at the support,
- Sheeting with hinged joints in the span, so called “Gerber system”,
- Sheeting with overlapping over supports

These different systems are illustrated in *figure 13.32*.

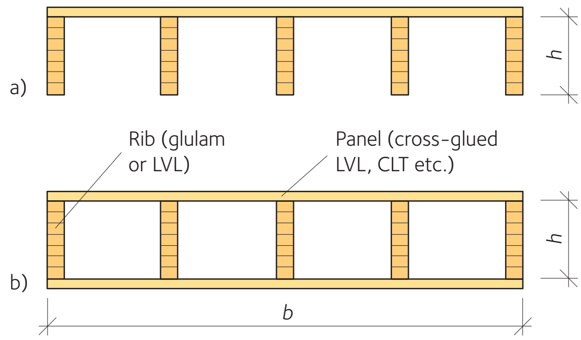


Figure 13.30 **Typical cross-section of wood-based roof systems**
a): Open cross-section,
b): box cross-section.

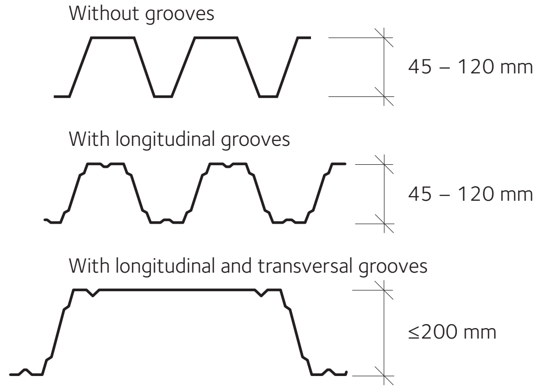


Figure 13.31 **Typical cross-section of trapezoidal metal sheeting**

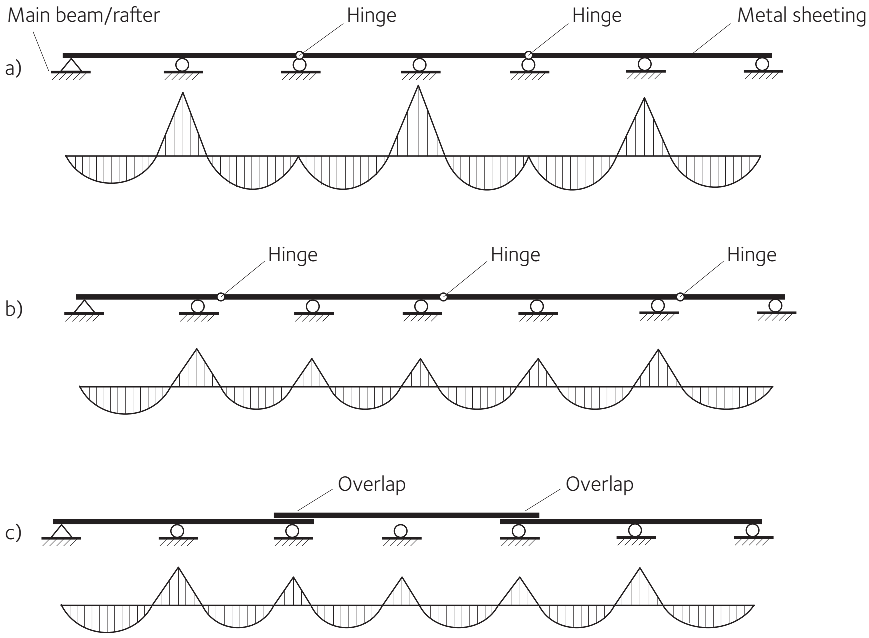


Figure 13.32 **Different joint systems for trapezoidal metal sheeting with corresponding bending moment diagram for uniformly distributed load**
a) Hinged joints at the support,
b) Gerber system,
c) overlap system.

Cow shed, Lyrestad, Sweden.

Overlapping sheeting systems have the advantage of a doubled carrying capacity over the supports, where the moments are largest. The overlap is made sufficiently long to reduce the maximum bending moment in each sheet to about half; the sagging moment then becomes critical. "The Gerber system" is designed so that positive and negative moments are equal. To reduce the risk of progressive collapse, the system should be designed so that every second bay is free from hinges.

Some fixing details for roof bracing

Figure 13.33 illustrates some possible fixing details appropriate for the connection of the bracing members to the main beams/rafter.

Figure 13.34 shows a typical detail of connections between a trapezoidally corrugated metal sheeting and a timber beam.

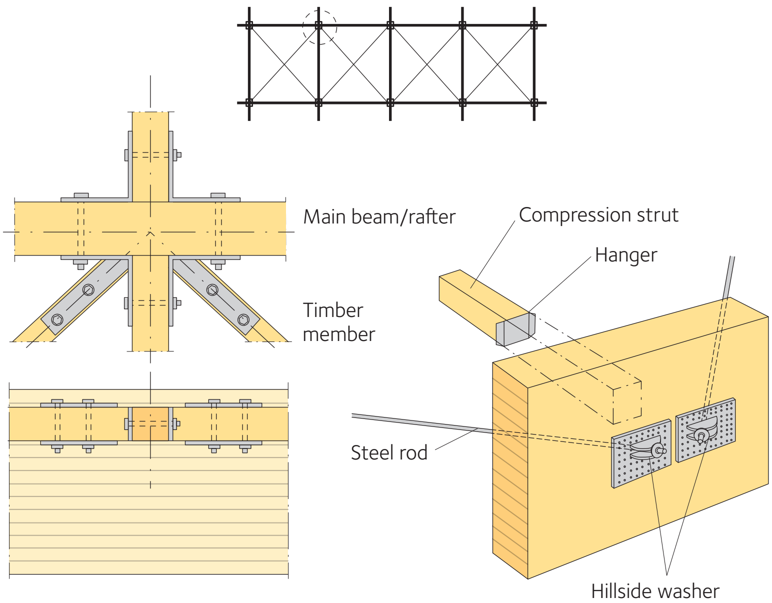


Figure 13.33 **Examples of connections of different bracing members to the main beam**

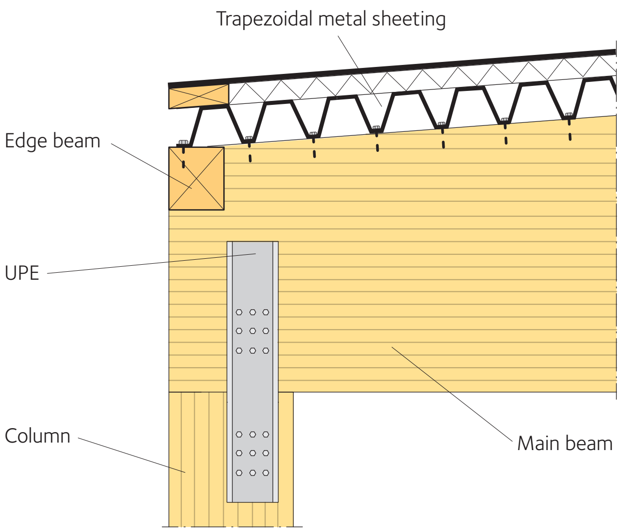


Figure 13.34 **Example of connections of trapezoidally corrugated metal sheeting to the main beam**

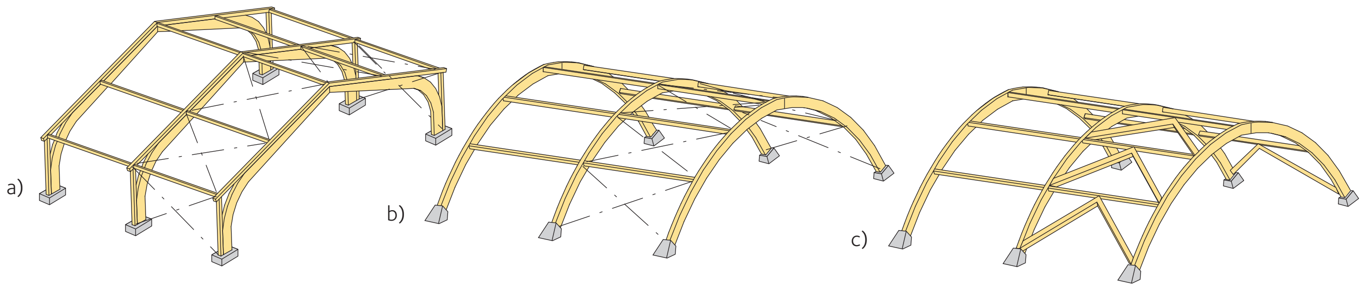


Figure 13.35 **Example of frame and arch bracing**
a) Bracing by steel rods,
b) bracing by steel rods,
c) bracing by timber diagonals.

13.3.6 Bracing of frames and arches

Frames and arches – regardless of whether they are statically determinate or indeterminate – are basically stable in their own plane. They must, however, be braced in the longitudinal direction.
The principle of frame and arch stabilisation by means of transversal bracing is shown in *figure 13.25*. The frame/arch members are often used as bottom and top chords in the truss, with some of the purlins as compression struts. Only diagonals are added as stiffening, often crosses of steel rods, *see figure 13.25 a) and (b)*, or sometimes timber diagonals, *see figure 13.25 c)*. The joints between the various components forming part of the wind truss must be designed with regard to the forces that arise, and the eccentricity. Truss posts and chords are often in different planes.

13.4 Strength and stiffness requirements for bracing systems

The main function of lateral bracing is to provide lateral support to a member in order to prevent it from moving laterally at the bracing position. Normally, the same bracing elements that are used to prevent lateral movements due to external transversal loading also serve to increase the buckling strength of the primary members, such as beams and columns.

13.4.1 Requirements for bracing of columns

For the idealized case of perfectly straight members with full bracing there is no force in the braces even at buckling because there is no displacement at the brace point. However, in real members, brace forces do develop during loading. As an example consider a series of hinged columns, each subjected to a vertical load, P, laterally braced by a stabilising system. Due to inevitable out-of-plumb of the columns, horizontal forces, F_{br}, are generated at the top of each column. The bracing system, therefore, must be strong (and stiff) enough to counteract the resultant of these forces. **Note** that normally it is conservatively assumed that all columns lean toward the same side, *see figure 13.36, page 190*.

Bridge, Virserum, Sweden.

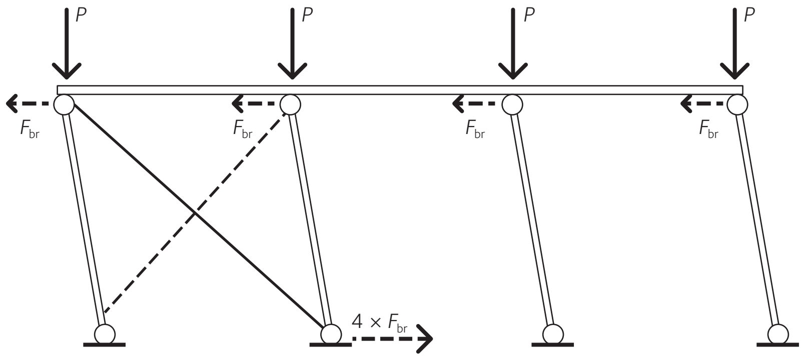


Figure 13.36 Brace force generated by the out-of-plumb of the columns

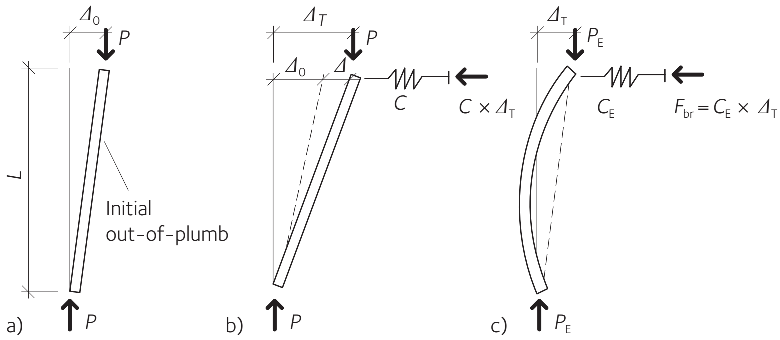


Figure 13.37 Relative bracing system

A simple design formulation is to assume that the brace force, F_{br}, is approximately 1–2 percent of the compressive force, P. However, such a simple criterion — without any specification concerning the required stiffness — might not be sufficient for a proper design of the bracing system. In fact, if the bracing system is too flexible, lateral displacements — and thus the brace force — may grow into an unacceptable size. Consider, for example, the relative bracing system shown in *figure 13.37*, where the brace is represented by the spring with stiffness, C, at the top of the column.

For the idealized case of a perfectly straight member, it can be shown by simple equilibrium considerations that the so called "ideal spring stiffness" C_E, i.e. the spring (or brace) stiffness that is required to reach the corresponding Euler buckling load, P_E, according is:

13.5 $$C_E = \frac{P_E}{L}$$

For the out-of-plumb column, the relationship between P, C and Δ_T is plotted in *figure 13.38 a)*. If $C = C_E$, P_E is reached only if the sway deflection gets very large. Unfortunately, such large displacements produce large brace forces F_{br}, since $F_{br} = C \times \Delta$. For practical design however, Δ must be kept small. This can be accomplished by specifying $C > C_E$, e.g. $C = 2 \times C_E$. For example, if $C = 2 \times C_E$, then $\Delta = 2 \times \Delta_0$ when $P = P_E$, as shown in *figure 13.38 a)*. It is interesting to observe that the larger the brace stiffness, the smaller the brace force, *see figure 13.38 b)*.

The above model — though slightly more complicated — can also be used to determine the critical load on a column with several intermediate braces each with the same stiffness C. Assuming typical initial out-of-straightness between supports (or out-of-plumb at the top

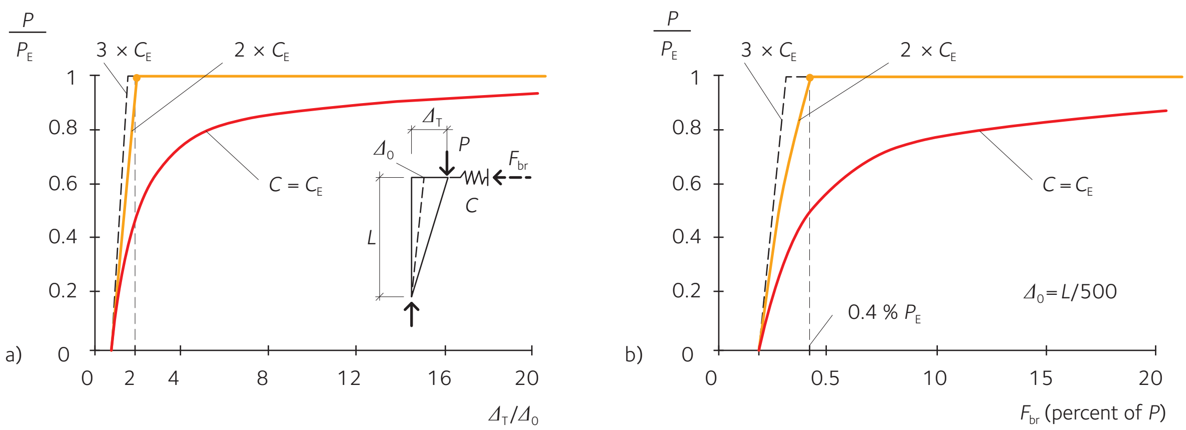


Figure 13.38 Effect of the initial out-of-plumb

of the column in the case of a cantilever) for glulam member with magnitude of 1/500 of the span, a simple design criterion can be derived. Due to the above-mentioned reasons, the recommended brace stiffness C_{min}, is twice the "ideal" stiffness C_E. In this criterion, the Euler column buckling load P_E has been replaced by the design compressive force in the column P_d, due to the fact that in practice the column design load is well below the buckling load. Moreover, it is recommended to choose the bracing force, F_{br}, at least 1 percent of the axial force, even though according to *figure 13.38 b)* F_{br} never exceeds 0.4 percent of the axial force, if $C = 2 \times C_E$. The reason for such a choice is that the brace force recommendation is based on the assumption of an initial out-of-straightness $\Delta_0 = L/500$. However, due to e.g. wind or other lateral forces, bolt oversize, etc. Δ_0 may exceed $L/500$. Thus the brace force, F_{br}, could be larger than 0.4 percent of the axial force.

13.4.2 Requirements for beam bracing

There are two general types of beam bracing — lateral and torsional. An efficient bracing system should prevent relative lateral displacements of the top and bottom side of the beam (i.e. twist of the section). Lateral bracing (purlins or sheeting panels attached on the top side of a simply supported beam) and torsional bracing (cross bracing or diaphragm between adjacent beams) can effectively prevent twist.

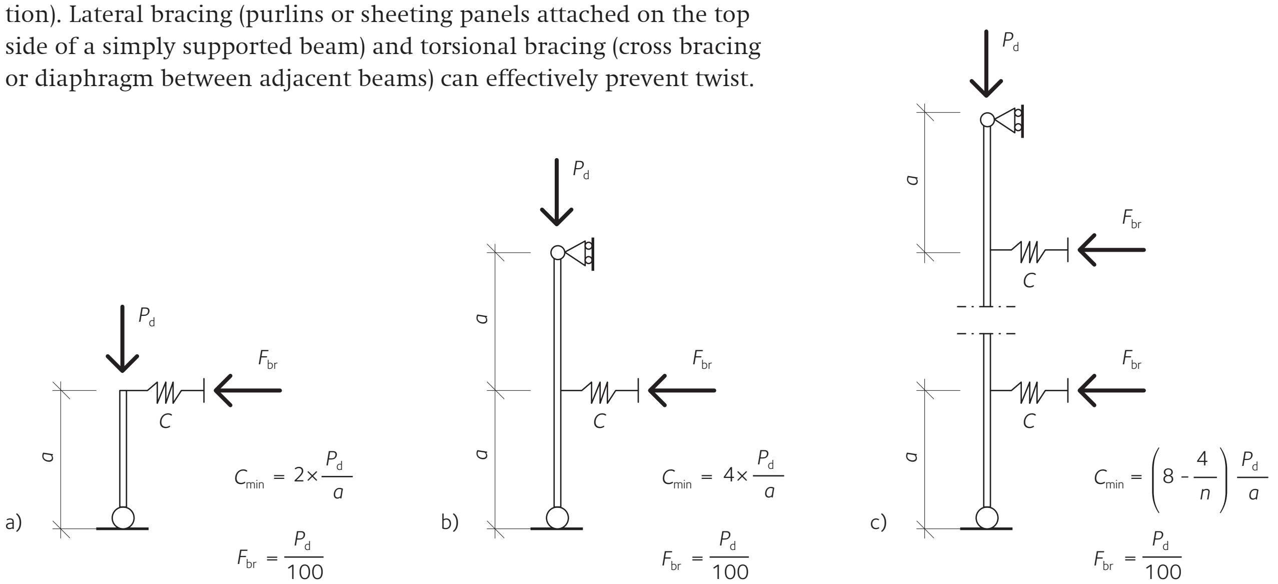


Figure 13.39 Recommended minimum brace stiffness, C_{min}, and corresponding brace force, F_{br}, where "*n*" is the number of brace points

Combined lateral and torsional bracing is more effective than either lateral or torsional bracing acting alone. However, in timber structures it is common practice to use only lateral bracing in the span, *see figure 13.40*, with some torsional bracing devices only located at the supports of the beam.

To prevent lateral buckling, the bracing system must be stiff and strong enough. The design approach presented in this section is applicable only for bracing attached near the top side of the beam. A simplification is made in the determination of the lateral forces generated by each beam. Those forces have to be resisted by the bracing system. Due to initial out-of-plane imperfections, the initial shape of a beam is assumed to be defined by a vertical, curved surface similar to that of *figure 13.41 a)*. In the simplified model the beam is analysed as a compression strut subjected to a load $N_d = 1{,}5 \times M_d/H$, where M_d is the maximum design moment acting on the beam with depth H, *see figure 13.41 b)*. The out-of-plane deformation of the strut can be approximated with a parabolic shape, *see figure 13.41 c)*. Due to the curvature of the strut, the compressive forces N_d gives rise to lateral (radial) distributed forces q_R to be resisted by the bracing.

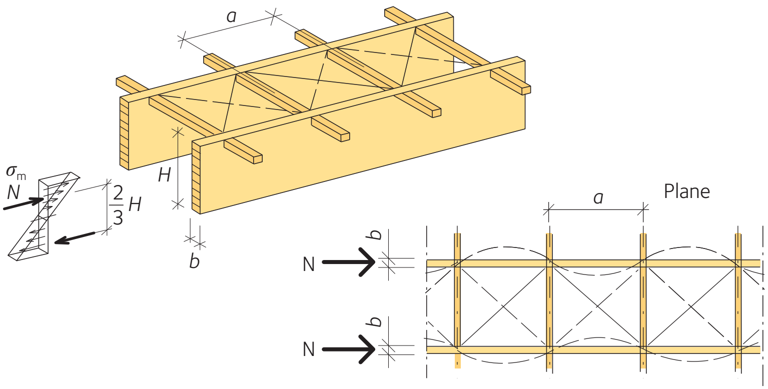


Figure 13.40 **Bracing of beam system by means of a horizontal truss to prevent lateral buckling**

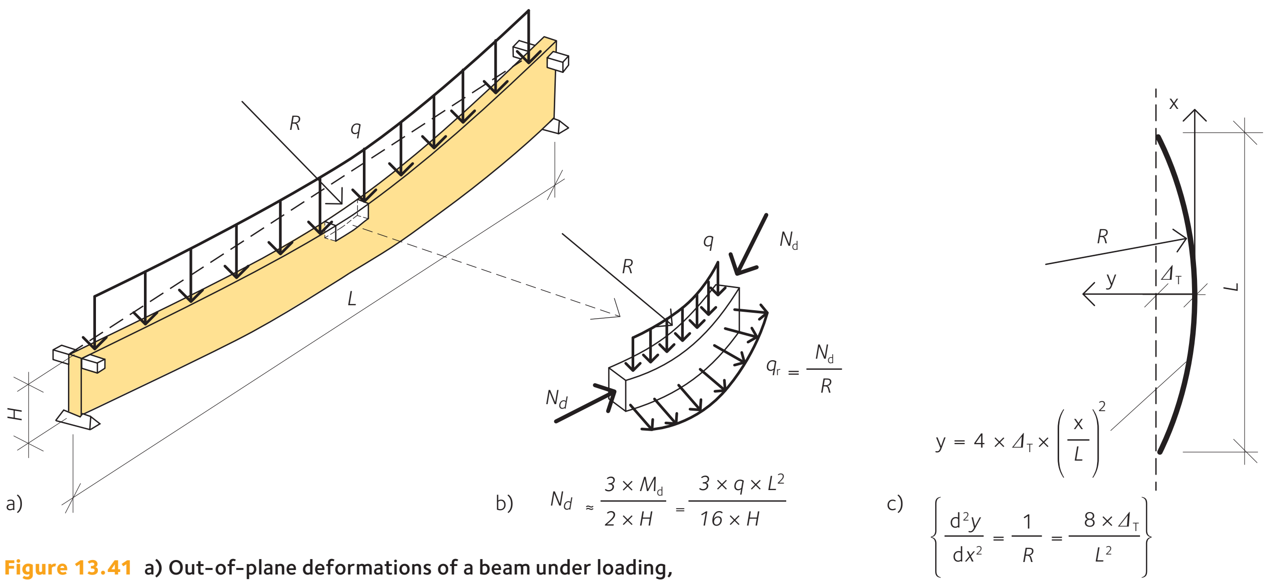


Figure 13.41 **a) Out-of-plane deformations of a beam under loading, b) lateral forces in the equivalent strut, c) assumed out-of-plane deformations**

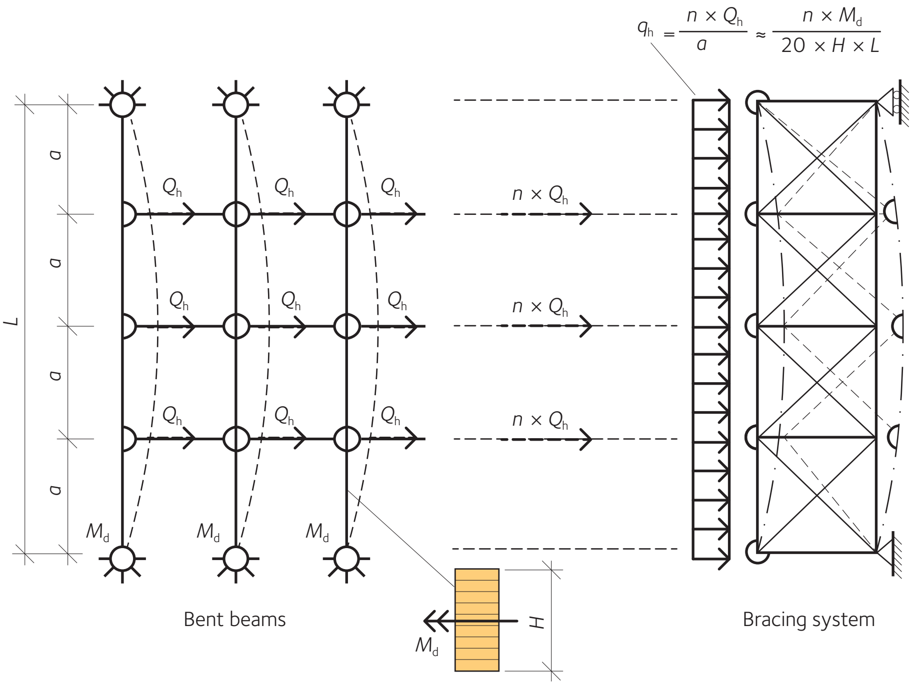


Figure 13.42 **Model for the determination of the brace forces: bracing system for a series of bending members**

The magnitude of the lateral forces can easily be derived by the considerations illustrated in *figure 13.41, page 192*:

$$q_R = \frac{8 \times N_d \times \Delta_T}{L^2} = \frac{N_d}{L} \times 8 \times \frac{\Delta_T}{L} \qquad 13.6$$

For glulam structures an initial out-of-straightness Δ_0 of about $L/500$ is realistic. Furthermore, according to Eurocode 5, the additional out-of-plane displacement Δ due to q_R and any other external load (e.g. wind), should not exceed $L/500$. This means, that the maximum allowed lateral displacement shall be limited to $\Delta_T = (\Delta_0 + \Delta) = L/250$. Thus:

$$q_R = \frac{N_d}{L} \times 8 \times \frac{1}{250} \approx \frac{1}{30} \times \frac{N_d}{L} = \frac{M_d}{20 \times L \times H} \qquad 13.7$$

The load acting on the bracing system increases with the number of beams connected to it. In such a case, the lateral load q_h is the sum of the contributions from all beams connected to the bracing system, *see figure 13.42*.

The approach according to Eurocode 5 is very similar to the one described above. According to Eurocode 5, the lateral destabilizing load, i.e. design load for the bracings, can be determined using the following formula:

$$q_h = n \times \frac{M_d}{k_{f,3} \times H \times L} \times \left(1 - k_{crit}\right) \qquad 13.8$$

where:

- M_d is the design moment in the beam.
- H is the depth of beam.
- L is the span of the beam.
- n is the number of laterally braced beams.
- $k_{f,3}$ is a modification factor ($k_{f,3}$ = 30 as Swedish national preference).
- k_{crit} is the reduction factor for lateral buckling when the beam is unbraced, *see Chapter 4, page 53*.

Pergola

The factor $(1-k_{crit})$ takes into account the slenderness of the beam. When $k_{crit} = 1$, there will be no generation of horizontal destabilizing loads q_h. This means that, for common beam depth-to-width ratios H/b less than approximately 6 – 7, stabilizing loads can be neglected ($q_h \approx 0$) if the beam length-to-width ratio $L/b < 20 - 22$.

In the Eurocode 5 formulation, the right part of *equation 13.8, page 193*, is multiplied by a reduction factor k_l, which should take into account the fact that greater care in workmanship can be expected in large structures. However, that factor k_l is omitted here, due to the fact that it is not representative of real situations.

The bracing structure, moreover, should be stiff enough to limit the deflection due to the design load q_h to $L/700$ and due to the total load, including e.g. wind load, to $L/500$.

The connection of purlins or roofing sheets to the roof beam can be designed for a force of:

13.9 $$Q_h = \frac{q_h \times a}{n}$$

where:

q_h	is the total destabilising load acting on the bracing system.
a	is the distance between purlins or, in sheets direct on the roof beams, distance between screws.
n	is the number of laterally braced beams.

A purlin used to brace several beams, and its connection to the laterally bracing structure, is designed for a force: $n \times Q_h$. Symbols as in *figure 13.43*.

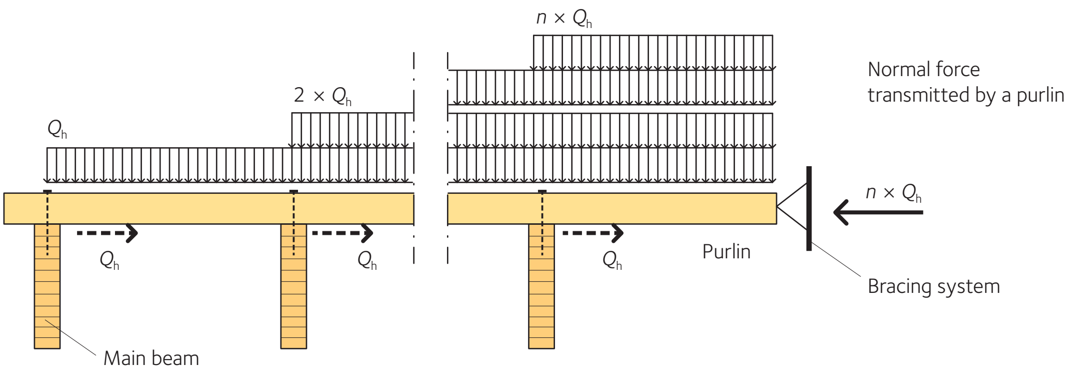


Figure 13.43 **Lateral loads transmitted from the main beams to a purlin**

13.5 Special issues

In this section some special issues related to bracing of large span glulam structures are discussed, namely:
- Bracing of timber members subjected to compression at their unrestrained side.
- Force generated by geometry changes.

13.5.1 Timber members subjected to compression at their unrestrained side

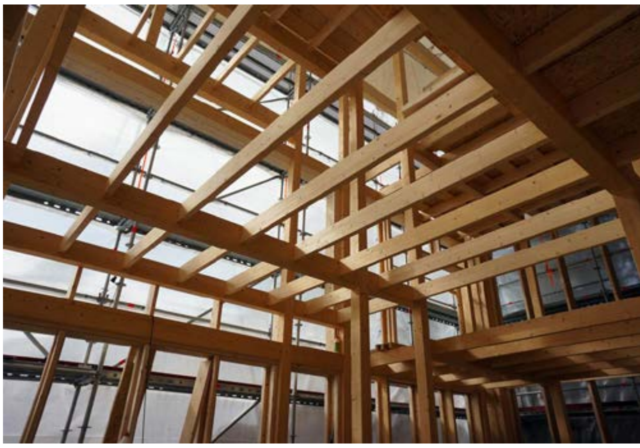
For a simply supported beam subjected to gravity loading, the bottom side will be in tension and the upper side in compression. Lateral restraint of the beam is normally provided along its length, for example by purlins that support the roof cladding. In the case of a continuous beam, on the other hand, at the zones with negative bending moment the beam is subjected to compression at the underside. In such a circumstance, the critical components are, *see figure 13.44*:
- Column in compression,
- Unbraced underside of the beam at the intermediate supports.

If the column top is unbraced, the critical buckling length of the column increases dramatically. For example, if the column is hinged to the foundation and its top is free to translate (unbraced), the buckling length of the column will approach infinity and the system will become unstable.

The other critical component is the beam, which has no bracing points at its underside and thus may be prone to lateral buckling in the zone of negative bending moment.

Structural failures have occurred because of structural arrangements similar to improper or inexistent bracing of the beam-column system, where the beam is subjected to negative bending moment.

Another similar situation where compression stresses occur at the bottom side of a beam (or bottom chord of a truss) is the case of load reversal, e.g. wind suction on the light roof structure. *Figure 10.45 b)* shows a simply supported truss subjected to an uplift load: the bottom chord is subjected to compression and therefore it is prone to buckle laterally.



Villa Moelven, Nacka, Sweden.

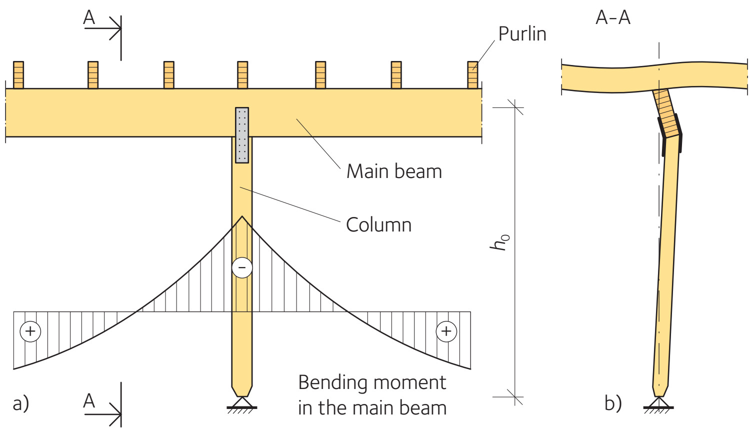


Figure 13.44 a) Continuous beam supported by a column at an intermediate support. b): Possible instability mode.

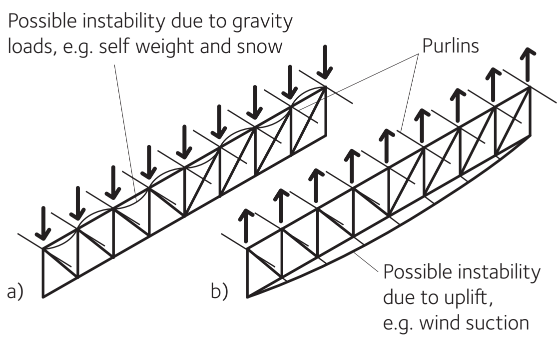


Figure 13.45 Instability modes for
a) gravity loads,
b) uplift loads.

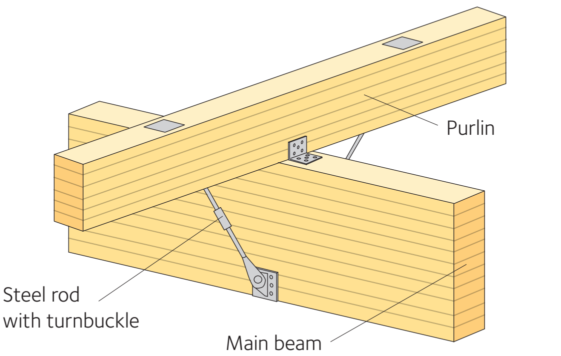


Figure 13.46 Bracing of the underside of a beam by steel rods from the underside of the beam to the roof purlin

A method of restraint to the underside of a beam or a truss is the use of ties, which are inserted along the length of the building at a spacing governed by the slenderness limits of the compression member (bottom chord of a truss or bottom side of a beam). Such ties need obviously to be restrained at their ends by a suitable bracing system. The bracing of the underside of the beam can be performed for example as shown in *figure 13.46*, where steel rods are provided from the underside of the beam to the roof purlin.

The bracing members should preferably be such that they can only take tension. This is to avoid possible torsion of the beam due to uneven load distribution that could occur, for example, when strong wind takes place after snowing. In such a case, big masses of snow could collect in one part of the roof, either to the left or to the right of the main beam, leaving the other part more or less unloaded. Thus, if the bracing diagonals shown in *figure 13.46* were able to take both compression and tension, the main beam would be loaded in torsion with possible risk for collapse of the roof or part of it.

For a simply supported beam with span L, subjected to a maximum negative moment M_{up} due to uplift load (e.g. wind suction), the horizontal force acting on the bracing can be calculated by the following formula, taking into consideration the model shown in *figure 13.47*:

13.10 $$F_{br} = \frac{M_{up}}{20 \times H \times L} \times \frac{a}{\sin\alpha}$$

Similar bracing arrangement as that shown in *figure 13.47* may be needed also for frames or arches in circumstances where negative bending moment can occur, *see figure 13.48*.

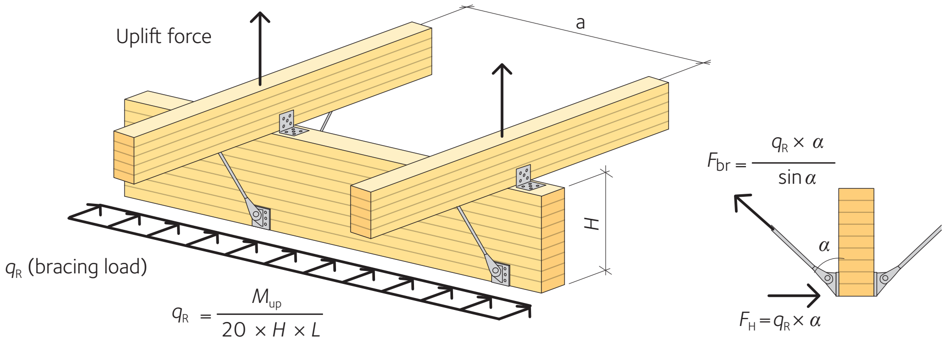


Figure 13.47 Model for the estimation of the bracing force in the steel rods

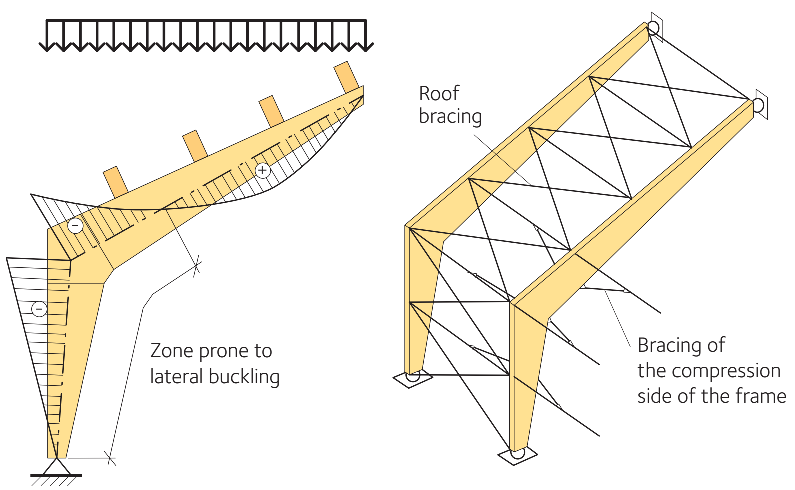


Figure 13.48 Bracing of a portal frame

13.5.2 Forces generated by geometry changes

In design it should be observed that bracing systems are seldom in a horizontal plane. If the primary load-bearing system is also used as a part of the bracing system, non-negligible forces may arise due to the change in slope of the primary members. For example, at the ridge of a double-pitched roof, extra forces will be generated due to the horizontal load (e.g. wind load), namely an upward force at the windward and a downward force at the leeward side, *see figure 13.49*.

These forces can be critical for the design of the primary loadbearing system. The model of *figure 13.50* can be used for the estimation of the downward and upward forces at the ridge of a pitched roof.

The upwards and downwards forces can be calculated by the following equation:

$$F_{\text{up}} = \left| F_{\text{down}} \right| \approx 2 \times \frac{M}{a} \times \sin \alpha \qquad \textbf{13.11}$$

where M is the maximum bending moment in the wind truss due to the wind load. Clearly, the upward and downward actions increase with increasing roof slope.

For more complex structures, especially when the slope of the roof is large, it is suggested to perform a three dimensional analysis of the entire structure, in order to take into consideration the effect of possible changes in geometry in the primary loadbearing system.

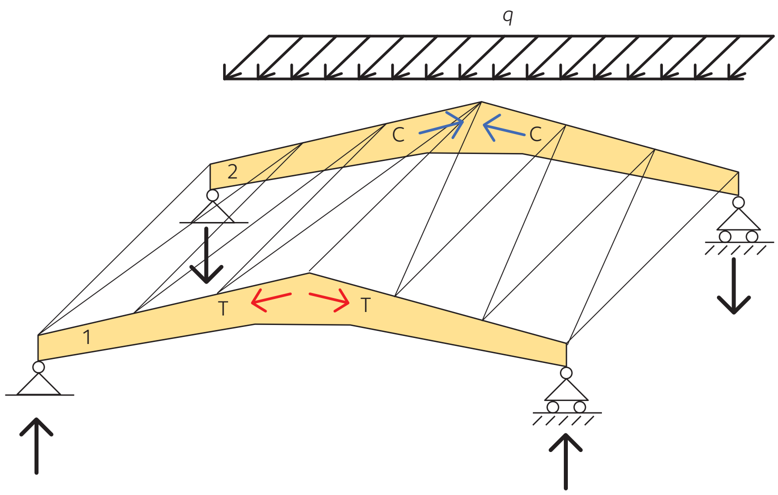


Figure 13.49 Force generated by geometry change at the ridge of a double-pitched roof

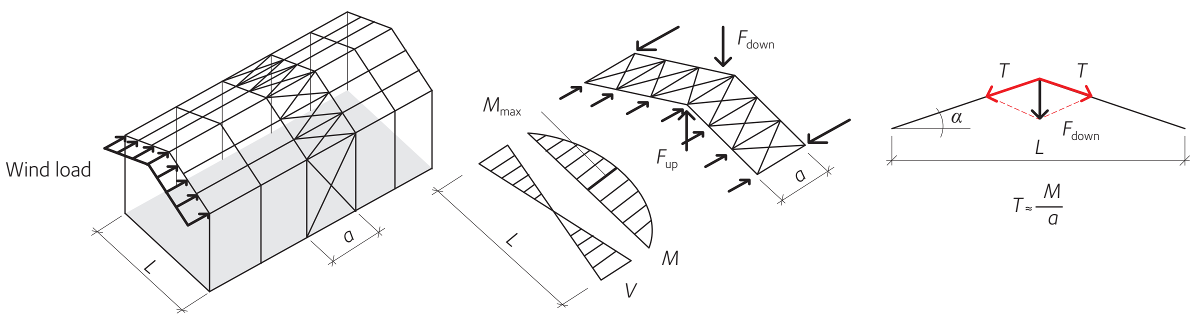


Figure 13.50 Model for estimation of upwards and downwards forces

Joints and connections

Joints are often weak parts in timber structures, often determining the load-bearing capacity of the complete structure. In addition there is a risk that the joint, if not properly designed, can give rise to brittle failure of the structure. Brittle failure modes should in general be avoided in favour of failure modes that clearly indicate that there is a risk of collapse. This can be assured by designing the joints to fail through e.g. yielding in steel parts. In such cases the yielding will lead to large and visible deformation prior to collapse.

Most joints for glulam are based on steel plates in combination with nails, screws or dowels. Long self-tapping screws have become quite common in later years and are being used both for joints and for local reinforcement. Also bonded-in rods (glued-in rods) are being used to some extent in joints and for reinforcement.

The design of timber structures involves a larger number of parameters than the structural design of other materials (duration of load, relative humidity, load to grain angle). When designing joints for timber structures, the orthotropy of the material and its hygroscopic nature are the most important aspects to keep in mind. It is of the utmost importance that engineers are familiar with these aspects.

Section 14.1, page 199, gives a brief overview of the different types of joints found in timber structures.

In *section 14.2, page 199*, general issues related to the design of timber joints are discussed together with some basic principles for the design of steel parts. The different types of joints are then more thoroughly described in *sections 14.3 – 14.9, pages 208 – 227*. *Sections 14.3 – 14.9* are intended to give a general understanding of the load transfer in the joints, and thus examples of reasonable load transfer models are given. The examples are not given as complete design examples, which can instead be found in *The Glulam Handbook, Volume 3*.

14.1 Connections and fixing details – overview

In a building, a large number of different types of connections and other fixing details are used. *Figure 14.1* shows seven different types of connections needed in a simple beam-column framed structure.

Each type of connection can be designed in many different ways; and technical development means the number of different types of fixing details is still increasing. The following text gives a brief description of the different types of connections, each with a general description of the design calculation approach to take. Fully detailed and calculated examples are given in *The Glulam Handbook, Volume 3*. Standard, off-the-shelf connections, plates and connectors are in general favourable from an economic point of view. For glulam structures it is, however, sometimes found that, due to the dimensions needed, these are not available as standard products, in stock. Thus, specially ordered fixing details, plates and connectors must be used, which can be much more expensive than standard products.

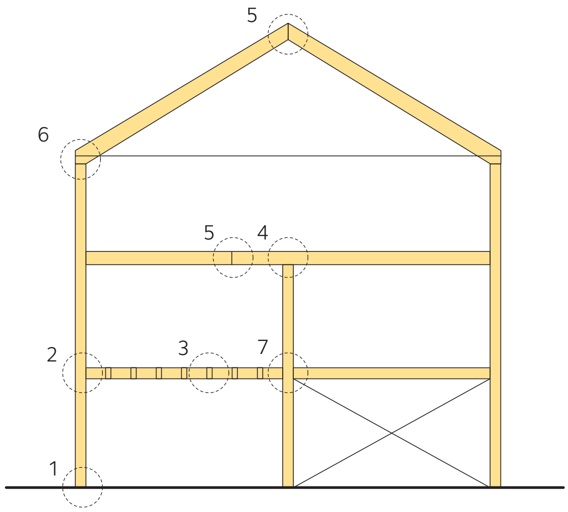


Figure 14.1 **Typical connection types in a beam-column framed structure**

1. Column base.
2. Beam-column connection.
3. Beam-to-beam connection.
4. Column top.
5. Beam joint or ridge joint.
6. Tie fixing.
7. Tie fixing.

14.2 Special considerations

14.2.1 General

Since the design of a joint is often the critical point of a timber structure, the engineer must think through the intended statical way of action of the joint carefully, i.e. you need to have a clear idea of the actual force transfer through the joint, and to enable this force transfer by careful design. In timber joints there are a number of aspects to take into consideration in the design, apart from determining the load-bearing capacity of the joint. These aspects are briefly summarised below.

To be able to perform a proper strength design, the engineer has to determine the forces and moments acting in the joint. These forces and moments are then, in a second step, to be transferred by the fasteners of the joints. In this two-step procedure it is of course of the utmost importance to make use of proper (accurate) mechanical models, otherwise the forces in the joint or in a single fastener can be grossly underestimated.

Wood is a hygroscopic material showing large movements related to variations in relative humidity. When designing timber joints that consist of steel parts, it is of great importance to allow for the wood to expand and shrink during moisture variations, without over large forces being induced.

The low strength of the wood perpendicular to the grain, makes it important to avoid loading in this direction as much as possible.

The metal parts used in joints are, in terms of corrosion and of load bearing capacity at high temperatures (during fire), generally weaker than the wood.

The geometry of the joint will in many cases introduce a weakening of the timber cross-sections by e.g. the screws, slotted-in steel plates or dowels running through the timber. The influence of such a reduction of the cross-sections must be taken into account in the structural design.

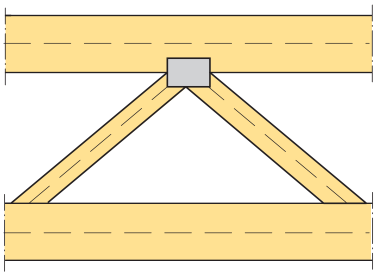


Figure 14.2 A joint which is placed eccentrically in relation to system lines (dashed lines)

14.2.2 Modelling aspects: system lines and eccentricities

By joints, or connections, is meant those load transferring "details" that join different parts of a structure. These parts, e.g. columns and beams, are in most cases designed making use of simple one-dimensional structural models, based on some beam theory or the like. In such cases, there is a direct link between the parts of the structural model (e.g. a beam element in a structural design software) and the physical object (the glulam beam). The structural models used in design require some basic assumptions to be fulfilled in order to make sufficiently accurate predictions. This involves such concepts as the slenderness of a beam element, in order to fit traditional beam theory assumptions, or the assumption of hinged connections in a truss. If such assumptions are not fulfilled in reality, the models used will normally produce poor predictions. As an example, the size (extension) of a dowel type joint in a trussed arch can be large in relation to the length of a diagonal. This in turn raises the question of how to model the diagonal itself, e.g. in determining its critical buckling load.

The system lines of a structure should in general be assumed to coincide with the centre of gravity lines of the structural elements. Furthermore, it is commonly assumed that these are joined in either pinned or fully interacting and moment resisting joints. A joint with large dimensions can give rise to considerable moments, and cannot be modelled as a pinned connection at a single point. A poorly designed joint with large eccentricity will give rise to a moment, and cannot be modelled as a pinned connection in the system line junction, no matter what the actual moment stiffness of the real joint is.

14.2.3 Moisture and duration of load

The movement and deformation of wood when subjected to varying moisture has to be taken into account in the structural design of joints. In addition to a possible initial drying out of the structure, the wood shrinks and swells over the year as the relative humidity varies. Large timber joints are often composed of metal parts that do not move with varying moisture. Thus, if poorly designed, the restraint imposed on the timber by the metal parts can induce large stresses in the wood. Since the strength in tension perpendicular to the grain is quite low, there is a risk of cracking as the timber dries.

The current methods used in structural design calculations are not adapted to this. The modification factors, k_{def} and k_{mod}, *see sections 6.2.2, page 85, and section 2.3.2, page 36*, relate to the lower strength and increased creep of timber at increased moisture content, and consequently predict a higher strength at lower moisture content. Thus, the risk of cracking due to restrained moisture movements must be handled in another manner. To quantify these stresses is, however, very difficult, so it is of the utmost importance that geometric design and the interaction of the various parts of the connection are thoroughly studied.

As an illustrative example, we can consider the shrinkage or swelling of a glulam beam. The total elongation or contraction is given by:

14.1 $$\Delta x = \Delta \mathrm{MC} \times \alpha \times x$$

where:

- Δx is the dimensional change, e.g. given in mm.
- ΔMC is the change in moisture content, in percentages.
- α is the coefficient of expansion.
- x is the initial length, in mm.

Typical values of the coefficient of expansion are approximately $\alpha = 0.0001$ along grain and $\alpha = 0.002$ perpendicular to grain, see example in *figure 14.3*.

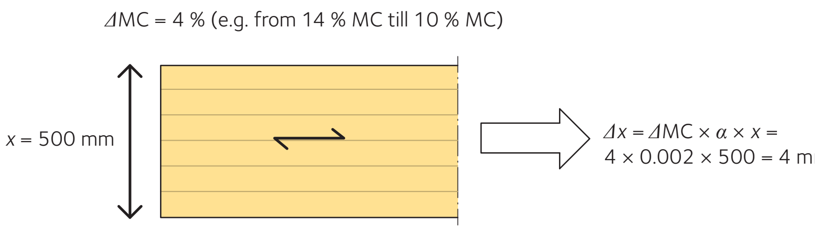


Figure 14.3 Illustration of the magnitude of the shrinkage due to drying for a glulam beam. MC = moisture content.

The largest moisture movements are thus the movements perpendicular to the grain, which is also the direction with the lowest strength. Therefore, one should always avoid designs that give rise to stresses due to constrained movement perpendicular to the grain. One example of how such designs can be avoided is shown in *figure 14.4*.

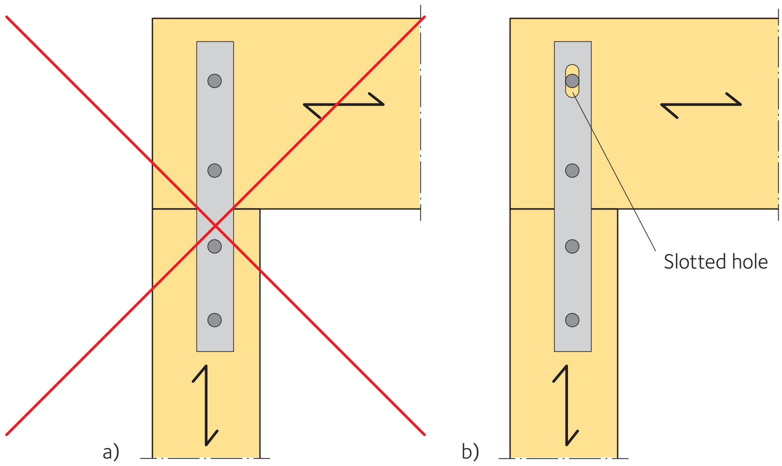


Figure 14.4 a) A connection where movement perpendicular to the grain is constrained in the top beam
b) Alternative design to allow moisture movements perpendicular to the grain

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14.2.4 Splitting failure

Due to its structure, wood has very different strength in different directions. The strength in loading perpendicular to the grain is in the order of only 1 percent of the strength in loading along the grain. Apart from the difference in strength values, there is also a large difference in failure characteristics. In tension, the failures are relatively brittle to very brittle, while failures in compression are ductile — sometimes so ductile that deformation criteria are used in determining the ultimate load.

The wood is weakest in tension perpendicular to the grain, and such loading modes should in general be avoided. The figures below show a few situations where tension perpendicular to the grain will arise. A situation where there is an obvious risk of splitting failure due to tension perpendicular to the grain is when a beam is loaded by a connector as indicated in *figure 14.5*.

Brittle splitting failures can also be obtained in moment-resisting connections, where a force from a single connector has a component acting in a direction perpendicular to the grain. Also in pure tension loading, there can be a risk of splitting, depending on the choice of connector stiffness, in relation to timber thickness and end distance, *see figure 14.6*.

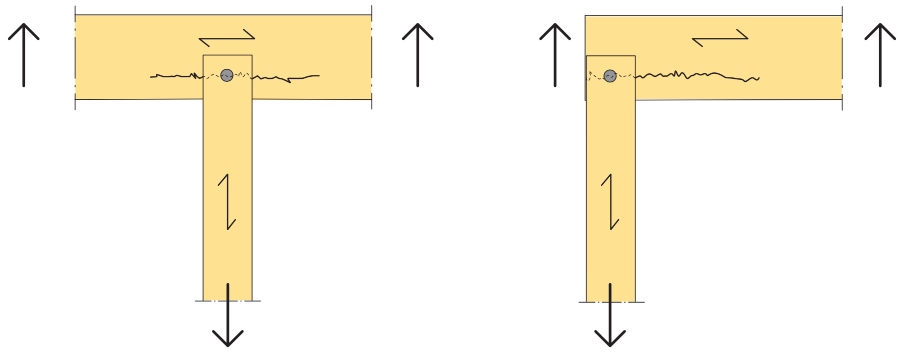


Figure 14.5 Tension perpendicular to the grain can cause splitting in the wood

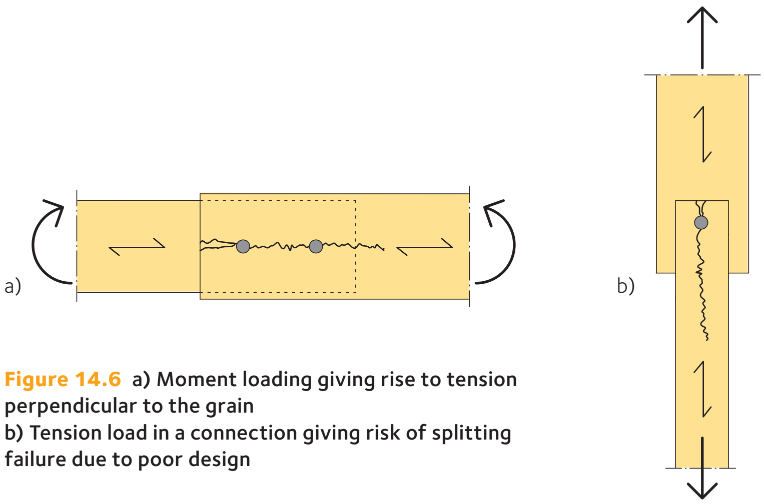


Figure 14.6 a) Moment loading giving rise to tension perpendicular to the grain
b) Tension load in a connection giving risk of splitting failure due to poor design

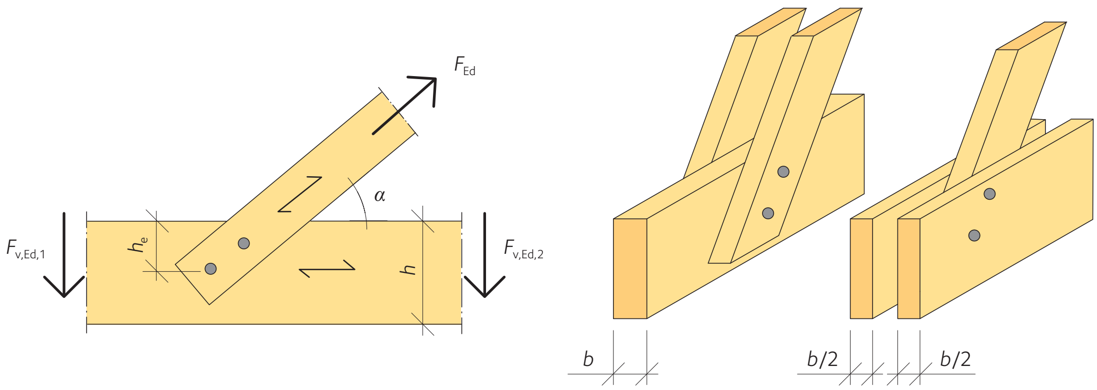


Figure 14.7 Splitting failure due to loading at an angle to grain

The risk of splitting failures should be evaluated whenever loading perpendicular to the grain is introduced.

According to *EC5, section 8.1.4*, and referring to *figure 14.7*, the following criteria should be satisfied:

$$F_{\mathrm{v,Ed}} \leq F_{90,\mathrm{Rd}} \quad \text{where} \quad F_{\mathrm{v,Ed}} = \max \begin{cases} F_{\mathrm{v,Ed,1}} \\ F_{\mathrm{v,Ed,2}} \end{cases} \tag{14.2}$$

where:

$F_{\mathrm{v,Ed}}$ is the design shear force on either side of the joint, *see figure 14.7*.

$F_{90,\mathrm{Rd}}$ is the design resistance, calculated from the characteristic value $F_{90,\mathrm{Rk}}$.

$$F_{90,\mathrm{Rk}} = 14b\sqrt{\frac{h_\mathrm{e}}{\left(1-\frac{h_\mathrm{e}}{h}\right)}} \tag{14.3}$$

where:

$F_{90,\mathrm{Rk}}$ is the characteristic resistance [N].

h_e is the distance between loaded side and centre of the most distant single connector [mm].

h is the timber depth [mm].

b is the timber width [mm].

The above EC5-approach does not take into account any possible effect of having a number of dowels in the direction of the grain of the member loaded perpendicular to the grain. An alternative approach is given in the German NA to the EC5 for all cases that are not covered by *figure 14.7* (e.g. joints cross-connections with rows of fasteners parallel to the grain in the member loaded by tensile stresses perpendicular to the grain). This approach is as follows, referring to *figure 14.8:*

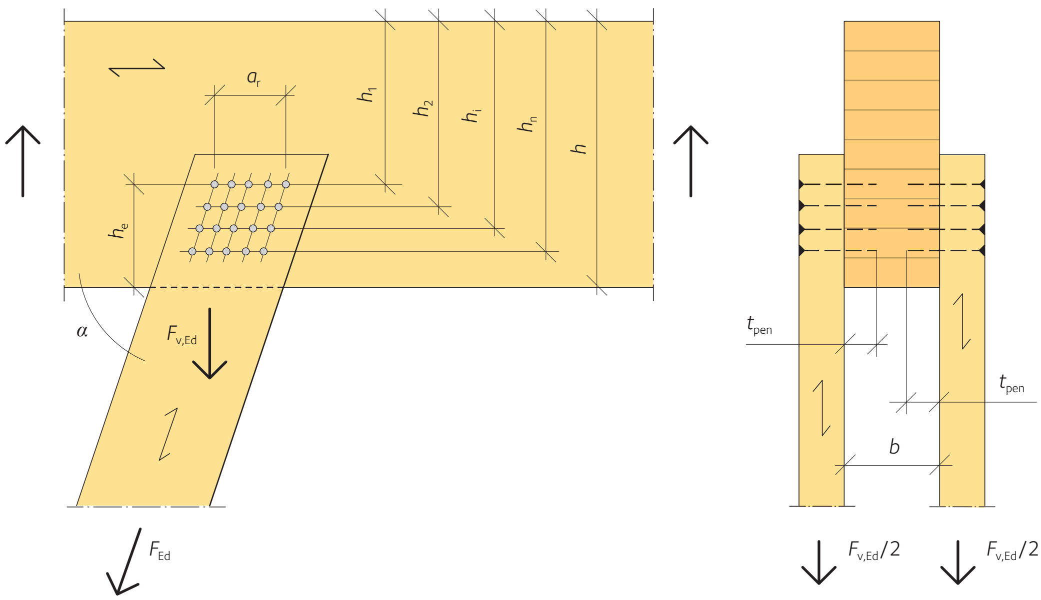


Figure 14.8 Notation used in design for splitting failure

For members with a rectangular cross-section that are loaded at an angle α to the grain, *see figure 14.8, page 203*, the tensile stresses perpendicular to the grain due to a tensile force component perpendicular to the grain $F_{v,Ed} = F_{Ed} \times \sin\alpha$ may be accounted for as follows:

1. For cross-connections with $h_e/h > 0.7$, *see figure 14.8, page 203*, no further verification is required. Cross-connections with $h_e/h < 0.2$ should only be loaded by forces of short duration (e.g. wind suction).
2. For cross-connections with $h_e/h \leq 0.7$, the following expression should be satisfied:

14.4 $$F_{v,Ed}/F_{90,Rd} \leq 1.0$$

where:

14.5 $$F_{90,Rd} = k_s \times k_r \times \left(6.5 + 18 \times h_e^2/h^2\right) \times \left(t_{ef} \times h\right)^{0.8} \times f_{t,90,d}$$

where:

14.6 $$k_s = \max\left\{1;\ 0.7 + 1.4 \times a_r/h\right\}$$

and:

14.7 $$k_r = \frac{n}{\sum_{i=1}^{n}\left(\frac{h_1}{h_i}\right)^2}$$

Universeum, Gothenburg, Sweden.

$F_{v,Ed}$ is the design value of the force component perpendicular to the grain [N].
$F_{90,Rd}$ is the design splitting capacity of the member [N].
$f_{t,90,d}$ is the design value for tensile strength perpendicular to the grain [MPa].
k_s is a factor to take into account the spacing of fasteners in a row of fasteners parallel to the grain.
k_r is a factor to take into account multiple rows of fasteners; for bonded-in rods, $k_r = h/(h-h_e)$.
h_e is the loaded edge distance to the centre of the most distant fastener; for bonded-in rods, h_e is the projected length $l_{ad} \times \sin\alpha$ [mm].
a_r is the spacing between the centres of the two outermost fasteners in a row of fasteners parallel to the grain, the spacing of the fasteners within one row parallel to the grain of the member under tensile stresses perpendicular to the grain should not be greater than $0.5 \times h$ [mm].
h is the member depth [mm].
t_{ef} is the effective depth, see definition below [mm].
n is the number of rows of fasteners.
h_i is the unloaded edge distance to the axis of the row of fasteners considered [mm].

3. For members with a cross-connection in the centre or on both sides of the member, the following applies:

$t_{ef} = \min\{b;\ 2 \times t_{pen};\ 24 \times d\}$ for timber-to-timber or panel-to-timber connections with nails or screws.
$t_{ef} = \min\{b;\ 2 \times t_{pen};\ 30 \times d\}$ for nailed steel-to-timber connections.
$t_{ef} = \min\{b;\ 2 \times t_{pen};\ 12 \times d\}$ for dowelled or bolted connections.
$t_{ef} = \min\{b;\ 100\ \text{mm}\}$ for connections with split ring; shear plate and toothed plate connectors.
$t_{ef} = \min\{b;\ 6 \times d\}$ for connections with bonded-in rods.

Alpin Ski Lodge, Romme, Sweden.

where:

- b is the timber member width [mm].
- d is the diameter of the fastener [mm].
- t_{pen} is the penetration depth of the fastener [mm].

4. For members with a cross-connection on one side of the member the following applies:

   $t_{ef} = \min\{b; t_{pen}; 12 \times d\}$ for timber-to-timber or panel-to-timber connections with nails or screws.
   $t_{ef} = \min\{b; t_{pen}; 15 \times d\}$ for nailed steel-to-timber connections.
   $t_{ef} = \min\{b; t_{pen}; 6 \times d\}$ for dowelled or bolted connections.
   $t_{ef} = \min\{b; 50\ \text{mm}\}$ for connections with split ring; shear plate and toothed plate connectors.

5. For members with more than one group of fasteners, the design splitting capacity of the member for one group of fasteners may be determined according to *equation 14.4, page 204*, provided that the clear distance parallel to the grain between both groups of fasteners is greater than $2 \times h$.
6. If the clear distance parallel to the grain between two groups of fasteners is less than $0.5 \times h$, the fasteners in these groups should be treated as one group of fasteners.
7. If the clear distance parallel to the grain between two groups of fasteners is greater than $0.5 \times h$ and less than $2 \times h$, the design splitting capacity of the member for one group of fasteners $F_{90,Rd}$, according to *equation 14.4, page 204*, should be reduced by the factor k_g:

   $$k_g = l_g / (4 \times h) + 0.5 \qquad (14.8)$$

   where l_g is the clear distance parallel to the grain between both groups of fasteners.

Recommendations for reinforcement

8. Cross-connections with $a_r/h > 1.0$ and $F_{v,Ed} > 0.5 \times F_{90,Rd}$ should be reinforced.
9. For members with more than two groups of fasteners at a distance $l_g \leq 2 \times h$, for which the design value of the force component perpendicular to the grain, $F_{v,Ed}$, is greater than half of the design load-carrying capacity of the member, $F_{90,Rd}$, reduced by the factor k_g, these force components should be carried by reinforcement.
10. This also applies to cross-connections with a clear distance to the end of the cantilever smaller than the member depth h, if $F_{v,Ed} > 0.5 \times F_{90,Rd}$.

It should also be pointed out that in very simple connections there is a risk of splitting failure if the connection is loaded with a combination of normal forces, shear forces and moments. To design such (simple) connections one must calculate a different resistance value for every single connector in the joint, since the load direction in relation to the grain direction is different for every connector. Depending on this load-to-grain angle, according to *EC5, section 8.5.1* different embedment strength values are to be used in the design formulae. Those strength values do not, however, take into account the risk of splitting failure. It is thus not obvious how this risk should be assessed in multiple dowel joints loaded by moments.
A possible approach could be to use the above expressions for each dowel in the joint.

Bridge, Gislaved, Sweden.

14.2.5 Failure in steel plates

Eurocode 5 states that the strength of the steel plates in connections must be checked. This should be done using *Eurocode 3 – Steel structures*. There are different types of failure modes that can arise: tension, compression, shear, and bending, a combination of these and failure in compression at the hole edge. With the combinations of plate thickness and dowel diameters commonly in use, there is very seldom any risk of failure at the edges of the holes in the plate. This failure mode is therefore not considered further in this publication.

Tensile failure of steel plate

Two possibilities exist for this failure mode: either failure of the full cross-section, or failure of the net cross-section (taking into account the smaller cross-section due to the holes for the connectors).
The lesser of these two capacities then constitutes the capacity of the steel plate in tension.

The load-bearing capacity of the full cross-section, $N_{pl,Rd}$, is given by:

14.9 $$N_{pl,Rd} = \frac{f_y \times A}{\gamma_{M0}}$$

The capacity of the net cross section, $N_{u,Rd}$, is given by:

14.10 $$N_{u,Rd} = \frac{0.9 \times f_u \times A_{net}}{\gamma_{M2}}$$

where:

f_y is the yield strength of the steel [MPa].
f_u is the ultimate strength of the steel [MPa].
A is the full cross-section of the steel [mm²].
A_{net} is the net cross-section of the steel (through a row of holes) [mm²].
γ_{M0} is the partial coefficient of the material, 1.0.

$$\gamma_{M2} = \max\left[1.1; 0.9 \times \frac{f_u}{f_y}\right]$$

Compressive failure of steel plates

$N_{c,Rd}$ is the resistance value in compression under the assumption that no local instability occurs, and can be calculated according to:

14.11 $$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}}$$

where:

f_y is the yield strength of the steel [MPa].
A is the full cross-section of the steel [mm²].
γ_{M0} is the partial coefficient of the material, 1.0.

It is not necessary to take into account the effect of the holes as long as these are filled by the connectors. It is not necessary to check for the risk of buckling of the steel plate if the distance between the connectors, a_1, is smaller than a permitted value:

14.12 $$a_1 \leq 9t \times \varepsilon = 9t\sqrt{\frac{235}{f_y}}$$

In other cases a control of the risk of buckling should be performed. In such an analysis it is then assumed that the steel plate acts as a column in compression (Euler buckling). The buckling length is set to be 0.6 times the distance between the dowels, i.e. 0.6 a_1, where:

t is the thickness of the steel plate [mm].
ε is a dimensionless factor for determining the cross-section class of the steel plate.

Bending failure of steel plates

If the complete cross-section can plasticize:

$$M_{c,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} \qquad (14.13)$$

where:

$M_{c,Rd}$ is the design bending moment capacity [Nmm].
W_{pl} is the plastic section modulus of the steel plate, see below [mm³].
f_y is the yield strength of the steel [MPa].
γ_{M0} is the partial coefficient of material, 1.0.

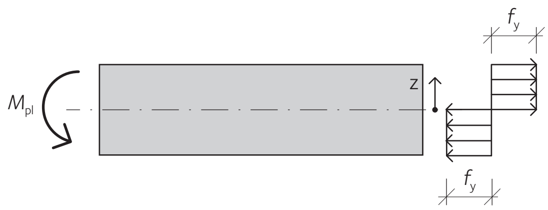


Figure 14.9 The plastic section modulus is defined by the bending moment resulting from a fully plasticized cross-section

The plastic section modulus is (by definition) calculated from the bending moment resulting from a fully plasticized cross-section, *see figure 14.9*:

$$M_{pl} = \int_A \sigma z \,\mathrm{d}A = W_{pl} f_y \qquad (14.14)$$

For a rectangular cross-section, the bending moment due to the stress distribution indicated in *figure 14.9* is obtained as:

$$M_{pl} = 2 \times f_y b \frac{h}{2} \times \frac{h}{4} = \frac{f_y b h^2}{4} \qquad (14.15)$$

where h is the depth of the cross-section and b is its width.

From *equation 14.14* and *14.15* we obtain:

$$W_{pl} = \frac{bh^2}{4} \qquad (14.16)$$

It is not necessary to take into account the effect of holes in the tensile zone as long as the following condition is fulfilled:

$$\frac{A_{net} \times 0.9 \times f_u}{\gamma_{M2}} \geq \frac{A \times f_y}{\gamma_{M0}} \qquad (14.17)$$

f_u is the ultimate strength of the steel [MPa].
A is the area of tensile zone [mm²].
A_{net} is the net area exposed to tensile stress [mm²].
γ_{M0} is the partial coefficient of material, 1.0.

$$\gamma_{M2} = \max\left[1.1; 0.9 \times \frac{f_u}{f_y}\right]$$

In compression, there is no need to account for the holes as long as these are filled by the connectors.

Shear failure of steel plates

For the case of a cross-section that can be fully plasticized, the shear capacity can be calculated according to:

14.18 $$V_{c,Rd} = V_{pl} = \frac{A_v \left(f_y/\sqrt{3}\right)}{\gamma_{M0}}$$

where:

$V_{c,Rd}$ is the design sheer resistance [N].
f_y is the yield strength of the steel [MPa].
A_v is the shear area [mm^2].
γ_{M0} is the partial coefficient of material, 1.0.

Combined states of stress

For states of stress, which includes a combination of normal stress and shear stress, the following criterion can be used:

14.19 $$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right) + 3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right) \le 1$$

where:

$\sigma_{x,Ed}$ is the design value of longitudinal normal stress.
$\sigma_{z,Ed}$ is the design value of transversal normal stress.
τ_{Ed} is the design value of shear stress.
f_y is the yield strength of steel.
γ_{M0} is the partial coefficient of material, 1.0.

This criterion is, however, simplified on the safe side, since it does not include any beneficial effects due to plasticity. In order to get a more realistic estimate of the load-bearing capacity of the steel plate, other interaction formulae for sectional forces can be used (normal force, *N*, shear force, *V*, and moment, *M*). Such expressions can also be used for the sections containing holes, by using the net cross-sectional quantities instead (i.e. reduced area and moment of inertia due to the presence of the holes in a row).

Riding school, Bökeberg, Sweden.

14.3 Column base

Columns used in glulam structures are commonly classified as being either pinned (no moment being transferred) or rigidly fixed (moment resisting base). The choice will affect not only the design of the column itself, but also of the foundation. The connection to the foundation can be made in different ways, e.g. by casting the fixing plates into the concrete floor, or by welding them to steel fixings already cast into the concrete. A third alternative is to use anchor bolts; both mechanical (expansion) anchor bolts and chemical (adhesive) anchor bolts are available. The design of such fixings to the concrete itself is not dealt with here, and should be carried out in general according to the provisions of *Eurocode 2 – Concrete structures*.

Ends of columns resting directly on concrete, brick/blockwork or other hygroscopic materials should be fitted with some kind of damp membrane, e.g. 4.8 mm oil tempered hardboard nailed and glued to the column base, or a rubber membrane. For outdoor columns or columns in spaces where water is regularly present, e.g. swimming pool facilities, the connection to the foundation must be designed so that the end of the column is protected from water and can dry out quickly if it should become wet — by having high foundations, for example.

The fixtures are usually formed with external steel side plates nailed, screwed or bolted 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.

Fixtures are delivered separately, except for glued-in bolts, which are always glued in the factory. Holes in columns for bolted joints should preferably be drilled during erection, as this will avoid fitting problems, especially with cast-in fixtures.

14.3.1 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.

Externally mounted flat plates

The simplest and most commonly used type of pinned column base is made using pairs of steel plates. The steel plates are mounted on the wider sides of the column with nails, screws or bolts, *see figure 14.10*. Such connections are suitable for both smaller and larger horizontal forces.

Prefabricated steel plates with different hole patterns and thicknesses can be ordered, as well as with appropriate surface treatment, 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 diameter of the fastener — so typically a 5 mm hole would be used for a 4 mm anchor nail.

In this type of joint, the vertical compressive force is transmitted by contact between the column end and the ground. The horizontal force $F_{E,z}$ and any possible tensile vertical force $F_{E,x}$ are both transferred by the nails into the nail plate, which transfers the forces to the ground. The plate is assumed to act as a cantilever, being rigidly connected to ground.

The following possible failure modes are to be checked:

- Shear failure of the nailed connection in the column.
- Block and plug shear failure, *cf. EC5, Annex A*.
- The risk of splitting failure.
- Moment, normal force and shear force in the steel plate (both full and reduced cross-section).
- Buckling of the steel plate for compressive force.

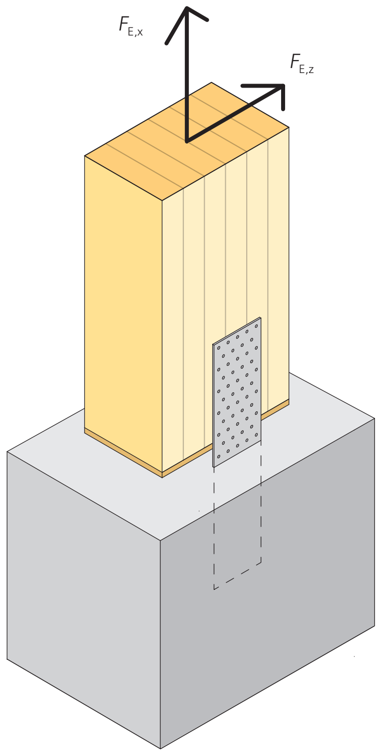


Figure 14.10 Pinned column base with externally mounted steel plates. Schematic figure. Fasteners can be nails, screws or bolts. Moisture protection between glulam and concrete.

The horizontal force and a possible vertical tensile force are assumed to act in the centre of gravity of the nail group. Thus a resulting force F_E can be calculated:

$$F_E = \sqrt{{F_{E,x}}^2 + {F_{E,z}}^2} \qquad 14.20$$

Nails with a diameter less than 8 mm have, according to EC5, a capacity that is independent of the load to grain angle. The capacity per fastener, $F_{v,Rd}$, can be determined according to *EC5, Section 8.3*. From that, the number of nails needed, n, can be calculated:

$$n = \frac{F_E}{F_{v,Rd}} \qquad 14.21$$

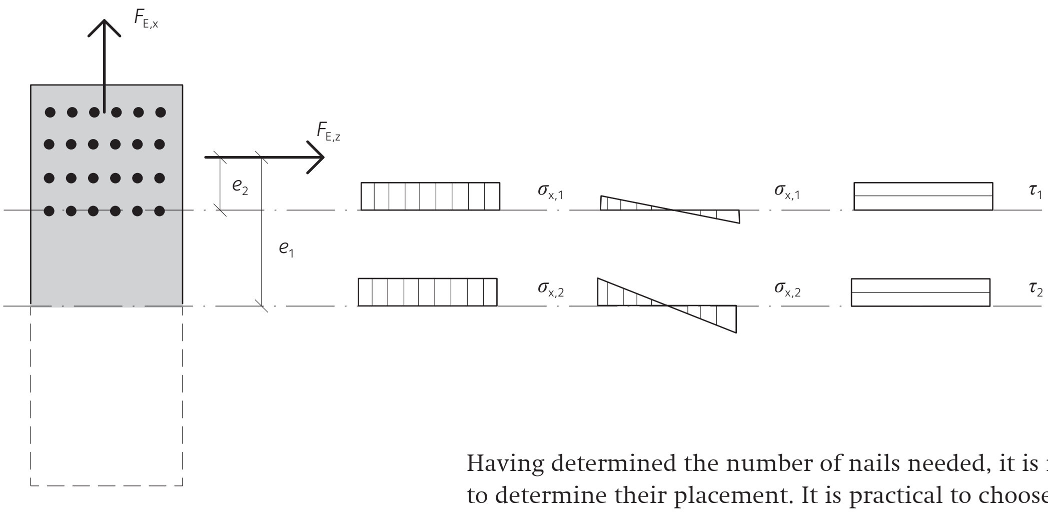


Figure 14.11 Stress distribution in the steel plate

Having determined the number of nails needed, it is necessary also to determine their placement. It is practical to choose a distance between the nails of at least 14*d* (*d* is the diameter of the connector), since no account has to be taken of the influence of the number of nails in a row, *see EC5, table 8.1*. If standard nail plates are being used, then of course the nail distance is also determined by the hole pattern of the plate. Block and plug shear failure is checked according to *EC5, Annex A*. Splitting failure does not have to be checked if the distance from the column edge to the most distant connector is larger than 0.7 times the width of the column. Otherwise, splitting failure is to be checked according to *section 14.2.4, page 201* or *EC5, 8.1.4*.

The steel plate is loaded by a vertical force and by a horizontal force. In addition, the eccentricity of the horizontal force, *see figure 14.11*, gives rise to a moment. At the built-in end of the steel plate:

14.22 $$M_E = F_{E,z} \times e_1$$

And at the mostly stressed row:

14.23 $$M_E = F_{E,z} \times e_2$$

From the moment and the vertical and horizontal forces, the stress at various locations in the steel plate can be calculated. The combined effect of these loads can then be evaluated according to *section 14.2.5, page 206*.

For thin steel plates it can be necessary to check the risk of buckling of the plate in compression. However, if the requirements of distance between connectors in the steel plate are fulfilled, this check will not be necessary. This means that the distance between the connector holes in the steel plate, and between the first row and the ground, cannot be larger than the smaller of 14*t* (*t is the plate thickness*) and 200 mm. The fixing of the steel plate to the ground must of course also be checked. If the concrete around the steel plate is cast in place, the adhesion to the concrete must be checked. If the steel plates are welded to cast-in details, the weld must also be checked.

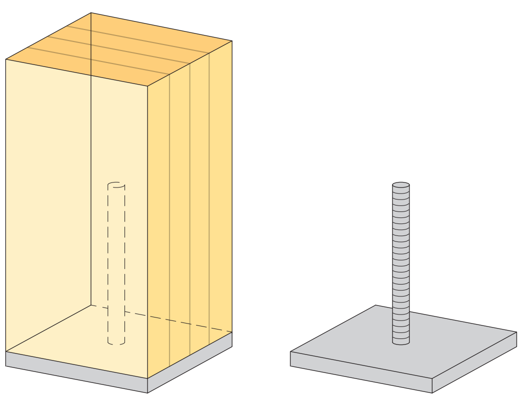


Figure 14.12 Pinned column connection based on bonded-in rod

Bonded-in rod

Another possible solution for a pinned column base is to use a bonded-in rod, *see figure 14.12*. One advantage of this type of joint is that it is practically invisible, another is the fire resistance gained by having the steel parts embedded into the timber. This type of joint should not be used in connections exposed to dynamic loads, or in service class 3. The gluing needs to be done under controlled conditions, and is therefore performed at the factory (the procedure is performed under separate factory production control routines). The most commonly used version of this connection consists of the bonded-in rod and a steel plate, which in turn is welded or bolted to the foundation.

This column base connection type should only be used for columns exposed to small or moderate loads. The connection has very limited moment capacity, which means that the column must be stabilised during erection of the structure.

The following failure modes have to be checked:

- Failure of the rod.
- Pull-out of the rod.
- Failure of the wood in splitting,tension, or compression.

Design formulae for bonded-in rods are not given in EC5. Separate design instructions can instead be found in technical approvals. In Sweden there is one such technical approval (No. 1396/78, issued by SITAC) that is valid for threaded rods. Typical of the design procedure for bonded-in rods is that these include formulas for different loading situations and minimum requirements on end- and edge distances, bonded-in lengths etc.

The expression used in the Swedish national technical approval for the axial pull-out capacity is the following:

$$R_{\text{td}} = \min \begin{cases} 0.6 \times f_{\text{bu,k}} A_{\text{s}} / 1.2 \\ \pi d_{\text{ekv}} l_{\text{i}} f_{\text{ax,k}} \frac{\tanh \omega}{\omega} k_{\text{mod}} \kappa_1 / 1.25 \end{cases} \qquad 14.24$$

where:

$f_{\text{bu,k}}$	is the characteristic ultimate strength of the threaded rod [MPa].
A_{s}	is the cross-sectional area of the threaded rod [mm^2].
d_{ekv}	is the equivalent diameter of the threaded rod [mm], with $d_{\text{ekv}} = \min(1.15\, d_{\text{nom}}, d_{\text{hole}})$.
d_{nom}	is the nominal diameter of the threaded rod = outer diameter.
d_{hole}	is the hole diameter ($> d_{\text{nom}}$).
l_{i}	is the bonded-in length [mm].
$f_{\text{ax,k}}$	is 5.5 MPa.

$$\omega = \frac{0.016 l_i}{\sqrt{d_{\text{ekv}}}} \quad \text{(nondimensional brittleness ratio)}$$

k_{mod}	is a modification factor to account for the influence of duration of load and moisture, *see table 2.4, page 36*.
κ_1	is 1.0 for service class 1 and 0.8 for service class 2.

In addition to this a minimum bonded-in length, l_{min}, has to be considered:

$$l_{\text{min}} = \max \begin{cases} 0.5 d_{\text{nom}}{}^2 \\ 10 d_{\text{nom}} \end{cases} \qquad 14.25$$

The first formula of *equation 14.24* relates to failure in tension of the threaded rod and contains the partial coefficient for steel, $\gamma_{\text{M}} = 1.2$. The second formula of *equation 14.24* relates to failure in tension of the threaded rod and contains the partial coefficient for glulam, $\gamma_{\text{M}} = 1.25$. When designing a bonded-in rod connection, the ductile failure mode of tensile failure of the rod should be aimed at.

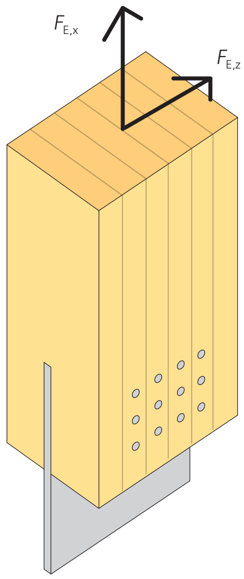


Figure 14.13 Slotted-in steel plate for pinned column base

The parameter $f_{ax,k}$ (MPa) corresponds to the formal shear strength of the joint at a perfectly uniform shear stress distribution (which is obtained for small values of ω for which $(tanh(\omega))/\omega = 1.0$).

The above expressions are valid also for compression. The risk of buckling of the threaded rod should, however, be considered when the stress in the rod exceeds 300 MPa.

As mentioned above, the risk of failure in the wood is also to be considered. Apart from the obvious failure mode of tensile (or compressive) failure of the wood in the grain direction, this could also include the risk of splitting failure, due to tension perpendicular to the grain, e.g. when for tension in a rod glued-in perpendicular to the grain.

For bonded-in rods that are loaded perpendicular to the rod, the design approach of *EC5, 8.2 and 8.6* for dowels can be used (for rods glued in perpendicular to the grain). For rods glued in parallel to the grain the embedment strength is set to 10 percent of the embedment strength for dowels inserted parallel to the grain. The risk of splitting failure must be assessed.

See also *The Glulam Handbook Volume 1* on glued-in dowel pins, which are a simpler, non-force transferring variant of the bonded-in rod and may be sufficient where all that is required is control of the column.

Slotted-in plates

Another possibility, which also has the benefit of producing an almost invisible connection, is to use slotted-in steel plates and dowels. The plates are often welded to a base plate, which in turn is bolted to the foundation. Apart from aesthetic benefits, being hidden gives the connection increased fire resistance.

This connection, *see figure 14.13*, has some moment resisting capacity, making it possible to use the connection during erection.

The following failure modes should be checked:

- Shear failure of connection (dowel action).
- Block and plug failure.
- Moment, normal force and shear force in the steel plate, gross and net cross-sections to be checked.

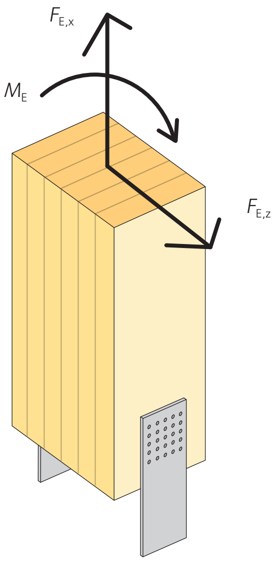


Figure 14.14 Externally mounted steel plates for fixed column base

The dowel group is assumed to take the resulting force from uplift and horizontal actions by dowel action in the joint. The design for this is done according to the provisions mentioned previously; and, of course, if, multiple plates are used, the corresponding multiple shear plane formulae of EC5 should be used. For connector diameters exceeding 6 mm, the influence of the load-to-grain angle must be taken into account. Block and plug failure modes are checked according to *EC5, Appendix A*.

14.3.2 Rigidly fixed column base

The moisture characteristics of the timber and its relatively low strength perpendicular to the grain – in comparison with its bending strength – mean that rigidly fixed glulam columns require special care when designing details, 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 may be a suitable alternative. However, glued-in bolts are only suitable when the column has relatively small end moments. For large end moments, steel fishplates with nailed or screwed joints are used.

Externally mounted flat steel plates

The most commonly used type of column base for a rigidly connected column, is by the use of flat steel plates on both sides of the column, *see figure 14.14, page 212*. The plates are normally attached to the narrow faces of the column by the use of nails, but screws can also be used. The connection type is simple to manufacture and can be used for a wide range of horizontal forces. The steel plates can be cast-in or welded to a cast-in base plate. The same general considerations as those mentioned for the pinned connection should be observed.

Below, a calculation model is briefly described, *see figures 14.15 and 14.16*. A detailed design example is given in *The Glulam Handbook Volume 3*.

It is assumed that vertical compressive forces are transferred to the foundation by contact. Vertical tensile forces in the column are transferred through the dowel action of one of the plates of the joint.

$F_{E,x}$ is taken in contact with the foundation if it is compressive, by the dowel action in the connectors if not. The moment M_E is taken by a force couple in the two steel plates. The horizontal force $F_{E,y}$ is transferred by contact against the steel plates. If the steel plate is slender enough to buckle, then the bending moment must be taken by contact between the column end and the foundation, in combination with tension in one of the steel plates (and dowel action in the dowels mounted in that plate).

The following failure modes should be checked:

- Shear of the connection (dowel action).
- Block and plug failure of the connection.
- Normal force in the steel plate, gross and net cross-sections, including risk of buckling.
- Contact between steel plate and column, i.e. stresses perpendicular to grain.
- Contact between column and foundation, i.e. stresses parallel to grain.

The shear of the connection (dowel action) is checked according to *EC5, section 8.3* and block/plug shear according to *EC5, Appendix A*. The capacity of the steel plate can be calculated according to the above *section 14.3.1, page 209*. In addition, the edge distances of the connectors must be checked. Concerning the distance between connectors, it is advisable (for simplicity) to place them far enough apart, to avoid having to consider the effect of the number of connectors in a row.

The vertical tension force to be transferred by the one plate, F_v, is given by:

$$F_V = f_{c,0,d} \times y \times b - V \quad (14.26)$$

The distance y, *see figure 14.15*, is obtained from equilibrium:

$$y = h\left(1 - \sqrt{1 - \frac{2M + v \times h}{f_{c,0,d} \times b \times h^2}}\right) \quad (14.27)$$

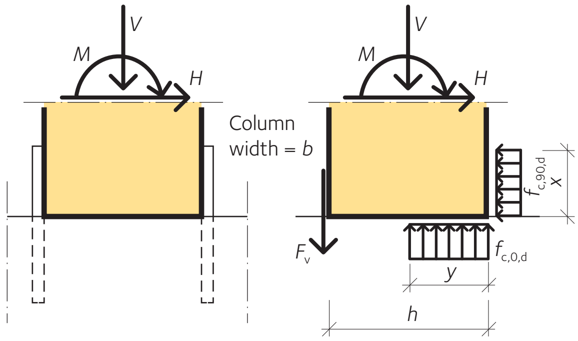


Figure 14.15 Calculation model for rigidly fixed column base (for compressive normal force)

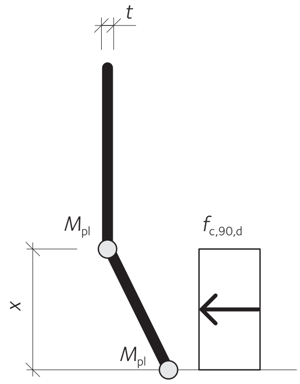


Figure 14.16 Calculation model for a steel plate supporting horizontal load by bending

The horizontal force is transferred to the rigidly connected steel plate as a uniformly distributed load. The height x of the stress volume can be calculated using plasticity theory assuming two plastic hinges. The plastic bending moment is given by:

14.28 $$M_{pl} = \frac{c \times t^2}{4} f_{y,d}$$

where c is the width of the steel plate, t its thickness and $f_{y,d}$ is the yield strength of the steel. Considering moment equilibrium we obtain:

14.29 $$2M_{pl} = \frac{c \times x^2}{2} f_{c,90,d}$$

where $f_{c,90,d}$ is the strength of the glulam in compression perpendicular to the grain. Thus, the sought variable can be expressed as:

14.30 $$x = t\sqrt{\frac{f_{y,d}}{f_{c,90,d}}}$$

And the capacity can be expressed as:

14.31 $$F_{E,y} \leq c \times x \times f_{c,90,d}$$

For a complete design it is also necessary to check the connection to the ground using EC2 and EC3.

Slotted-in steel plates

Another possibility, which also has the benefit of producing an almost invisible connection, is to use slotted-in steel plates, *see figure 14.17*. The plates can be welded to a base plate, which in turn is bolted or cast into the foundation. Apart from aesthetic benefits, being hidden, provides increased fire resistance for the connection.

The following failure modes must be checked:

- Shear of the connection (dowel action).
- Block and plug failure of the connection.
- Normal force in the steel plate, gross and net cross-sections, including risk of buckling.
- Contact between steel plate and column.

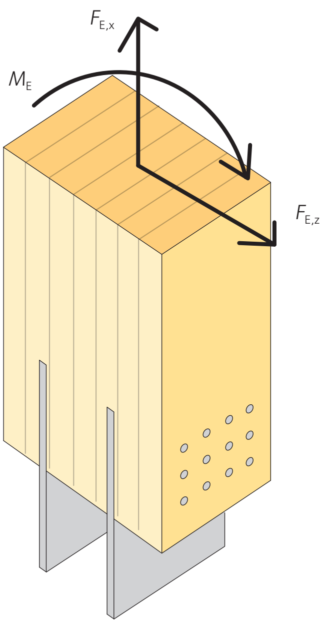


Figure 14.17 Slotted-in steel plates for fixed column base

The shear of the connection (dowel action) is done according to *EC5, section 8.3* and block/plug shear according to *EC5, Appendix A*. The capacity of the steel plate can be calculated according to *section 14.3.1, page 209*. In addition, the edge distances of the connectors must be checked. Concerning the distance between connectors, it is advisable (for simplicity) to place them far enough apart to avoid having to take account of the effect of the number of connectors in a row.

A reasonable calculation model to adopt is similar to the one described for flat steel plates: the moment is taken by a force couple with a cantilever equal to the distance between the plates. The vertical force is taken by contact between the column and the foundation in compression, and for uplift, the force is taken by the dowel action in the connection. The horizontal force is taken by the contact between the glulam and the steel plates as described previously for other connection types, with the only difference being that in the present case all plates can contribute since they are slotted-in.

Since the slots introduce a considerable weakening of the cross-section, the design must include the effect of the diminished cross-section.

For a complete design it is also necessary to check the connection to the ground using EC2 and EC3.

Bonded-in rods

Another possible solution for a rigidly fixed column base is to use bonded-in rods, *see figure 14.18*. One advantage of this type of joint is that it is practically invisible, another advantage is the fire resistance gained by having the steel parts embedded into the timber. This type of joint should not be used in connections exposed to dynamic loads, or in service class 3. The gluing needs to be carried out under controlled conditions, and is therefore performed at the factory (the procedure is done under separate factory production control routines). The most commonly used version of this connection consists of the bonded-in rod and a steel plate, which in turn is welded or bolted to the foundation.

The following failure modes have to be checked:

- Failure of the rod.
- Pull-out of the rod.
- Shear failure of the wood at the rod.
- Failure of the wood in splitting, tension, or compression.

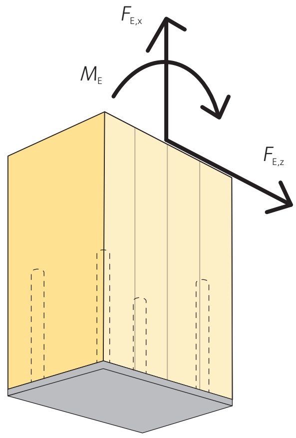


Figure 14.18 **Fixed column base with bonded-in rods**

The design load per rod is determined according to *equations 14.24, page 211, and 14.25, page 211*.

A reasonable calculation model is to assume that the moment is taken by force couples in the bonded-in rods. The vertical force can be taken also in the rods or, as an alternative for compressive vertical force and, if the geometric design makes such an assumption reasonable, by contact between the column and the ground. It is also necessary to check the interaction between shear force (transversely loaded rods) and normal force. Such interaction formulas are given in e.g. the national approval no. 1396/78.

For a complete design it is also necessary to check the connection to the ground using EC2 and EC3.

14.4 Beam-to-column connections

Beam-to-column connections are often made as pinned connections, transferring vertical and horizontal forces only (no moment). A variety of designs exist, but today steel hangers of some kind are most commonly used. The manufacturers of steel hangers normally provide a wide range of products as well as design recommendations. For large dimensions of beams and columns, as is commonly found in glulam structures, suitable standard products are normally not available. In such cases, specially designed steel hangers have to be ordered at a higher cost. If small load levels are to be transferred, a very simple and versatile connection is the one made from long self-tapping screws applied at an angle.

Riding school, Gävle, Sweden.

14.4.1 Screws applied at an angle

The use of long self-tapping screws applied at an angle in relation to the grain direction (and beam, axis), *see figure 14.19*, has become popular during the 21st century. This has been made possible thanks to the development of longer screws and screws with larger diameters.

When designing these types of connections, it is assumed that the shear force and any tensile normal force in the beam are taken by the screws in a combination of shear (dowel action) and pull-out. A compressive normal force can be transferred to the column by contact between the beam end and the column.

The following failure modes have to be checked:

- Shear capacity of the screws (dowel action).
- Pull-out capacity of the screws and pull-through capacity of the screws.
- Interaction between shear and pull-out of the screws.
- Tensile strength and shear strength of the screw (steel failure).
- Interaction between shear and tension.
- Contact pressure between beam end and column.

The shear capacity of the joint is calculated according to *EC5, section 8.2* and the pull-out strength according to *EC5, section 8.7*. The interaction between the shear and the pull-out of the screws can be calculated according to *EC5, section 8.7.3*. The capacity of the screw in terms of the strength of the steel material is determined according to *EC3*. In addition to this, it is necessary to check for the end distances, both with regard to shear and with regard to pull-out. The use of pair-wise slanted screws in X-formation, *see figure 14.19*, makes it possible to fit in more screws for a given width. It can, however, be difficult to fulfil the requirement of the end distance, $a_{1,CG}$ according to *EC5, table 8.6* ($a_{1,CG} \geq 10d$).

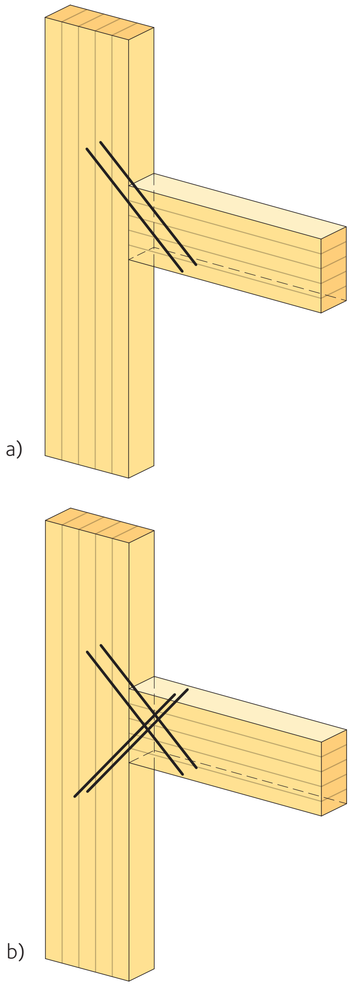


Figure 14.19 **Beam-to-column connection with self-tapping screws**
a) Two screws at an angle (in tension from the dominant downward load),
b) four screws in pair-wise X-pattern (in compression and tension from vertical load).

14.4.2 Welded hangers

If larger forces need to be transferred, welded hangers can be used. *Figure 14.20, page 217*, shows two types of hangers. The main difference between these is the visibility of the steel parts. Apart from

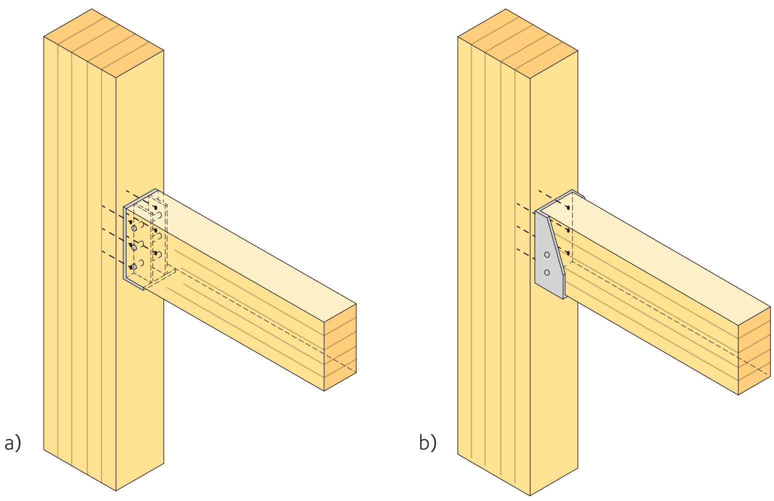


Figure 14.20 **Two different types of beam hangers**
a) With slotted-in plates,
b) with external plates.

being, in most cases, a more aesthetic solution, having the steel parts embedded into the glulam is beneficial from a fire resistance point of view. For both types shown here, the force transfer is mainly in terms of the contact pressure between the steel and the beam, which is then transferred through dowel action (shear) in the connectors to the column.

Below is an example of a possible calculation model to use for a beam hanger based on a slotted-in steel plate. A detailed design example is presented in *The Glulam Handbook Volume 3*.

Conservatory

Welded beam hanger, slotted-in

The following is a detailed presentation of design aspects for a beam-to-column connection using a beam hanger based on a slotted-in steel plate and 'running-through' dowels, *see figure 14.21*. The beam hanger is in turn secured to the column by screws through the back plate. The connection is exposed to a shear force, $F_{E,x}$ and a horizontal axial force $F_{E,y}$ which can be tensile or compressive. It is assumed that the shear force is balanced by the contact pressure between the beam and the bottom steel plate. This force is then transferred by the welds to the mid-plate and further by the mid-plate welds to the back plate. There the force is transferred through the shear (dowel action) of the screws into the column. Due to the eccentricity *e* of the force acting on the bottom plate there will be a moment to be taken by contact pressure between the bottom part of the back plate and the column, and by tension in the upper screws. A compressive horizontal force is transferred through contact between the beam end, back plate and column. A tensile horizontal force is transferred through dowel action into the mid-plate, through the mid-plate welds to the back plate and through tension in the screws into the column. The screws between the back plate and the column are thus typically exposed to a combination of pull-out and shear, and have to be designed for that combination.

The following failure modes have to be checked:

- Contact between beam and bottom plate.
- Shear of beam dowels.
- Block and plug shear at the beam end.
- Welds of the hanger.
- Pull-out of screws in column.
- Shear of screws in column.
- Interaction between pull-out and shear for screws in column.
- Contact between back plate and column.
- Splitting failure according to *section 14.2.4, page 201*.
- Bending and shearing of steel plates.

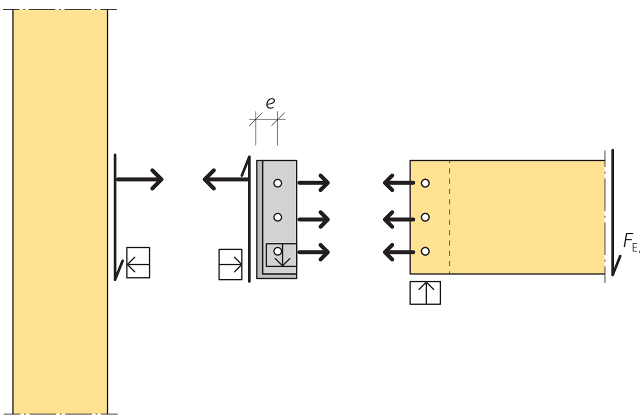


Figure 14.21 Static model for welded beam hanger with slotted-in plate

Contact pressure is checked according to *EC5, section 6.1.5*. Shear in the connectors (both dowels and screws) is checked according to *EC5, section 8.2*, while the pull-out for the screws is checked according to *EC5, section 8.7*. Block and plug shear is checked according to *EC5, Appendix A*.

Welded beam hanger with external mount

The hanger with external mount, *see figure 14.20 b), page 217*, can be designed with the same calculation model as the hanger with the slotted-in steel plates. The only difference is that the connection transferring the horizontal force is designed as a steel-timber joint with externally mounted plates.

Kolmården zoo, Sweden.

14.5 Beam-to-beam connections

For beam-to-beam connections, hangers supplied by several manufacturers are readily available for small to moderate beam dimensions. These manufacturers also supply design values to be used, and general recommendations for the installation of their products. For larger dimensions, however, one has normally to resort to specially designed and manufactured steel details, often designed as some kind of hanger. These hangers can be intended for fixing to the side of the primary beam, or for being mounted over the primary beam, *see figure 14.22*, a top-mounted hanger. From the point of view of force transfer, the hanger mounted over the primary beam is preferred. It is possible to design the hangers so they can transfer not only the shear forces and normal forces, but also bending moments. In order to increase the degree of bracing of the beam, steel angles can be mounted at the top of the beam.

If the connection is single-sided, a twisting moment (torque) will be introduced in the primary beam. This has to be taken into account in the design of the primary beam. The risk of splitting failure in the connection is increased the lower the connection point is on the primary beam.

14.5.1 Top mounted hanger with external plates

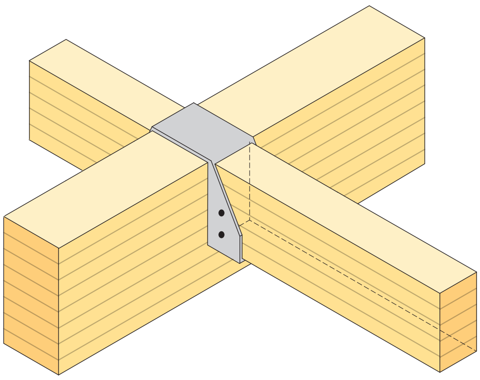


Figure 14.22 **Top-mounted hanger**

The main advantage of the top mounted hanger, *see figure 14.22*, is that the vertical force is introduced into the primary beam by contact pressure. Thus, relatively large vertical forces can be transferred.

The force is transferred from the secondary beam by contact with the bottom plate. This force is then transferred through the vertical plates of the hangers and via contact to the top of the primary beam. Due to the eccentricity of the vertical load on the bottom plate of the hanger in relation to the contact point with the primary beam, a moment has also to be transferred. This moment is taken by contact between the back plate of the hanger and the primary beam and by tension in the top plate of the hanger. A compressive normal force in the secondary beam is transferred by contact of the secondary beam end to the back plate and to the side of the primary beam. A tensile normal force in the secondary beam is transferred by the connectors of the beam hanger into the hanger. In the case of a non-symmetric load situation this will give rise to a torque acting on the primary beam, which has to be taken into account in the design of that primary beam.

The failure modes that have to be checked for this type of connection are:

- Contact between secondary beam and bottom plate of the hanger.
- Contact between top plate and top of primary beam.
- Shear in connectors.
- Welds of the hanger.
- Contact between back plate and side of primary beam.
- Bending and shearing of steel plates.

In addition it is of course necessary to check all end and edge distances.

14.5.2 Single-sided top-mounted hanger

A possible solution for a single-sided top-mounted hanger is shown in *figure 14.23*. This solution is designed such that the stiff end-plate of the secondary beam can transfer the (small) bending moment close to the support. Thanks to its stiffness, the end plate can transfer the vertical support action to the centre of the primary beam, without introducing any torque (twist of) in the primary beam. Thus, the top-plate must be stiff and also designed such that the force introduction is ensured at the centre of the primary beam.

For single sided hangers in general, it is extremely important to include in the design of the primary beam the possible effect of the torque introduced by a non-symmetric load introduction. If, as shown with the example in *figure 14.23*, such non-symmetric load introduction can be avoided, the design is much simplified. The following failure modes must, nevertheless, always be checked:

- Contact between secondary beam and bottom plate of hanger.
- Contact between top plate and top of primary beam.
- Shear in the connectors in the hanger.
- Welds of the hanger.
- Bending and shearing of steel plates.

In addition it is of course necessary to check all end and edge distances.

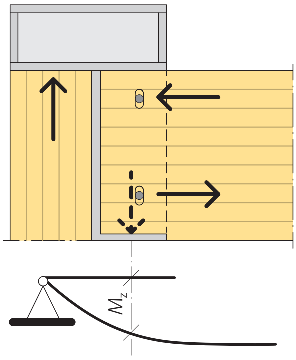


Figure 14.23 Single-sided, top-mounted hanger

14.5.3 Side-mounted hangers

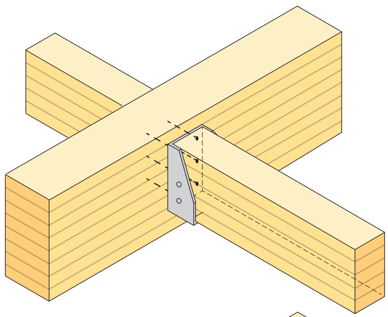
It is also possible to use side-mounted hangers, *see figure 14.24*, in which case screws are used for fixing the hanger to the side(s) of the primary beam. These hangers work in the same way as the beam-to-column connections previously dealt with, *see section 14.4.2, page 216*. However, note that the main loading now is in a direction perpendicular to the grain in the primary beam, as opposed the situation with a column. This calls for some caution, and the risk of splitting must be considered, *see section 14.2.4, page 201*. The hanger must be mounted as high up as possible on the primary beam, referring to *figure 14.7*, $h_e > 0.7h$.

The following failure modes must be checked:

- Contact between hanger bottom plate and beam.
- Shear of dowels.
- Block and plug failure at beam end.
- Welds of the hanger.
- Pull-out of screws in primary beam.
- Shear of screws in primary beam.
- Interaction of pull-out and shear in primary beam.
- Contact between hanger back plate and primary beam.
- Splitting failure according to *Eurocode 5, section 8.1.4*, or *section 14.2.4, page 201* in this Volume.
- Bending and shearing of steel plates.

In addition it is of course necessary to check all end and edge distances.

a)



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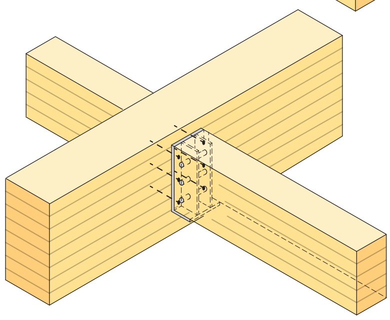

Figure 14.24 Hangers side-mounted by means of screwing
a) External plates,
b) slotted-in plates.

14.6 Column top

The connection between a beam and the top of a column is often designed as a pinned connection, and thus it should only transfer vertical and horizontal forces. In order to reduce the risk of cracking it is important to allow for rotational changes in the connection. Thus, the force transmitting parts of the connection should be placed as close to the re-entrant corner (the inner edge of the column) as possible.

Experience dictates that a suitable edge-spacing between the inner edge of the column and the centre of the screw is 4 *d* if the connector transfers horizontal forces, and 3 *d* if the connector only transfers vertical forces. If nail plates and anchor nails/screws are used, the corresponding spacing is 10 *d* and 5 *d*.

The connection can be designed with externally mounted nail plates transferring the forces. If a hidden mount is desired for aesthetic reasons, bonded-in rods can be used. The latter are, however, only possible to use in service class 1 and 2, and for moderate load levels. A third option is to use a recessed connection between the beam and the column. That design is often used at the gables to transfer horizontal loads to the column. In some cases it is possible to have some stabilising action to the beam by the connection, but in many cases this cannot be assumed. In such cases, it is important to make sure that e.g. transverse lateral buckling does not occur.

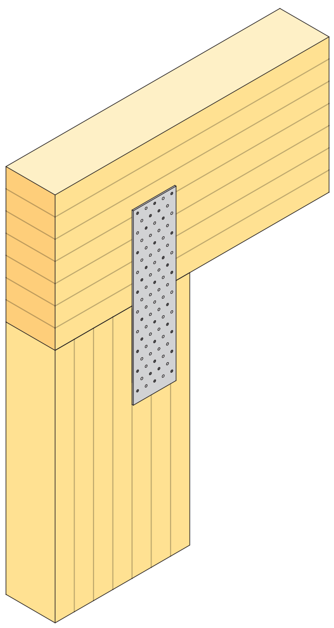

Figure 14.25 Column top with externally mounted nail plates

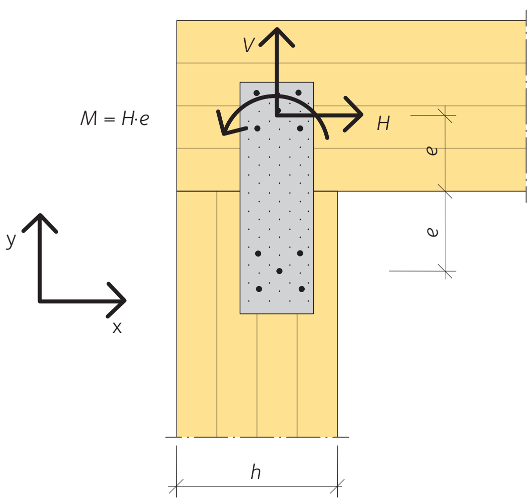

Figure 14.26 Forces and moment acting on the top half of the connection (at its centre of gravity)

14.6.1 Externally mounted plates

Nail plates

For moderate load levels, nail plates are a good alternative. From an economical point of view, it is desirable to choose plates with punched holes, which means that the steel plate thickness is not greater than the diameter of the hole. The diameter of the hole is chosen to be 1 mm larger than the diameter of the fastener – so typically a 5 mm hole would be used for a 4 mm anchor nail.

Below is a brief overview of a possible calculation model. A detailed design example is given in *The Glulam Handbook, Volume 3*.

The basic assumption made is that the externally mounted steel plates act as beams that are rigidly fixed in both ends. The forces to be transferred are a horizontal force and possibly a vertical uplift force. Any vertical loads acting in compression are assumed to be taken by contact between the glulam beam and the glulam column.

Failure modes to be checked are:

- Shear capacity of the connectors.
- The strength of the steel plates.
- Possible buckling of the steel plates.
- Contact pressure between glulam beam and column.

The forces, a horizontal force H and any possible vertical uplift V, are assumed to act at the centre of gravity of the connection, *see figure 14.26*. A moment due to the eccentricity of the horizontal force, $H \times e$, and due to the assumption of the plate being rigidly connected at its both ends, is also introduced at the centre of gravity of the connection.

Forces acting on a single connector are given by:

$$F_Y = \frac{V}{n} + \frac{H \times e \times r_{xi}}{I_p} \qquad 14.32$$

$$F_X = \frac{H}{n} - \frac{H \times e \times r_{yi}}{I_p} \qquad 14.33$$

where:

- n is the number of connectors (usually anchor nails or anchor screws).
- e is the eccentricity of the horizontal force (half the distance between centres of gravity).
- r_{xi}, r_{yi} are the distances in the x- and y- directions between the centre of gravity and a single connector.
- I_p is the polar moment of inertia of the connection.

$$I_p = \sum_{i=1}^{n} \left(r_{xi}{}^2 + r_{yi}{}^2 \right) \qquad 14.34$$

The resulting force on a single connector is thus:

$$F_E = \sqrt{F_x^2 + F_y^2} \qquad 14.35$$

For nails with a diameter of less than 8 mm no consideration of the influence of the load to grain angle has to be taken. The value of the capacity for a single connector, $F_{v,Rk}$, is given by *EC5, section 8.3*. A first estimate of the number of nails needed can be obtained by:

$$n_{prel} = \frac{\sqrt{H^2 + V^2}}{F_{v,Rk}} \qquad 14.36$$

Following this, an appropriate nail configuration (pattern) can be chosen. This choice then includes the number of nails, their arrangement in rows and columns, including the connector spacing and edge and end distances. A final check is then made of the mostly stressed connector in the joint using the above expressions, which include the eccentricity moment and the polar moment of inertia. For simplicity it is worth choosing the distance between the nails in a row in such a manner that no consideration has to be taken regarding the number of connectors in a row. Block and plug shear failure is checked according to *EC5, Appendix A*. It is not necessary to check the risk of splitting, if the distance from the edge of the column to the connector furthest away from that edge exceeds 0.7 times the column width h. Otherwise, the risk of splitting failure has to be checked following the provisions of *EC5, section 8.1.4*, or *section 14.2.4, page 201* in this Volume.
The steel plate will be loaded by both the horizontal force and the vertical force, *see figure 14.27, page 222*. In addition to this there is the eccentricity moment acting at the centre of gravity of the connection.

Thus for the steel plate at a distance e_2 from the centre of gravity, the bending moment is given by:

$$M_E = F_{E,z} \times \left(e_1 - e_2 \right) \qquad 14.37$$

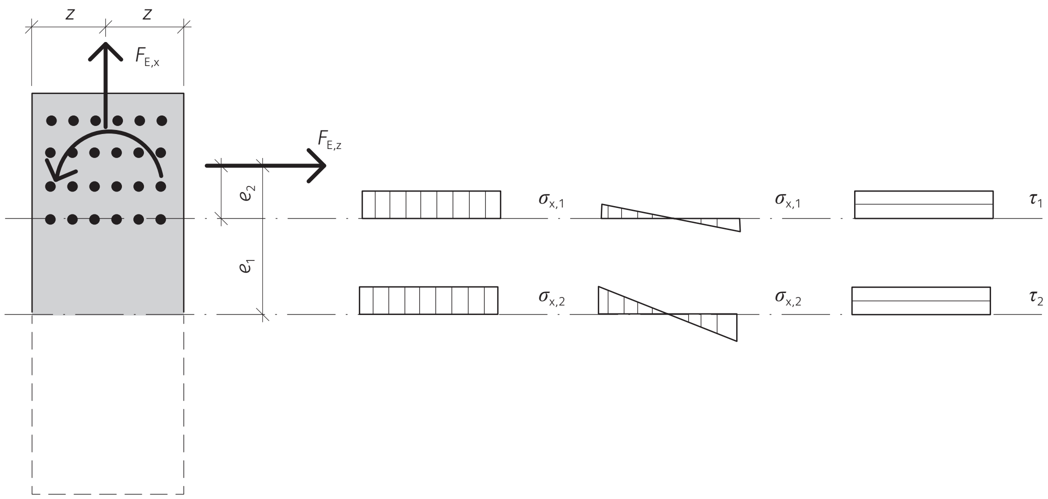

Figure 14.27 Stress distribution in the steel plate

Knowing the bending moment and the vertical and horizontal forces, it is then possible to calculate the stress distribution in the steel plate at various sections. The contribution due to the vertical force is given by:

14.38
$$\sigma_{xi} = \frac{F_{Ex}}{A}$$

where the area A of the steel cross section is, when appropriate, corrected for the holes. The contribution due to the bending moment is given by:

14.39
$$\sigma_{xi} = \frac{M_E z}{I}$$

Here z is half the plate width and I is the second moment of inertia of the plate (where appropriate I is calculated taking the holes into account). The shear stress to be transferred is given by (assuming a perfectly plastic behaviour of the steel plate):

14.40
$$\tau_i = \frac{F_{E,z}}{A}$$

where the area A is, when appropriate, corrected for the holes.

The combined effect of the normal stress and shear stress should then be checked with the interaction formula given in *section 14.2.5, page 206*.

For thin plates it may be necessary to check the risk of buckling of the plate. If the requirements on maximum distance between the connectors are fulfilled, this is, however, not necessary. The requirements are that the distance between the holes must not exceed 14*t* (plate thickness) or 200 mm, whatever of these two is smaller.

Pergola

Steel bars with rectangular or U-shaped cross-sections

For column top connections where large forces are to be transmitted, it can be reasonable to use steel bars with either rectangular or U-shaped cross-section, *see figure 14.28,* instead of using steel plates. The use of such steel bars with bolts is beneficial not only because of the increased load transferring capability, but also because of the possibility of introducing some transverse stabilising action for the beam, thus reducing the risk of transverse lateral buckling. In order to facilitate the assembly, the holes are drilled with a diameter 1 mm larger than the bolt diameter.

The design procedure for this type of connection is similar to the previous one discussed. Using bolts, the diameter would typically exceed 8 mm, and thus the influence of load to grain angle must be taken into account.

14.6.2 Bonded-in rods

By using bonded in rods it is possible to obtain a concealed connection. The threaded rod is bonded into the column and the beam is then mounted over the rod and is fastened with a washer and nut, *see figure 14.29*. The washer and the nut can be hidden through countersinking. Designs using bonded-in rods cannot be used in service class 3, or for columns exposed to dynamic loading. The gluing is done in a factory. In order to account for moisture movements, the use of bonded-in rods should be limited to beams with depths not exceeding 500 mm. For beams slightly deeper than this, this limitation can be bypassed by countersinking to a level less than 500 mm above the column top.

A bonded-in rod does not provide any lateral restraint and thus lateral stabilisation of the beam must be provided by other means.

14.6.3 Recessed beams

So-called recessed beams are often used in gable walls in order to improve the horizontal load transferring capacity. The force transfer is often by means of bolts. If loads are small, steel angles can also be used.

The bolts will transfer any vertical uplift forces and will also provide support through tensile action for horizontal forces. Horizontal forces towards the column top are taken by contact between the side of the beam and the notched column top. Vertical compressive forces are transferred to the column by contact between the beam and the column at the notch. The beam will introduce an eccentrically placed normal force into the column, and this eccentricity has to be taken into account in the design. If the contact pressure is too large, the contact area between the beam and the column can be increased by the use of a steel support plate.

Nordens Ark zoo, Hunnebostrand, Sweden.

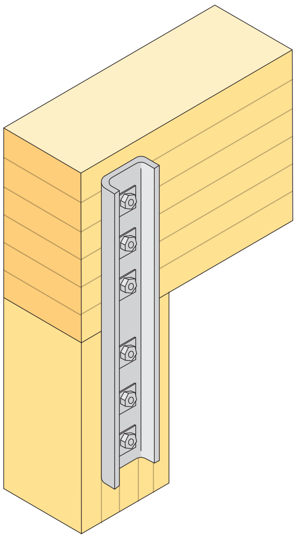

Figure 14.28 **U-shaped steel bars with running-through bolts**

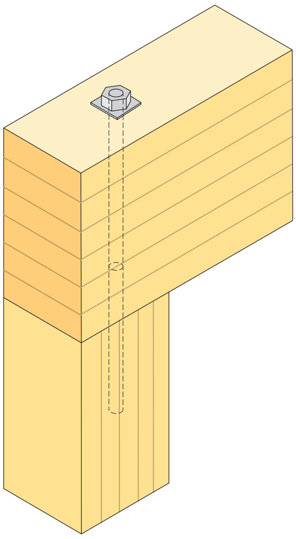

Figure 14.29 **Column top with a bonded-in rod**

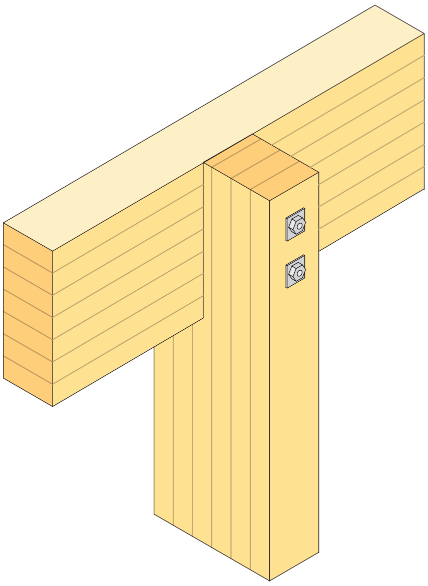

Figure 14.30 **Column top with a recessed beam and through bolts**

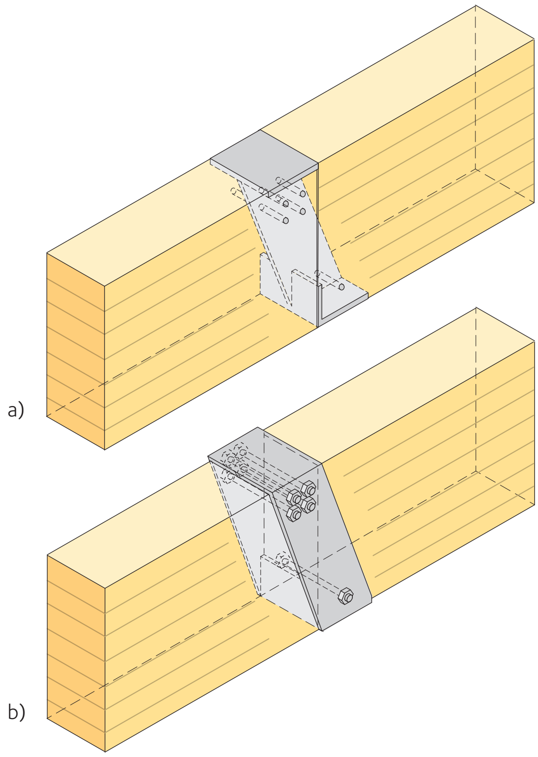

Figure 14.31 Gerber joints with
a) Slotted in steel plate,
b) external steel plates.

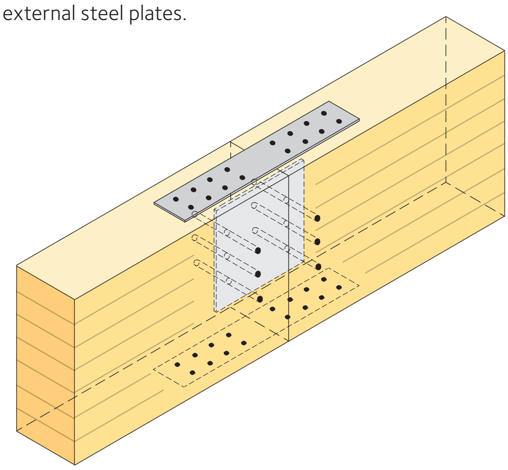

Figure 14.32 Moment-resisting beam joint

14.7 Joints – beam joints, ridge joints, Gerber joints

Pinned beam joints

A pinned beam joint transfers vertical forces, and in many cases also horizontal forces have to be transferred. In a pinned joint no bending moment should be transferred. Thus such a beam joint should be designed in a way that does not restrain the beam cross-sectional rotation. This will ensure that no moment is being transferred in the case of e.g. unwanted deformation of the beam support, and thus reduces the risk of inducing high stress concentrations. In *figure 14.31*, see below, examples of these, so-called Gerber joints are shown. For their use in purlins, *see section 12.2, page 168*. Another possibility is to use nail plates, and for small loads even long self-tapping screws mounted pair-wise in X-formation could be an option.

Gerber joints should be used whenever considerable shear forces are to be transmitted. Such steel joint details are available in standardised dimensions for small and moderate beam sizes. For larger beam sizes, it might be necessary to have them specially made for a single project. The joint type is designed to transfer a shear force in one direction only, although it is possible to transfer small shear loads in the “wrong” direction by the use of screws.

In order not to hinder the free rotation of the beam ends, it is important to place such screws as close to the top and bottom steel plates as possible. If the joint is exposed to tensile normal force, extra flat steel straps can be welded onto the joint. Two different types exist: with externally mounted vertical steel plates or with a centrically placed, slotted-in, vertical steel plate. The latter is beneficial from an aesthetic point of view, and also from a fire resistance point of view.

When designing Gerber joints it is assumed that shear forces are transferred through contact between the top and bottom plates, and the top and bottom surfaces of the beams. The force is assumed to act at the centre of the plate. Due to the eccentricity of this force, a moment is transferred within the joint itself. This moment is taken by contact between the vertical plate at the lower edge of the beam and through dowel action at the top of the beam. A detailed design example is given in *The Glulam Handbook Volume 3*.

Moment-resisting beam joint

A moment-resisting joint for a beam can be made by use of a vertical, slotted-in steel plate, which is intended to carry the shear force. The bending moment is taken by contact between the beam ends on the compression side, and by a horizontal nail plate on the tension side. It is not possible to obtain a joint as stiff and strong as the full beam cross-section, and thus this type of joint should only be used for the case of minor bending moments in relation to the moment capacity of the full beam cross-section.

Pinned ridge joint

This joint is typically designed to transfer horizontal and vertical loads. Moments are not to be transferred and are therefore not considered in the design. It is thus essential to design the joints such that no constraints are built into the joint, i.e. it is important to allow for the ends to rotate in relation to each other. In order to assure this, it is often a good idea to have the top half of the beams cut as indicated in *figure 14.33* and *14.34, page 225*.

Two different designs of pinned ridge joints are shown here. The first relates to a joint with externally mounted nail plates to take the shear force. For cases where the shear force is of considerable magnitude, it is also possible to use a slotted-in steel bar with T-shape, *see figure 14.34*. A general rule of thumb is that, in order to allow for the differential rotation of the beam ends, it is regarded as good practice to place the fixings as close as possible to the lower edge of the beams.

Concerning the joint with the externally mounted nail plates, the nail plate is regarded as a beam with fully clamped supports at both ends, *cf. section 14.6.1, page 220*. The forces to be transmitted are a vertical force and a horizontal force and both are assumed to act at the centre of gravity of the group of nails at either end of the joint, *see figure 14.26*. The eccentricity of the centres of gravity in relation to the ends of the beams introduces an extra moment to be taken by the groups of nails. The design has to account for the capacity of the nail plate fasteners (shear by dowel action in the nails), the nail plate itself (tension, shear and bending), and the risk of splitting failure.

Considering the joint based on the use of a slotted-in, T-shaped bar, *see figure 14.34*, it is assumed that the nail plate is loaded by a horizontal force and that the vertical force, i.e. the shear force in the beam is transferred by contact between the steel bar and the glulam. The design has to account for the capacity of the nail plate joints (shear by dowel action in the nails) and of the nail plate itself (tension), the welds of the T-shaped bar, contact between steel bar and glulam, and the risk of splitting failure.

Swimming facility, Lindesberg, Sweden.

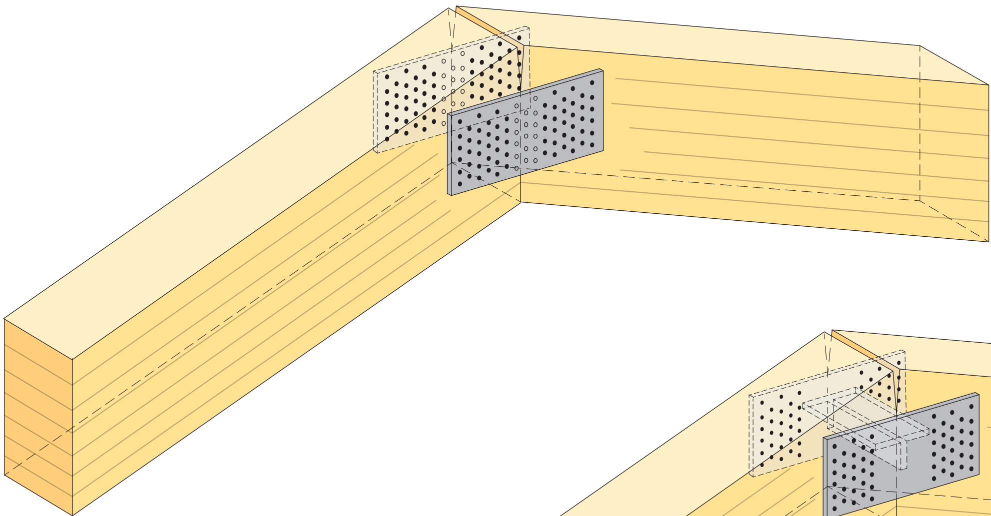

Figure 14.33 Pinned ridge joint based on externally mounted nail plates

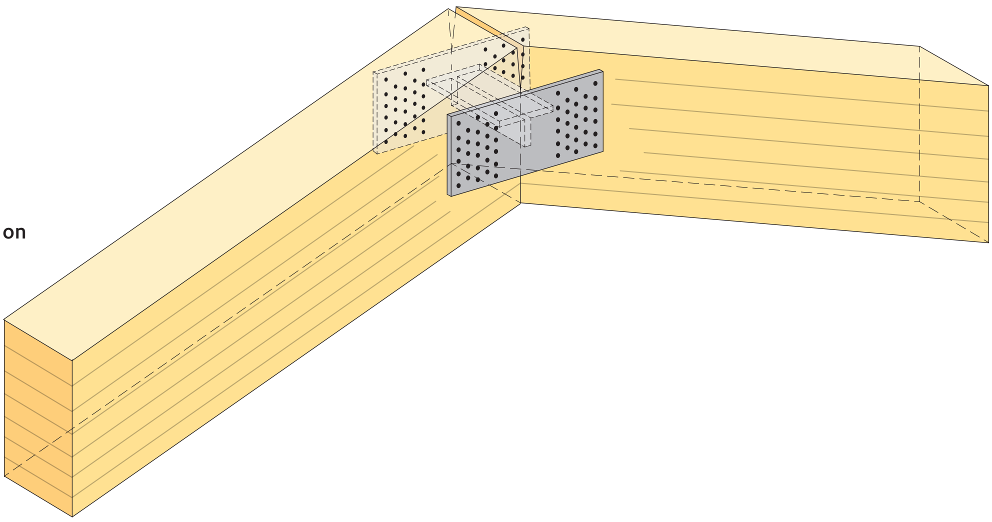

Figure 14.34 Pinned ridge joint made from slotted-in T-shaped bar

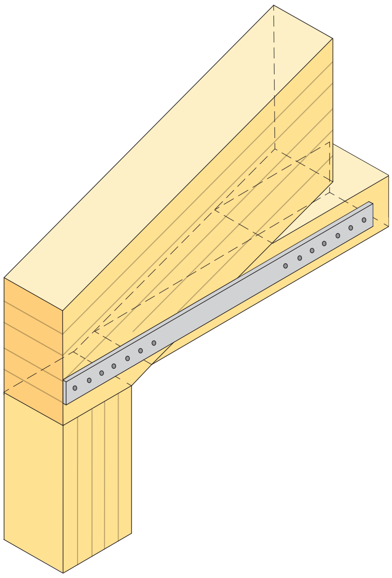

Figure 14.35 **Tie made from glulam with nail plate to transfer the load at its end**

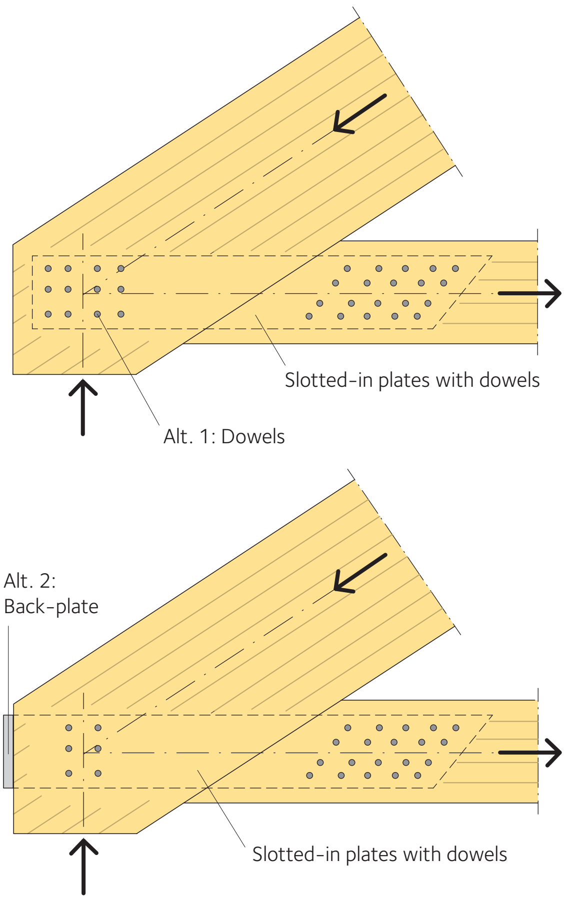

Figure 14.36 **Tie made from glulam with slotted-in steel plates to transfer the load at its end**
Note that the design must take account of the tie's reduced cross-sectional area due to the slot and dowels.

14.8 Tie fixings

Ties are often used to transfer the tensile force of the lower chord in a roof truss. The tie is only capable of transferring tension. It is recommended to make sure that the force introduced by the tie acts as closely as possible to the intersection of the system lines of the beam and the column.

If the tensile forces to be taken by the tie are small, it is possible to use glulam ties. In such a case nail plates or flat rectangular steel bars are used for the force transfer, *see figure 14.35*. Another alternative could be to use slotted-in steel plates, *see figure 14.36*.

For large tensile forces, high strength steel ties are normally used. Two different approaches for the tie fixings are common. Technically, the simplest way is to use two externally placed tie rods, one on either side of the main roof beam. These are fastened to an end plate that transfers the tension loads to the end of the main beam through contact pressure. For moderate tensile forces it is also possible to use a single, centrally placed tie rod fastened via a hole drilled through the beam, *see figure 14.37 b)*. If beam depths are large, and/or if the roof slope is small, such holes are both difficult and expensive to drill, and in some cases even impossible.

For all types of tie fixings discussed here, it is necessary to provide lateral stability of the beam end by other means than by the tie fixings.

The design of the tie fixings includes checking:

- Tensile capacity of the tie rod (including nut and washer capacity).
- Tensile capacity of the nail plate and the nailed connection.
- Compressive stresses at an angle to the grain from the end steel plate into the glulam.

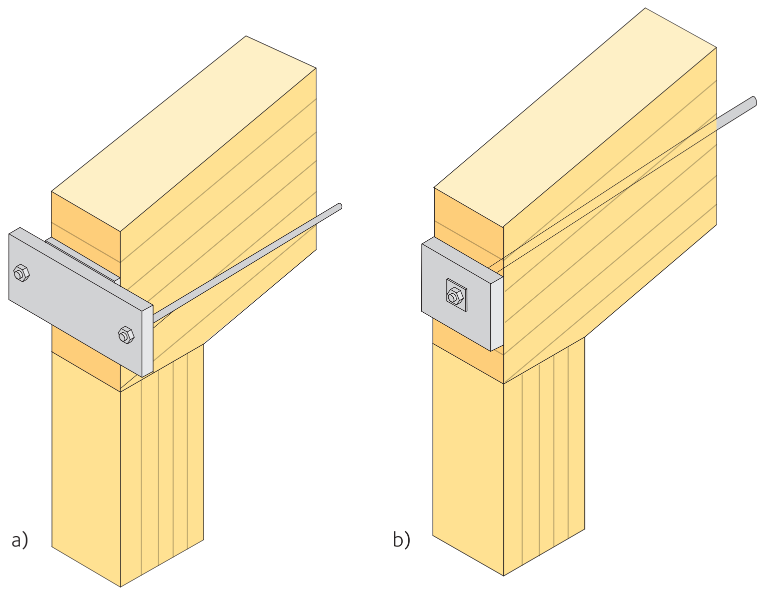

Figure 14.37 **Steel tie rods**
a) Two externally mounted tie rods,
b) one single tie rod.

14.9 Truss joints

Truss joints are often designed (multiple) dowel-type connections, based on the use of dowels, bolts or nails in combination with nail plates, special external steel fixings or slotted-in steel plates. Only in rare cases are glued joints used, since it is difficult to have an efficient force transfer in a glued joint with large area. Of special concern regarding the design of truss joints is the use of joints without eccentricity (i.e. making sure that system lines meet in a single point) and to reduce the extension of the joints as much as possible.

In some cases it could, however, be advantageous to design the truss joint with some eccentricity, although keeping the plate and dowel configuration without eccentricity, *see figure 14.39*. The advantages are related to the fact that smaller plates and smaller slot depths can be used. This is especially the case in large trusses, where large-dimension tension members imply that the distance to the centre of gravity of the tension member is very large. The eccentricity of the joint gives rise to a bending moment, which can either increase or decrease the loading on the chord. This must be taken into account in the design. The important thing in this case is that the joint itself (i.e. the plate and dowel configuration) is made without eccentricity.

Truss joints should, by definition, be designed for normal forces and shear forces only (a true truss consists of hinged joints). In practice, however, it is often necessary to take into account the moment resisting capacity of the joint. The design of the dowel type connection is done according to *EC5, section 8.2*. The single connector capacity is calculated according to *EC5, sections 8.3, 8.5, 8.6 or 8.7* depending on the type of connector being used.

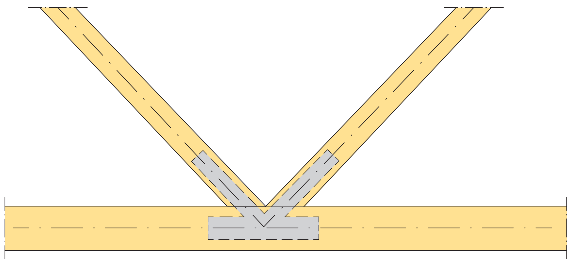

Figure 14.38 **Truss joint based on slotted-in steel plates**

Sports arena, Eksjö, Sweden.

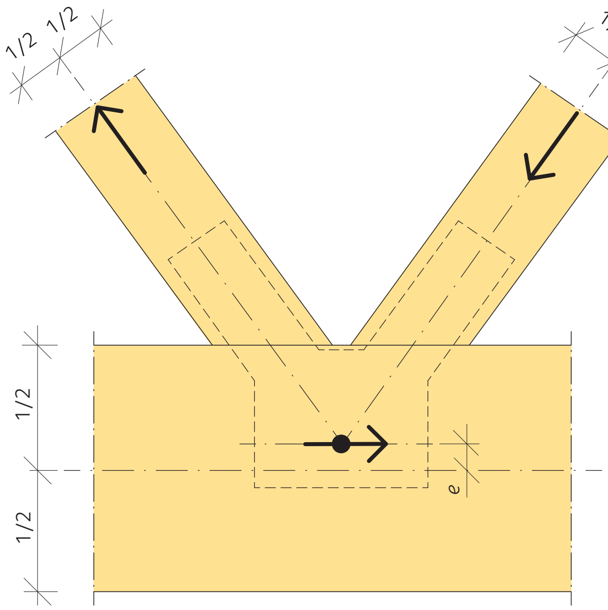

Figure 14.39 **Eccentric truss joint based on slotted-in steel plates**
The design must take account of the reduced cross-section due to the slot and dowels.

Detailing of glulam structures

Local failures and total collapses have occurred in glulam structures due to different reasons, also discussed in previous chapters, e.g.:

- Introduction of a hole with inappropriate placement and/or size of timber members, *see Chapter 5, page 70*.
- Introduction of too deep a notch in timber members, *see Chapter 5, page 70*.
- Curved and pitched cambered beams with too small a radius of curvature and/or too steep a slope of the extrados, *see Chapter 7, page 102*.
- Upside down tapered beams, i.e. beams with the tapered edge located at the tension side, *see Chapter 7, page 102*.
- Insufficient bracing, *see Chapter 13, page 170*.

The above-mentioned issues will not be discussed any further in this chapter (15). This chapter will deal with the detailing of glulam structures to avoid failures that have not been dealt with in the previous chapters. Proper detailing is important for the structural performance, durability and serviceability of any timber structure. However, the larger sizes and longer spans made possible with glulam components make the proper detailing of connections even more critical.

The basic design principles for timber structures can be divided into three broad categories:

- To allow efficient transfer of forces and avoid (or at least reduce) tension perpendicular to the grain.
- To allow the dimensional changes due to variation in moisture content of the wood members that otherwise can lead to tension perpendicular to the grain.
- To prevent wood decay.

The following pages contain figures that illustrate various detail types. These illustrative examples show common incorrect details along with examples of corresponding correct details. A description of the failures that may occur due to the incorrect detailing is also given. Note that the detailing presented is of a general nature and is not fully complete.

15.1 Detailing for efficient transfer of forces

Timber structures should be designed so that load transfer between members always occurs in a clear manner, i.e. without the possibility of ambiguous load paths. Moreover, it is vital to choose details that do not generate perpendicular to grain stresses during loading of the structure.

15.1.1 Heavy concentrated loads suspended from beam

Heavy concentrated loads, such as heating and air conditioning units, crane rails or main framing members suspended from the bottom of beams, induce tension perpendicular-to-grain stresses and may cause splits as shown in *figure 15.1 a)*. *Figures 15.1 b) and c)* show two possible methods to avoid risk of splitting.

15.1.2 Beam-to-column connection

When two simply supported timber beams have a common support – which could be a column made of either timber, concrete or steel – often U-brackets or T-brackets of steel are used to provide a connection between the beams and the column, *see figure 15.2 a) and b)* respectively. If the fasteners are located too far away from the contact surface between column and beam, they will restrain the beam rotation that is due to deflection under loading. This can cause splitting of the beams. Furthermore, if the beam shrinks, the bearing load may be transferred to the bolts. Shrinkage of the timber beam can also cause splitting of the beam and buckling of the steel bracket if this is too slender, *see figure 15.2 b)*. *Figure 15.2 c)* shows a possible method to avoid the risk of splitting.

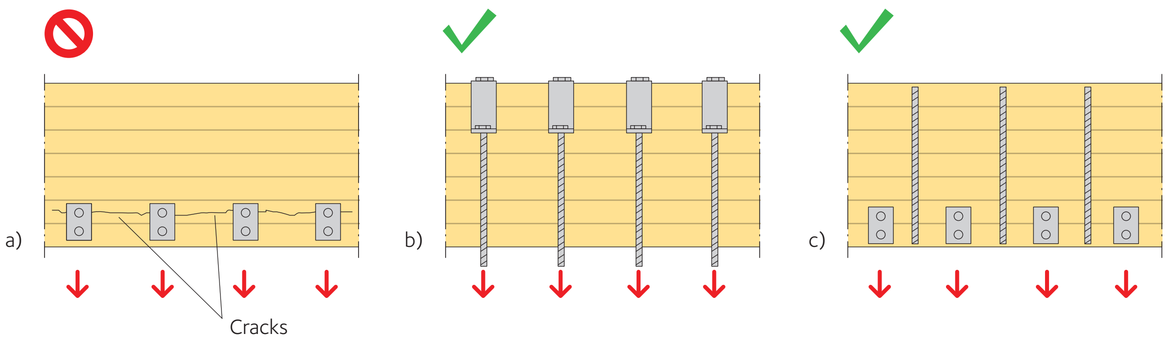

Figure 15.1 Beams subjected to heavy loads
a) Loads suspended from beam may generate splits.
b) Loads transferred in compression bearing.
c) Reinforcement by means of glued-in rods or self-tapping screws.

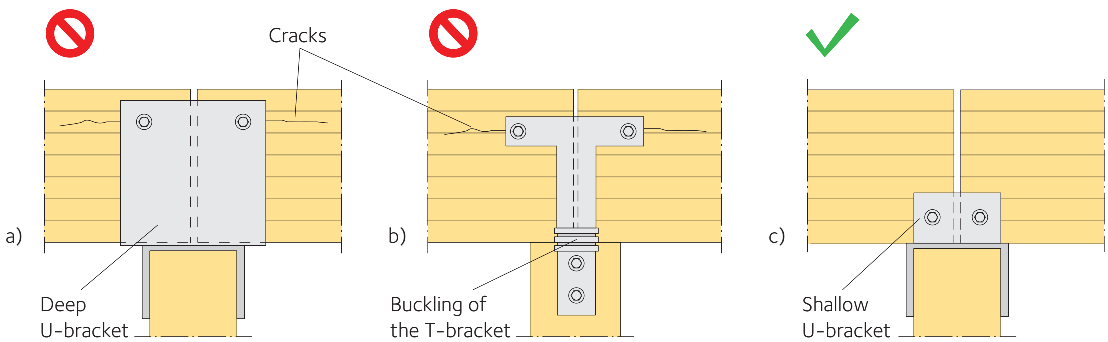

Figure 15.2 Beam-to-column connections
a) Connection with deep U-bracket.
b) Connection with T-bracket.
c) Connection with shallow U-bracket.

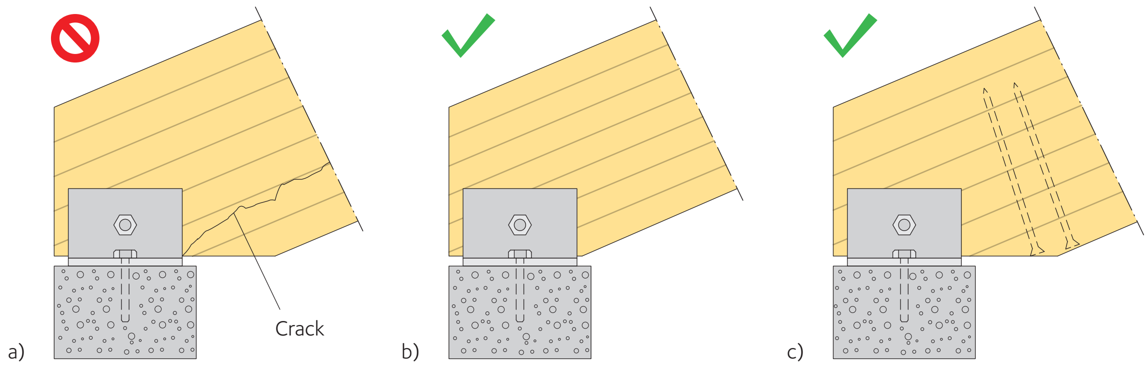

Figure 15.3 **Supports of inclined beam member or arch**
a) Horizontal sawn surface of the timber member is longer than the support length.
b) Horizontal sawn surface of the timber member entirely in contact with the underlying column or wall.
c) Reinforcement by means of glued-in rods or self-tapping screws.

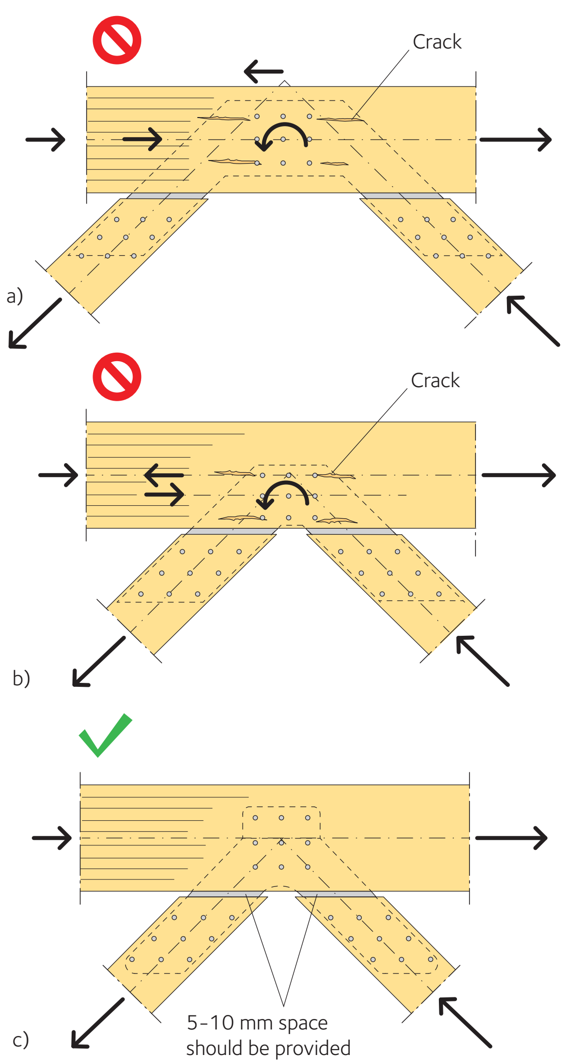

Figure 15.4 **Truss nodes with slotted-in plates and dowels**
a) Eccentricity due to the fact that the centre of rotation of the fastener group in the chord is located below the meeting point of the centre lines of the web members.
b) Eccentricity because the centre lines of the fastener group of the top chord and those of the web members do not meet at the same point.
c) Node without eccentricities.

15.1.3 Support of inclined struts, beams or arches

The support of inclined timber members or arch members should be designed in such a way that the horizontal sawn surface of the timber is entirely in contact with the underlying column or wall. Induced tension perpendicular-to-grain stresses, in combination with shear stresses, occur if the horizontal sawn surface of the timber member is longer than the support length. This phenomenon is similar to the one described in *Chapter 5, page 70*, and can lead to splitting of the timber. *Figure 15.3 b) and c)* show two possible methods to avoid the risk of splitting.

15.1.4 Eccentricity at truss nodes

Eccentricities at the nodes should be avoided in all cases, mainly due to the risk of splitting. Such eccentricities, in fact, give rise to secondary bending moments, which in turn generate tensile stresses perpendicular to the grain. Eccentricity may be due to the centre lines of the chord and of the web members not meeting at the same point as shown in *figure 15.4 a)*; or, albeit member centre lines meeting at the same point, the centre of rotation of the fastener group in the chord is located away from such a point as shown in *figures 15.4 b)*.

Most often, it is possible to design truss nodes without eccentricities. An example of such a truss node is shown in *figure 15.4 c)*.

15.1.5 Truss nodes with long steel plates in web members

Under loading, the node of a truss rotates. Fixed-angle gusset plates, especially long ones, prevent timber truss members from rotating under load. This may induce moments at the ends of members, which can cause splitting of web members at fastener locations, *see figure 15.5 a)*. The risk of splitting for this connection could be reduced by decreasing the distance between fasteners in the web members. *Figures 15.5 b) and c)* show two other possible methods which avoid the risk of splitting.

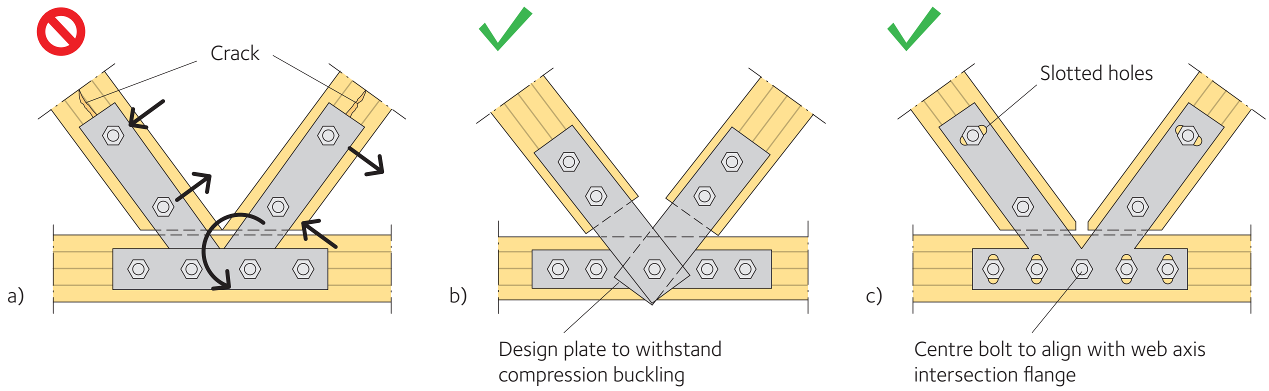

Figure 15.5 Truss nodes with external plates and bolts
a) Fixed-angle gusset plates prevent timber truss members from rotating under load.
b) Node with separate plates to joint truss members, with pinned connection at the intersection point.
c) Node with slotted holes to allow for unrestrained rotation of members.

15.1.6 Beam-to-beam connection at different levels

Occasionally, primary and secondary beams are placed at different levels. Application of load via fasteners below the neutral axis can cause a tension perpendicular-to-grain failure in the primary beam (i.e. the deeper beam of *figure 15.6 a)*). Location of the majority of fasteners above the neutral axis as shown in *figure 15.6 b)* or use of a top mounted hanger will minimize the possibility of splitting of the primary beam. Another alternative is to reinforce the beam by means of self-tapping screws or glued-in rods, *see figure 15.6 c)*.

Note, however, that if secondary beams are attached only at one side of the primary beam as shown in all assemblies in *figure 15.6*, torsion occurs in the primary beam — which has to be taken into account during design.

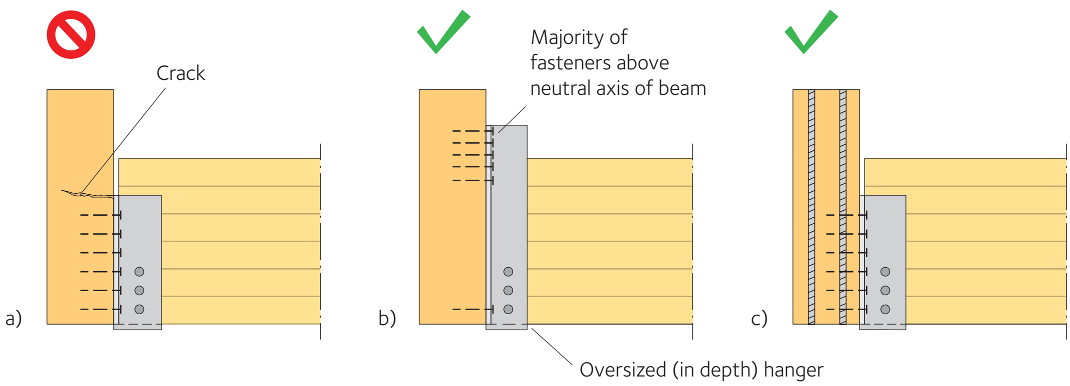

Figure 15.6 Primary beam to secondary beam connections
a) Application of load via fasteners below the neutral axis can cause a tension perpendicular-to-grain failure in the beam.
b) Oversized hanger, to allow for application of the majority of fasteners above the neutral axis.
c) Reinforcement by means of glued-in rods or self-tapping screws.

15.2 Detailing to allow for dimensional changes due to variation in moisture content

As explained in *Chapter 14, page 198*, careful consideration of moisture-related expansion and contraction characteristics of wood is essential when detailing glulam connections in order to prevent induced tension perpendicular-to-grain stresses. Changes in the moisture content of the timber will cause the timber to swell and shrink. The dimensional changes in the direction parallel to the grain can be ignored in most cases. On the other hand, the dimensional change in the perpendicular-to-grain direction can be large, especially if the moisture content variation is large and/or if the cross-sectional depth of the member is large.

15.2.1 Knee joint of frames

Occasionally, the connection of knee-jointed frames is made with dowels passing through three pieces of wood overlapping in the knee joint. Typically, the rafter is made up of a single glulam beam, while the column consists of two parallel glulam elements on each side of the rafter, see *figure 15.7 a)*.

Moisture content in wood will normally decrease some time after the erection of the frame, if the structure is located in an indoor environment. When moisture content in the wood members decreases, shrinkage in both the column and the rafter will occur, mainly in the perpendicular-to-grain direction. However, the connectors prevent such a shrinkage, which increases the risk of the timber splitting.

Splitting of the timber at the knee joint has a negative influence both on the moment capacity of the joint and on the shear strength of the members.

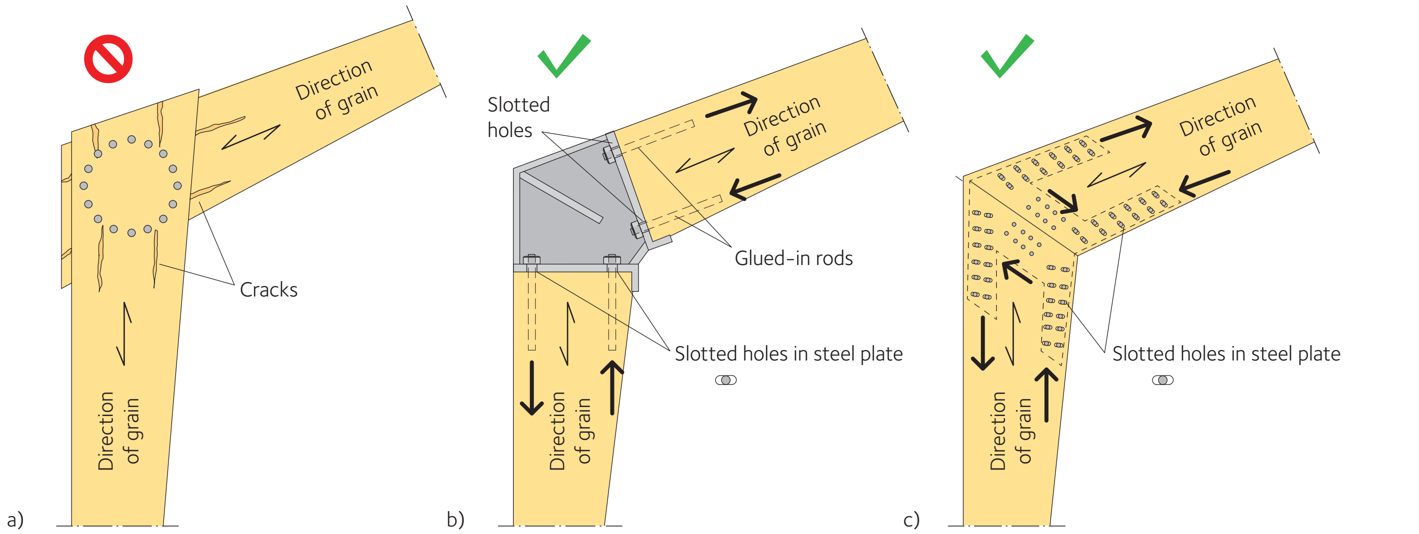

Figure 15.7 **Knee joints of frames**
a) Dowel connection with fasteners passing through the rafter (a single glulam beam) and column (two parallel glulam elements on each side of the rafter).
b) Knee joint consisting of a steel bracket connected to rafter and column by means of glued-in rods.
c) Slotted-in plates and dowels.

One way to avoid splitting in such types of structures is to use finger-jointed haunches, as described in *section 10.5.2, page 148*. Splitting can also be avoided by using separate steel elements at the knee of the frame. *Figure 15.7 b), page 232*, shows a knee consisting of a steel bracket connected to the rafter and the column by means of glued-in rods. *Figure 15.7 c), page 232*, shows a knee joint with one or more slotted-in plates and dowels.

15.2.2 Beam-to-beam connection

A Gerber type hanger with integral tension-tie (i.e. a horizontal plate welded to the Gerber type hanger) can cause tension perpendicular-to-grain stress to develop — and thus possible splitting — due to shrinkage of timber elements, *see figure 15.8 a)*. If a tension connection is required, a separate plate not connected to the Gerber type hanger may be used as shown in *figure 15.8 b)*. Alternatively, an integral tension-tie with slotted holes can be used, *see figure 15.8 c)*.

15.2.3 Beam-to-beam connection

Beam-to-beam connections can be designed in a multitude of different ways. Common methods involve the use of angle brackets or saddle hangers. Angle brackets with long rows of fasteners can cause splits to form in the suspended beam, as shown in *figure 15.9 a)*, due to tension perpendicular-to-grain stresses induced at the bolts due to beam shrinkage.

A saddle hanger with side plates and fasteners placed at the upper part of the suspended beam can also cause splits to form in the suspended beam, as shown in *figure 15.9 b)*. This is due to beam shrinkage, which lifts the beam off from the (horizontal) bearing plate and transfers the loads to the fasteners.

Figure 15.9 c) shows a connection consisting of a reinforced saddle hanger with slotted holes, which allows for free shrinkage movements of the suspended beam. The connection is also able to take the bending moment due to eccentricity between the row of bolts in the suspended beam and the axis of the main beam. Obviously, the fasteners must be designed to resist the moment caused by the eccentricity.

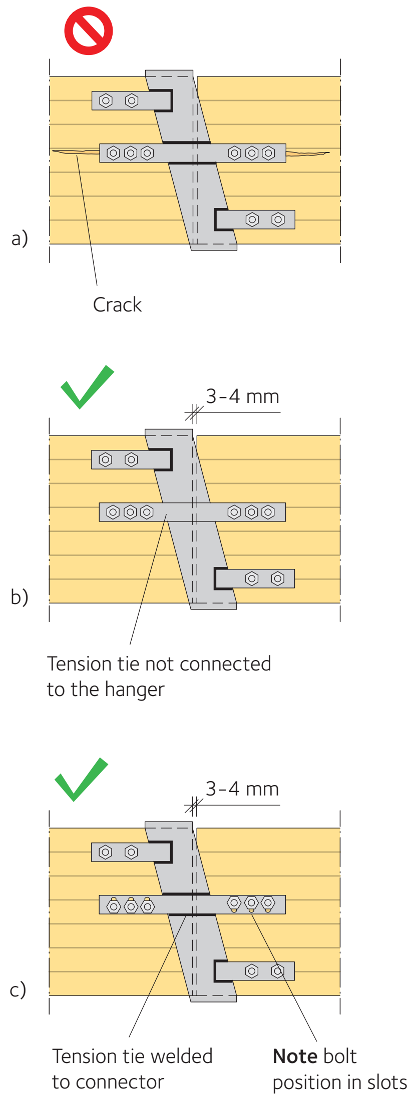

Figure 15.8 Gerber type hangers with tension-tie
a) Integral tension-tie can cause tension perpendicular-to-grain stress to develop due to restrained shrinkage of the members.
b) Tension-tie made with a separate plate.
c) Integral tension-tie with slotted holes.

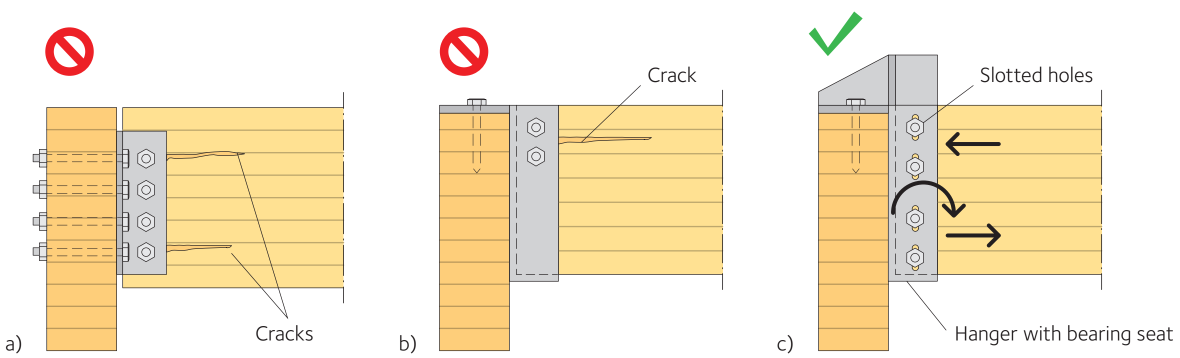

Figure 15.9 Beam-to-beam connections
a) Angle brackets with long rows of fasteners.
b) Saddle hanger with side plates and fasteners placed at the upper part of the suspended beam.
c) Reinforced saddle hanger with slotted holes.

15.3 Detailing to prevent wood decay

As discussed in *Chapter 1, page 8*, it is important to design glulam structures to insulate and protect all timber members from potential sources of excessive moisture. Moisture content in excess of approximately 20 % may promote the growth of decay-causing organisms, which in turn produce rot.

The best way to prevent rot is to keep the timber members as dry as possible. If this is not possible, the members must be designed so that the timber can dry out quickly after wetting.

The discussion in this section does not include pressure-treatment of the timber with protective fluids. The examples shown are mainly related to outdoor structures, such as bridges, towers, masts, etc., where the risk of high moisture contents in timber members is normally much more significant than for indoor structures.

15.3.1 Wood column to concrete base

Direct contact between wood and concrete is to be avoided because the concrete of the supports is likely to be a wet bedding, which can lead to humidification of the wood. This will not dry out easily and will also give rise to capillary suction, *see figure 15.10 a)*. Possible solutions to avoid humidification of the wood – especially for an outdoor environment – consist of separating the wood from the concrete and keeping the end-grain at the bottom face of the column at least 150–300 mm from the ground level, *see figure 15.10 b)* and *c)*.

For structures in service class 1 (i.e. an environment with a temperature of 20 °C and relative humidity that seldom exceeds 65 %), it is often sufficient to place a moisture barrier at the bottom of the timber column to prevent capillary suction, for example tempered wet-process hardboard.

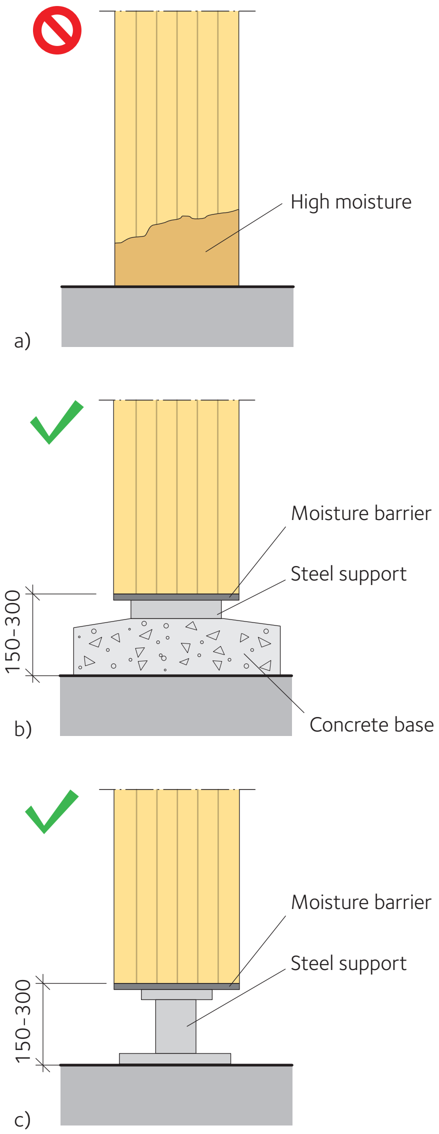

Figure 15.10 **Column base**
a) Timber column in direct contact with concrete. Solutions b) and c) can be used for humid environments, e.g. outdoors.
b)Timber column on pedestal of concrete and steel.
c) Timber column on pedestal made entirely of steel.

15.3.2 Water traps

Water accumulation in joints must be avoided. Metal shoes similar to that shown in *figure 15.11 a)* have been used as a base connection for e.g. columns, frames and arches. These kinds of connections cause formation of water/moisture retention zones and should be avoided.

An improved detail is shown in *figure 15.11 b)*, where a bracket keeping the column at a proper distance from the concrete foundation is shown. Self-tapping screws arranged at 45 degrees to the grain are used to connect the bracket to the column. A moisture barrier is attached at the bottom of the column to prevent capillary suction.

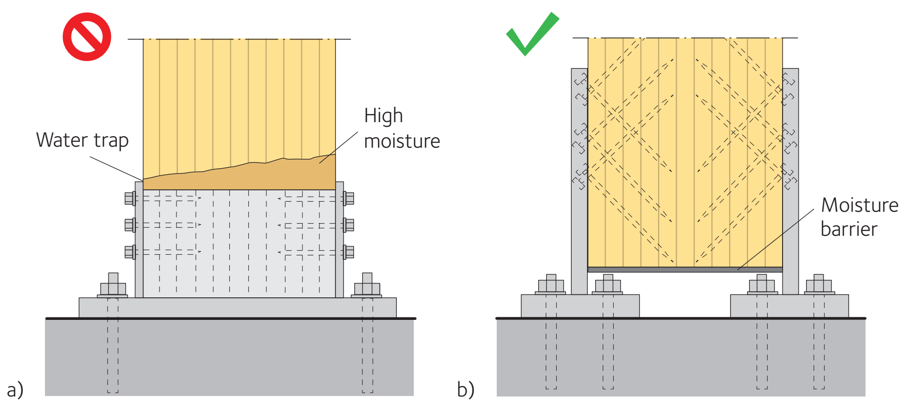

Figure 15.11 **Bracket at the base of a timber member (post, frame or arch)**
a) Water retention in the metal shoe.
b) Bracket with inclined self-tapping screws and moisture barrier at the end grain; the column is kept at a distance from the foundation to allow for ventilation.

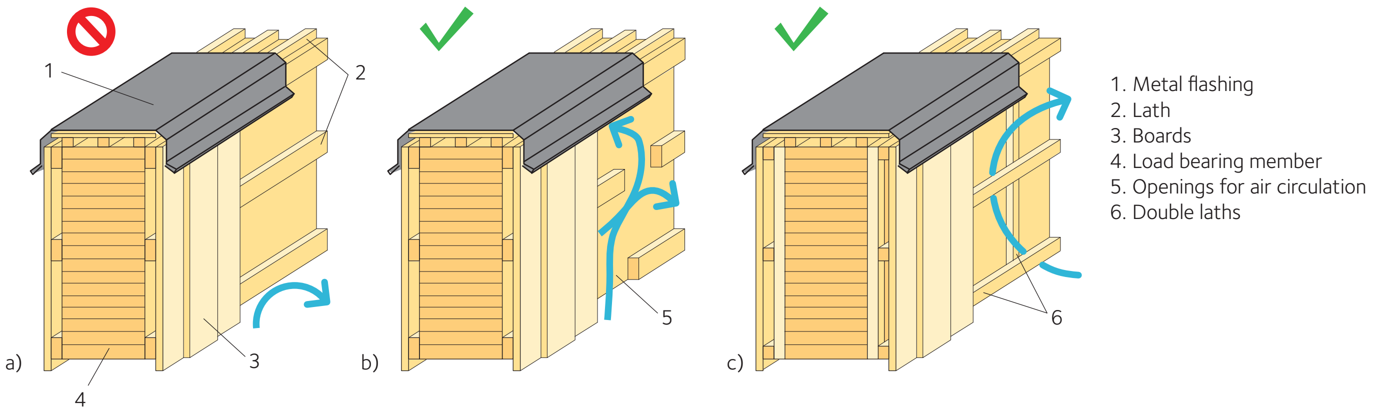

Figure 15.12 **Protection of load-bearing members**
a) With longitudinal continuous laths; air circulation is not properly permitted.
b) With longitudinal laths with openings; some limitation of the air circulation.
c) With horizontally and vertically arranged laths; air circulation is good.
Note that for the clarity of the drawing the protection arrangement at the end of the load-bearing member is not shown.

15.3.3 Board coating of load-bearing members

Protection of exposed timber members is often achieved by means of wooden boards, with thickness 22 – 34 mm and width 145 – 195 mm. To prevent large distortions, the ratio between width and thickness of a board should not exceed 8. The boards may be arranged horizontally or vertically; in either case, however, it is important to ensure good ventilation between the coating and the protected timber member.

To ensure good ventilation, the boards are usually spaced by a network of laths. Arrangements according to *figure 15.12* are commonly adopted. The arrangement according to *figure 15.12 a)* does not permit proper air circulation. The arrangement according to *figure 15.12 b)* allows air circulation, though in a limited fashion. The best solution is shown in *figure 15.12 c)* with laths arranged both horizontally and vertically, which allows good air circulation.

Note that the end of the timber member should always be carefully protected, e.g. by means of metal flashing or boards fixed on laths to allow for ventilation.

15.4 Summary

The illustrations in this chapter have been provided to show examples of both correct and incorrect methods of detailing glulam structures. The selected details serve to emphasize seven basic principles:
1. Allow for dimensional changes in glulam due to potential in-service moisture cycling.
2. Avoid the use of details that induce tension perpendicular-to-grain stresses in a member.
3. Avoid moisture entrapment at connections.
4. Allow for good ventilation of the glulam members.
5. Do not place glulam in direct contact with masonry, concrete or any other potential source of humidity.
6. Avoid eccentricity in joint details.
7. Minimize exposure of end grain.

Glulam and fire

16.1 Combustibility and aspects of thermal decay

Why choose a combustible material, such as timber and glulam, for structural elements that must ensure a given level of fire resistance?

Let us observe the gradual change (evolution) of the mechanical properties of some building materials when exposed to a "standard" fire, *see figure 16.1*. The parameters are measured with reference to the performance of physically defined elements. For all materials except wood, the test piece size and shape do not have a significant effect: for these materials, moreover, a constant temperature (slightly lower than the environment temperature) over the whole section can be hypothesized at any time, and it is therefore correct to think that all material properties vary accordingly. In wood by contrast, there is no significant temperature increase under the charred layer, and the material properties consequently remain unchanged.

Wood seems therefore to feature a better pattern, but what is being observed is not the temperature induced evolution of the material properties, but the evolution of performances for an element with a given (here 50 mm × 50 mm) initial cross-section, that is the decrease of the resisting section during the exposure to fire. The advantage, when utilising wood, does not lie in the variation of its mechanical parameters with temperature, but in its slow and predictable mass thermal 'evolution'.

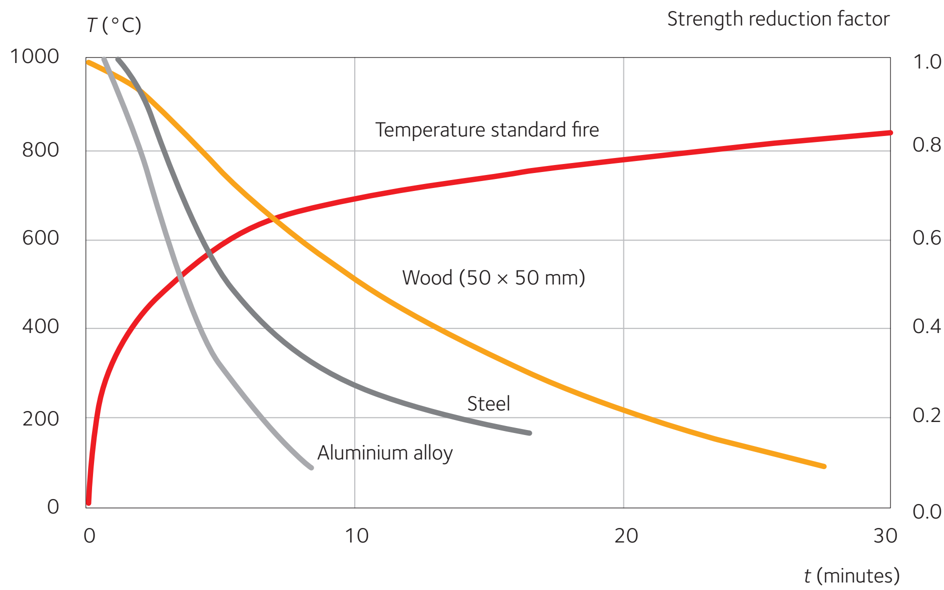

Figure 16.1 Evolution of the mechanical properties of some building materials when exposed to a standard fire

16.1.1 Combustibility

As wood is an organic material of vegetal origin, it is combustible, and can therefore be completely destroyed in the event of a fire, thus losing any physical and mechanical characteristics, when exposed to heat sources of sufficient intensity and duration: this results directly from its chemical composition, wood consisting up to 50 percent of carbon.

Wood combustion occurs by thermal decomposition of its constituent materials. This process is complex, generating more than 200 substances, but since wood tissues mostly consist of cellulose, hemi cellulose (polysaccharides with molecular weight lower than that of cellulose) and lignin, wood combustion clearly depends on the pyrolysis of these substances.

It must, however, be borne in mind that the behaviour of timber structures under fire cannot be explained by wood chemical composition alone. Material singularities play an important role, especially in the collapse phase.

Two spruce glulam beams with the same composition, equally loaded and both exposed to a standardized fire according to EN 1363-1 (previously ISO 834), exhibit decidedly different behaviour, the first one not undergoing failure, the test being interrupted at the highest deflection required by the standard, *see figure 16.2*, while the other collapsed much earlier, *see figure 16.3*.

This behaviour is clearly dependent on the presence of a knot in the third lamella; this feature has a negligible effect on the element before the attack, but becomes a fatal defect when exposed, because of the destruction of the outer lamellae.

Figure 16.2 Glulam beam without significant singularities after a fire

Figure 16.3 Glulam beam with a significant knot in the third lamella after a fire

16.1.2 Aspects of the thermal decay

Pyrolysis reactions consume energy, but produce substances that, in the hot environment that generated them, are either oxidised with high energy release, or decomposed into the main combustibles that wood can produce: carbon monoxide, CO, carbon dioxide, CO_2, and water vapour, H_2O. Consequently, from a point (a temperature) onwards, the combustion is self-sustaining.

Table 16.1 schematizes the time and temperature sequence of the involved phenomena: It must be remembered that EN 1995-1-2 fixes another "no return" point, where it places the charring theoretical line on the 300 °C isotherm inside the wood mass.

The combustion (and thermal demolition) of wood proceeds from its exposed outer surface towards the inside of its mass with a determined finite rate, *see figure 16.4, page 238*. This velocity, in the same environmental conditions and for the same material properties, depends mainly on the wood species, while, among environmental factors, temperature, heat contribution and ventilation are determining.

Among the material conditions, the most significant ones are moisture content and treatments that the material may have undergone. It can therefore be said that, in a fire, the depth of destroyed material is approximately proportional to the exposure time, or, more exactly, to the duration of the charring process. Another important point to be noted is that unaffected wood, *see figure 16.4, page 238*, exhibits temperatures below 100 °C, except for a small layer (10 – 20 mm) next to the pyrolysis zone.

Table 16.1 Sequence of combustion phenomena

Temperature (°C)	Phenomena
20	Sample temperature before ignition
100	Water loss
120	Lignin plasticization
170	Pyrolysis begins
Over 170	Pyrolysis products combustion
300	Charring begins

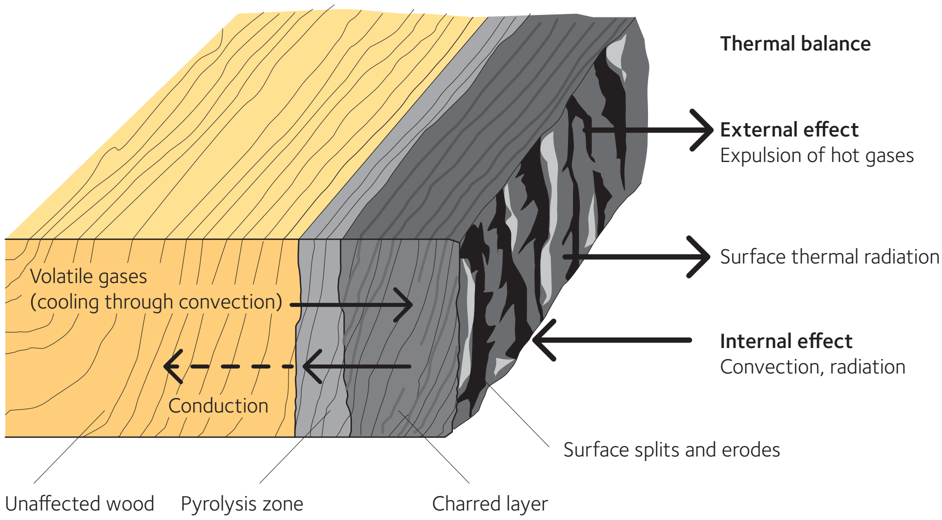

Figure 16.4 **Phenomena involved in the charring process**

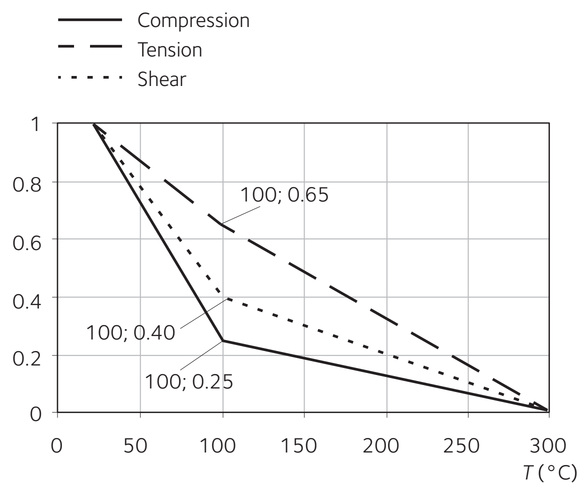

Figure 16.5 **Reduction factor for strength parallel to grain of softwood as a function of temperature and stress type**, according to *EN 1995-1-2 (Annex B, Advanced calculation methods).*

16.1.3 Temperature-induced changes in wood characteristics

Wood has poor thermal conductivity. During fire, a significant amount of heat is transmitted by mass transfer, by means of hot gas diffusion, as seems confirmed by significantly different charring rates observed in wood species characterised by close density (e.g. beech and oak), but by significant differences in gas and vapour permeability.

Moving inwards, these gases cause the temperature to increase until the wood impairment is initiated and charring begins, while in the charred zone they slow the increase of ember temperature.

If we remember that the temperature is (almost) unchanged immediately under the inner surface of the charred zone wood, we can conclude that the "good" behaviour of wood in fire is caused by:

- the constancy of mechanical properties up to temperatures of 110 – 115 °C;
- the screening action of the charred layer.

There is actually, immediately under the charred area, a very thin layer whose temperature is not high enough to start charring, but can impair wood properties. Different design standards are utilised depending on whether this aspect is considered or not. This phenomenon takes place at about 120 °C, when wood temperature in the ordinary end use conditions undergoes no significant changes ("unaffected wood" of *figure 16.4*).

Eurocode 5 (EN 1995-1-2) proposes for some mechanical properties the reduction coefficients presented in *figure 16.5*, decidedly more conservative than those that can be found in scientific literature. However, this depends on the different behaviour of clear wood test pieces and structural elements, where defects must be taken into account and whose effects become more severe as the temperature increases.

16.2 Design of fire resistant structures

Ulls Hus, Uppsala, Sweden.

Timber structures can be designed for fire safety using protective cladding and surface treatments, which leave the protected glulam beam's cross-section undamaged after a fire, with the load-bearing capacity dependent on the type of cladding and its thickness. After a fire, an unprotected glulam beam will have a reduced cross-section and thus also a reduced load-bearing capacity. Load-bearing capacity in the ultimate limit state should therefore be verified with the reduced cross-section.

Advances in building techniques, and, most of all, the test data available today, have led to approaches and technical solutions in which the designer is more directly involved.

It is especially interesting to study a wooden structure undergoing a fire, analysing parts of the structure, imposing the accidental action of fire and verifying that for each of them the following condition is satisfied [1)]:

$$A_{\mathrm{d,fi}}(t) \leq R_{\mathrm{d,fi}}(t) \qquad \mathbf{16.1}$$

where $A_{\mathrm{d,fi}}$ is the design value of the effect of action under fire conditions, $R_{\mathrm{d,fi}}$ the corresponding design resistance under the same conditions, and t the duration of fire exposure.

For the effects of the direct actions acting on the structure, the combination rule for the so-called exceptional combinations is adopted, that can be written as follows, *cf. Chapter 2, page 28, and Chapter 6, page 82*:

$$1.0 \times G_{\mathrm{k}} + 1.0 \times P_{\mathrm{k}}(t) + 1.0 \times \psi_{1.1} \times Q_{\mathrm{k},1} + 1.0 \times \Sigma\psi_{2,\mathrm{i}} \times Q_{\mathrm{k,i}} \qquad \mathbf{16.2}$$

where:

G_{k}	characteristic value of permanent actions.
$P_{\mathrm{k}}(t)$	characteristic value of pre-stressing force value (usually variable during exposure to fire).
$Q_{\mathrm{k},1}$	characteristic value of variable (principal) action.
$Q_{\mathrm{k,i}}$	characteristic values of other variable actions.
$\psi_{1,1}$	combination coefficient for the variable action assumed as principal.
$\psi_{2,\mathrm{i}}$	generic combination coefficient for other variable actions.

The values for the combination coefficients ψ are given as functions of the different categories of use for the different areas in buildings, *see EN 1991-1-1*, and usually range between 0 and 0.7. Caution should be used in those cases in which the maximum action can be foreseen during the fire event (e.g. libraries, archives and stores).

Moreover, a simplified method can be used to calculate $A_{\mathrm{d,fi}}$ when conditions are unchanged during the fire. Starting from the fundamental combination of actions A_{d} for normal temperature design, the values $A_{\mathrm{d,fi}}$ can be calculated using the following equation:

1) **Note** that the symbol E_{d} is used in EN 1995-1-2 for the "Design effect of actions", instead of the symbol A_{d} here reported. The change in the symbol is done purposely in the current chapter (16), in order not to cause misunderstanding with the symbol E_{d} used for defining the modulus of elasticity of timber in *The Glulam Handbook*.

16.3 $$A_{d,fi} = \eta_{fi} \times A_d$$

The factor η_{fi} depends on the different safety factors γ_G and γ_Q applied to the characteristic permanent and variable actions, as well as on the combination factor ψ_{fi} for frequent values of variable actions in the fire situation, given either by $\psi_{1.1}$ or $\psi_{2.1}$, *see EN 1991-1-2*, and it can be written as:

16.4 $$\eta_{fi} = \frac{G_k + \psi_{fi} \times Q_{k,1}}{\gamma_G \times G_k + \gamma_{Q,1} \times Q_{k,1}}$$

where:

γ_G is the partial safety factor for permanent actions.
$\gamma_{Q,1}$ is the partial safety factor for variable action 1.

This relationship can also be expressed in a different form:

16.5 $$\eta_{fi} = \frac{1 + \psi_{fi} \times \xi}{\gamma_G + \gamma_{Q,1} \times \xi}$$

It is thus a function of the ratio $\xi = Q_{k,1}/G_k$.

Figure 16.6 shows the η_{fi} diagrams as a function of ratio ξ and for different values of the combination coefficient $\psi_{1.1}$, assuming γ_G =1.35, $\gamma_{Q,1}$ = 1.5. Values 0.9, 0.7 and 0.5 correspond to category E (areas susceptible to accumulation of goods), C/D (meeting and shopping areas), A/B (areas for domestic and residential activities, and office areas). High values of the ξ ratio are usually featured by the so-called "lightweight" structures, such as the wooden ones.

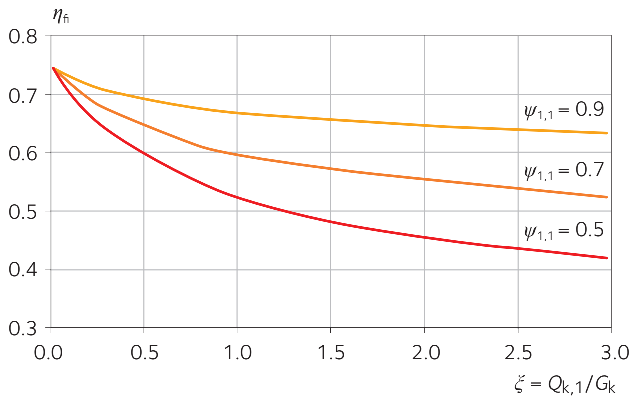

Figure 16.6 Values for η_{fi} as a function of permanent/variable actions ratio ξ

16.3 Determination of fire resistance: the European Standard approach

The timber structure's fire resistance can be calculated in accordance with Eurocode 5 (EN 1995-1-2) using a simplified method that gives satisfactory design equations and verification conditions. The following terms will then be utilised:
- Char line: transition area between charred layer and the residual cross-section.
- Residual cross-section: initial cross section minus the thickness of the charred layer.
- Effective cross-section: initial cross-section minus the thickness of the charred layer and that of an under-laying layer (d_0) whose strength and stiffness are assumed to be zero.

In European standards (EN 1995-1-2), three different design approaches are envisaged:
- Effective cross-section method.
- Reduced properties method (reduced strength and modulus) method.
- Advanced calculation methods, with reference to charring models, temperature profile and moisture gradient over the cross-section, and to wood strength and modulus variations with temperature and moisture.

The first method entails both simplicity of analysis and consistency with the physical development of the phenomenon, which is why it is examined in detail here and is used in the design examples in *The Glulam Handbook Volume 3*. For the other methods, see Eurocode 5.

Different levels of simplification are also envisaged in the European standards:
- Global structural analysis, verifying the inequality $A_{d,fi} \leq R_{d,fi}$; when effects do not increase during the fire, as usually happens, the value $\eta_{fi} = 0.6$ can be assumed, except for imposed loads according to category E (areas susceptible to accumulation of goods), where the recommended value is $\eta_{fi} = 0.7$.
- Analysis of parts of the structure, with an approximate evaluation of the interaction of the different parts of the structure.
- Analysis of single elements, where normal service conditions are taken into account as initial conditions.

In the effective cross-section method, an effective cross-section is calculated by subtracting from the initial cross-section the thickness of an effective charring depth d_{ef} given by *figure 16.7*:

$$d_{\text{ef}} = d_{\text{char,n}} + k_0 \times d_0$$ 16.6

where:

- d_{ef} the effective penetration depth (to be subtracted from the original measurement of the cross-section when calculating the effective cross-section).
- $d_{char,n}$ "notional" charring depth, $d_{char,n} = \beta_n \times t$, where β_n is a notional charring rate, including the (negative) effects of shakes and corner rounding, *see table 16.2, page 243*, as from EN 1995-1-2.
- k_0 coefficient ranging between 0 and 1 (to be defined further on).
- d_0 7 mm, highest difference between residual and effective cross-section.

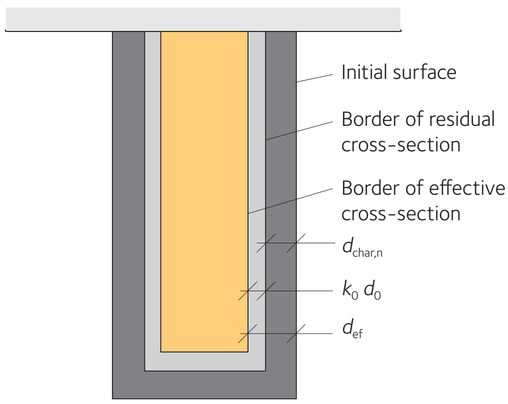

Figure 16.7 Residual and effective cross-section according to EN 1995-1-2

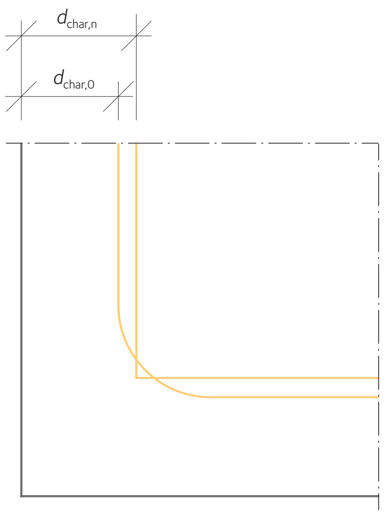

Figure 16.8 Corner induced cross section reduction according to EN 1995-1-2

If the corner rounding effect caused by simultaneous fire exposure on concurrent faces is taken into account, the charring rate β_0 can be utilised, *see figure 16.8*. Thus for a one-dimensional situation (e. g. a glued laminated beam), the charring depth can be calculated referring to a β_0 charring rate, close to the results of (one-dimensional) physical tests, *see figure 16.8*:

16.7 $$d_{\text{char},0} = \beta_0 \times t$$

The rounding radius of the corner must be assumed to be equal to the charring depth $d_{\text{char},0}$. This is allowed as long as the minimum cross-section dimension has a value greater than $b_{\min}$, which is obtained from:

16.8 $$b_{\min} = \begin{cases} 2 \times d_{\text{char},0} + 80 & \text{if } d_{\text{char},0} \geq 13\text{mm} \\ 8.15 \times d_{\text{char},0} + 80 & \text{if } d_{\text{char},0} < 13\text{mm} \end{cases}$$

If the minimum cross-section dimension is, or becomes smaller than $b_{\min}$, the β_n values apply instead. The following equation is commonly used for glulam structures:

16.9 $$d_{\text{char},n} = \beta_n \times t$$

The values for the coefficient k_0 are assumed to be equal to 1 for fire exposure times greater than 20 minutes, and linearly variable from 0 to 1 for times from 0 to 20 minutes. If a timber element is protected, the time at which $k_0 = 1$ is the lower of these two: the starting time of charring of the protected element or the collapse time of the protection.

The following equations apply to cross-section design strength and moduli, as well as for the connection resistances:

16.10 $$f_{\text{d,fi}} = k_{\text{mod,fi}} \times \frac{f_k \times k_{\text{fi}}}{\gamma_{\text{M,fi}}}$$

16.11 $$S_{\text{d,fi}} = k_{\text{mod,fi}} \times \frac{S_{0.5} \times k_{\text{fi}}}{\gamma_{\text{M,fi}}}$$

16.12 $$R_{\text{d,fi}} = \eta \frac{R_k \times k_{\text{fi}}}{\gamma_{\text{M,fi}}}$$

where f_k is the characteristic (5 % fractile) value of a strength, S_{05} the characteristic (5 % fractile) value of a modulus (E or G), and R_k is the characteristic value of a connection strength, all at normal temperature.

The following coefficients are utilized:

- k_{fi} allows to obtain the 20 % fractile values from the 5 % ones; different values for solid wood (1.25), glued laminated wood and wood based panels (1.15), wood to wood connections (1.15) and wood to steel connections (1.05) must be used.
- $\gamma_{\text{M,fi}}$ partial safety factor in fire situation (1.0).
- $k_{\text{mod,fi}}$ factor modifying mechanical properties that, for the method considered, has the value 1; it substitutes k_{mod} at normal temperature, *see EN 1995-1-1*.
- η is a reduction factor, a function of the time t of exposure to fire, as detailed further on.

Table 16.2 β_0 and β_n values for wood and wood based materials according to EN 1995-1-2

Material	β_0 (mm/min)	β_n (mm/min)
a) Softwood and beech		
• Glued laminated timber with characteristic density ≥ 290 kg/m³	0.65	0.70
• Solid timber with characteristic density ≥ 290 kg/m³	0.65	0.80
b) Hardwood		
• Solid or glued laminated timber with characteristic density ≥ 290 kg/m³	0.65	0.70
• Solid or glued laminated timber with characteristic density ≥ 450 kg/m³	0.50	0.55
c) LVL		
• with characteristic density ≥ 480 kg/m3	0.65	0.70
d) Wood-based panels		
• Wood panelling	0.90*	–
• Plywood	1.00*	–
• Wood-based panels other than plywood	0.90*	–

* These values apply to boards with a characteristic density value of 450 kg/m³ and a thickness of 20 mm. European Standard EN 1995-1-2 provides methods to derive values for different densities and thicknesses.

Charring rates for glulam are lower than those for solid wood of the same species. An obvious reason is the greater material homogeneity, but, even though thermoplastic adhesives are usually not allowed in the manufacture of glued laminated wood, caution is required because some thermosetting adhesives reach their glass-transition point, or are thermo-chemically degraded, thus losing their adhesion to the substrate and/or its shear strength, at under 150 – 160 °C.

In a glued laminated beam, decay of the glueline near the surface under the charred layer could instead cause:

- an increase of shear stress in a zone that, with both residual and effective cross-section methods, is a zone considered to contribute to the strength and stiffness of a beam;
- as a consequence, a separation of lamellae that would leave the glueline unprotected from thermal attack, thereby quickening the aforementioned process, thus making cavities occur that probably are responsible for the increases of charring rates observed by some researchers in the maximum shear stress zone of glued laminated beams exposed to a standardized fire.

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16.4 Fire resistance of connections

Metallic connection details and/or part of a structure made of metallic elements are often the weak points of a timber structure with respect to fire risk. In fact, metal connections allow heat to be transmitted by conduction into the wood mass, and all metal components may undergo, by thermal expansion, deformations incompatible with the stability of the structure.

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Table 16.3 **Fire resistances of unprotected connections with side members of wood according to EN 1995-1-2**

Wood-to-wood connections	$t_{d,fi}$ (min)	Provisions
Nails	15	$d \geq 2.8$ mm
Screws	15	$d \geq 3.5$ mm
Bolts	15	$t_1 \geq 45$ mm
Dowels	20	$t_1 \geq 45$ mm
Connectors (according to EN 912)	15	$t_1 \geq 45$ mm

t_1 = Thickness of the side member
d = Diameter of the fastener

European standards assume that the so-called unprotected connections fulfil the requirements for resistance classes R15 or R20, *see table 16.3*.

Beyond that value, additional requirements arise that need close scrutiny during the design phase, the main ones obviously being the thickness of the connected wood elements, and the distance of the connectors (nail, dowel) from the edges and ends of the element. This must also ensure the performance also at the required resistance time t_{req}.

A required resistance (time) $t_{req} \leq 30$ min allows some simplifications in the calculations for unprotected connections with nails, dowels and screws with non-projecting heads. It will then be sufficient to increase the thickness of the lateral wood elements, their width and the distance of each connector from the element edges by the amount, *see figure 16.9*:

16.13 $$a_{fi} = \beta_n \times k_{flux} \times \left(t_{req} - t_{d,fi}\right)$$

where the coefficient k_{flux} is introduced, that takes into account an increased thermal flow caused by the metal connector, usually assumed equal to 1.5. Moreover, according to *table 16.2*, β_n = 0.70 mm/min for glulam.

Another method involves the calculation of a resistance time for a shear connection as a function of a reduced load carrying capacity. The connection design resistance $R_{v,k,fi}$ can be derived from the shear strength $R_{v,k}$, obtained according to EN 1995-1-1, by means of the following equation:

16.14 $$R_{v,k,fi} = \eta \times R_{v,k}$$

where $\eta = e^{-k \times t_{d,fi}}$ and k depends on the connection type, according to *table 16.4, page 245*.

Consequently, the fire resistance of the unprotected shear connection can be calculated as:

16.15 $$t_{d,fi} = \frac{1}{k}\, ln\left(\frac{\eta_{fi} \times \gamma_{M,fi}}{\gamma_M \times k_{fi}}\right)$$

where γ_M is the partial safety factor for the connection resistance under normal condition (*see EN 1995-1-1*), $\gamma_{M,fi}$ is the partial safety factor for

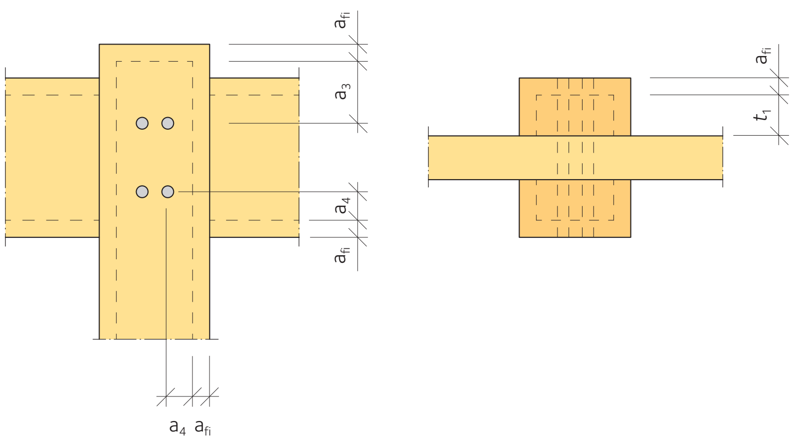


Figure 16.9 **Simplified requirements for connections utilizing dowels, nails or screws with non-projecting heads, for $t_{req} \leq 30$ minutes, according to EN 1995-1-2**

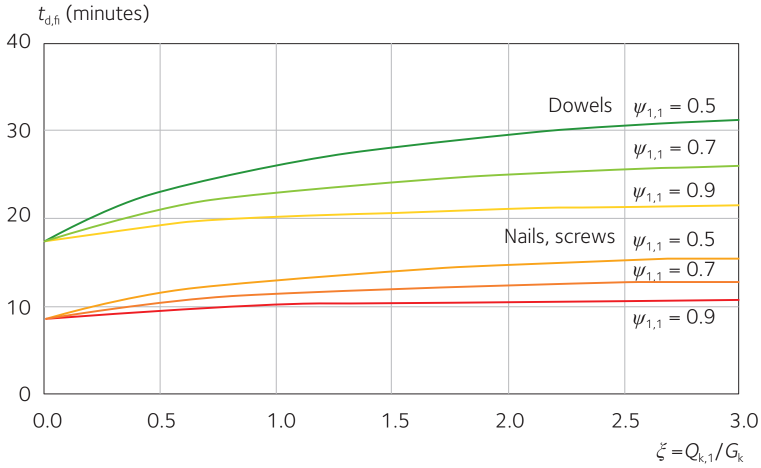


Figure 16.10 **Fire resistance for unprotected shear wood-wood connections as functions of ξ**

the connection resistance in fire conditions and η_{fi} is the previously introduced coefficient for the design load in fire situation, *see equation 16.5, page 240*.

It can be noted that unprotected connections almost always have fire resistances lower than 30 minutes. In *figure 16.10* fire resistances for unprotected wood-wood shear connections are plotted, which have been derived using the *k*-values of *table 16.4* and *equation 16.15, page 244*, and assuming that γ_M =1.3 and k_{fi} = 1.16. As functions of ξ the plots confirm that unprotected connections hardly satisfy the condition for R30.

These synthetic considerations on fire exposed connections obviously lead to the conclusion that to easily reach resistances R45 or R60, the designer must immediately take into account a protection for the connections, or arrange hidden (screened) connections.

Table 16.4 **Values of the parameter *k* to be used in *equation 16.15, page 244*, according to EN 1995-1-2**

Connection with	*k*	Maximum period of validity (min) for parameter *k* in an unprotected connection
Nails and screws	0.080	20
Bolts wood-to-wood ($d \geq 12$ mm)	0.065	30
Bolts steel-to-wood ($d \geq 12$ mm)	0.085	30
Dowels wood-to-wood * ($d \geq 12$ mm)	0.040	40
Dowels steel-to-wood * ($d \geq 12$ mm)	0.085	30
Other connectors (according to EN 912)	0.065	30

* The values for dowels are dependent on the presence of one bolt for every four dowels. When dowels project from wood more than 5 mm, the *k* values for bolts must be used.

Sports arena, Östersund, Sweden.

16.5 Timber structure protection from fire

16.5.1 Single element protection

The most frequently used mass or surface fireproofing treatments to protect wooden structures from fire are:

- Passive fire protection measures – usually compact claddings of non-combustible materials, whose only action is to screen wood from the heat action.
- Treatments directly acting on the material combustion process – their fireproofing or fire retardant action is usually obtained by adding to the material mass or applying to its surface compounds that are able to prevent or delay the ignition of the material.

Treatments of the material mass usually involve the addition to the wooden materials of substances, that are able:

- To inhibit oxidation reactions by neutralizing free radicals, which are apt to react with oxygen (e.g. H-, OH- etc.), or by emission of inert radicals, such as the halogen compounds (chiefly Br based);
- To develop non-combustible gases that dilute those produced by the thermal demolition of wood, thus preventing them from igniting. Examples are ammonium phosphates that, above certain temperatures, are decomposed into evolving gases, such as ammonia, which alter the composition of the mixture air/fuel and thereby prevent flame propagation.

This type of fire retardant treatment can result in a better surface material class, but does not normally change the charring of the wood and therefore does not affect a timber structure's fire resistance.

These latter are essentially made of a binder, the film-forming component, giving a share of the adherence needed to support the compounds. These compounds, under the heat action, are decomposed in a non-combustible residue, water and substances that heat causes to give rise to inert gases, which not only alter the composition of the mixture air/fuel and prevents the flame propagation, but also cause the solid residue to expand. Those substances thus also build a "foam" that solidifies as a light or – depending on the chemical composition –hard layers whose poor conductivity properties make it act as an insulating material. Such a layer can delay the start of the charring and so contribute to greater fire resistance.

Passive protection by means of insulating boards requires not only correct design, but also careful assembly to prevent faulty or loose fixing jeopardizing the effectiveness of the protection.

When utilising wood-based or gypsum boards (either A or H grade according to EN 520), it is sufficient to follow the producer-provided instructions. When dealing with calcium silicate boards (F grade according to EN 520), which can allow substantially increased fire resistance of an element or of a connection, a minimum embedment length $l_a \geq 10$ mm into the residual section is required for fasteners, *see figure 16.11.* In both cases, it is important to take into account the failure times of the boards when subjected to a fire.

It is then a general rule for the single protecting board of thickness h_p, utilised in a multilayer screen, that it should be connected to the wooden element and not to other boards, *see figure 16.11 and 16.12*, according to the minimum requirement described before.

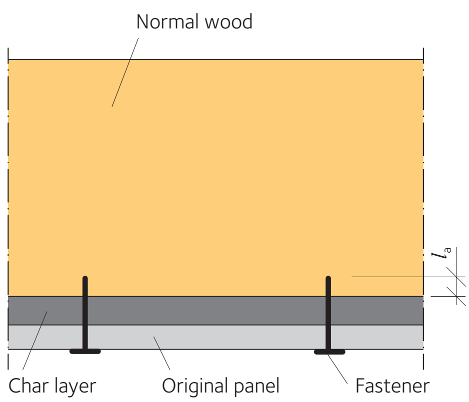


Figure 16.11 Fixing of an insulating gypsum plasterboard to wood, with penetration length l_a of the fasteners into un-burned wood, according to EN 1995-1-2

Connections must be spaced at least 60 mm, the fasteners spaced no more than 200 mm or 17 h_p, whichever is the least; the connection distance from the edges must not be smaller than 15 mm or 1.5 h_p, whichever is the least, and not greater than 3 h_p.

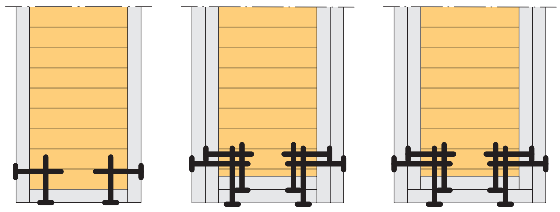


Figure 16.12 Examples of fixing for multilayer screens, according to EN 1995-1-2

16.5.2 Protection of the connections

It has already been pointed out that connections are the weak points of a wooden structure as far as fire resistance is concerned; for them, when more precise calculation or experimental data are not available, fire resistances can be taken as for unprotected connections, i.e. usually lower than 30 minutes, *cf. section 16.4, page 243*. The design of connections as a function of the required resistance and the design of the protection for the connections then become of utmost importance.

When connections are protected instead by means of wood-based panels or gypsum plasterboards (A or H grade, according to EN 520), the delay t_{ch} of the start of charring caused by the protection is given by:

$$t_{ch} \geq t_{req} - 0.5 t_{d,fi} \tag{16.16}$$

where t_{req} is the required connection resistance and $t_{d,fi}$ the non-protected connection resistance.

In the case of protections given by F grade (EN 520) gypsum plasterboards, the resistance required for the protection is significantly lower:

$$t_{ch} \geq t_{req} - 1.2 t_{d,fi} \tag{16.17}$$

The protection must obviously keep its position at least until wood charring starts, that is up to t_{ch}. Some minimal requirements must then be satisfied by protection elements and by the way they are assembled, *see figure 16.13*.

Protections for bolt and nut heads must first have a thickness at least equal to a_{fi}, *see equation 16.13, page 244*. The distance from the edges of the fasteners fixing the panels must be no less than a_{fi}, their spacing must be no greater than 100 mm along the edges and not greater than 300 mm for internal fasteners. The penetration depth of screws or nails cannot be less than 6 times their diameter (A

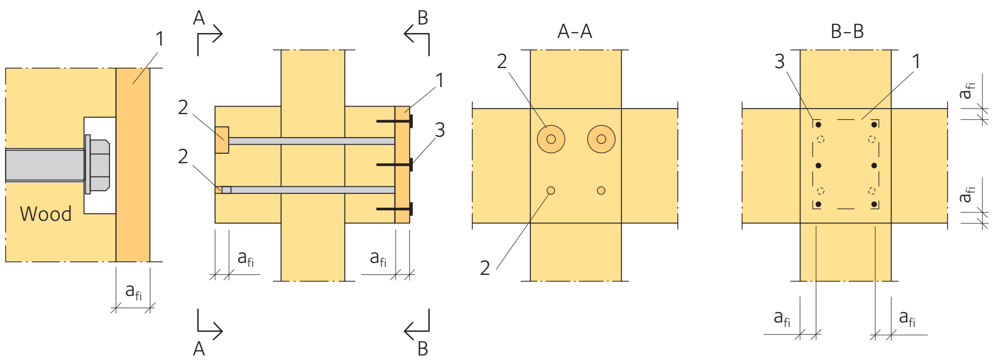


Figure 16.13 Protections for bolts and dowels
1. Detail of protections with board.
2. glued-in plugs.
3. Fasteners for panels (modified from EN 1995-1-2).

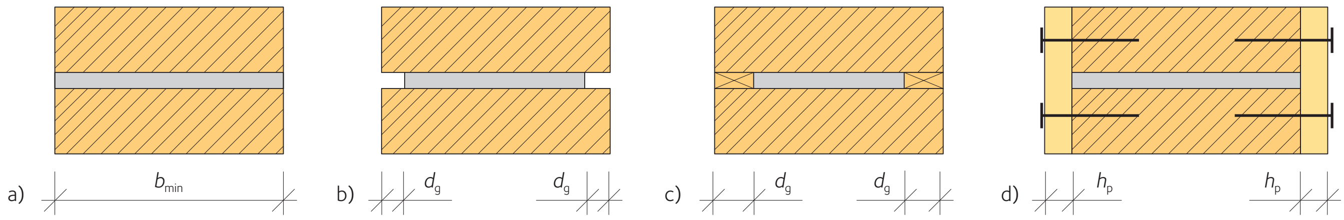


Figure 16.14 **Edge protection for steel plates inside wood**
a) Unprotected plate,
b) plate protected by gaps,
c) by glued-in strips,
d) by panels, according to EN 1995-1-2.

Table 16.5 **Widths of steel plates with unprotected edges**

	Fire resistance class	b_{min} (mm)
Unprotected edges (in general)	R30	200
	R60	280
Unprotected edges (on 1 or 2 sides)	R30	120
	R60	280

and H type panels); for F gypsum plasterboards a penetration depth into un-burned wood of at least 10 mm at time t_{req} is required.

For connections with internal steel plates, at least 2 mm thick, with no parts projecting beyond the wood surface, *see figure 16.14 a)*, *table 16.5* reports the minimum plate widths.

Steel plates narrower than the timber member can be assumed to be protected, thus allowing the minimum size given in *table 16.5*, to be disregarded, when:

- plates with thickness ≤ 3 mm and unprotected edges, where d_g is greater than 20 mm (R30) or 60 mm (R60), *see figure 16.14 b)*.
- plates with edges protected by fillets or boards, with thickness d_g or h_p ≥ 10 mm (R30) or 30 mm (R60), *see figure 16.14 c) and 16.14 d)*.

It is easily shown that protections made of external steel plates (that must be designed according to Eurocode 3, EN 1993-1-2) have very poor fire resistance - usually less than 20 – 30 minutes - since their face in contact with wood can be assumed to be not exposed. It is therefore more advisable to resort to wood or wood-based boards when designing protections. Steel plates can then be considered, so long as they are completely covered (both faces and edges) by elements whose minimum sizes a_{fi} are given by:

16.18
$$a_{\text{fi}} = \beta_{\text{n}} \times k_{\text{flux}} \times \left(t_{\text{req}} - 5\right)$$

This equation is analogous to *equation 16.13, page 244*, when the steel plate is given a resistance of 5 minutes.

The resistance of axially loaded screws protected from fire by wood, *figure 16.15*, taken from EN 1995-1-2, leads to interesting considerations on wood provided protection.

When (with a_1, a_2, a_3 in mm and β conventionally assumed to be equal to 1 mm/minute), η (parameter value in *equation 16.12*) can be assumed equal to 1, the protection provided to the connection by wood is regarded as sufficient.

For protection of mechanical connections in a wooden structure, it is therefore essential that significant fire resistance performances of timber elements are attained. These protections can be obtained by means of screens, highly effective either when the connection cannot be put inside the wooden element, *see figure 16.16, page 249*, or to protect exposed T-shaped steel profile connections. *Figures 16.16* and *16.17 b), page 249*, refer to R60 requirements.

As far as connections for joining main and secondary beams are concerned, *see figure 16.17, page 249*. There are different types of T-shaped metallic profiles on the building accessories market, which are to be inserted inside a wooden element with a better aesthetic effect, *see figure 16.17 c), page 249*, and *Chapter 14, page 198*.

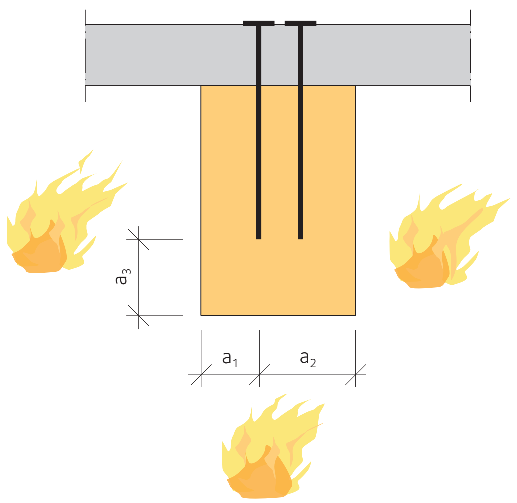
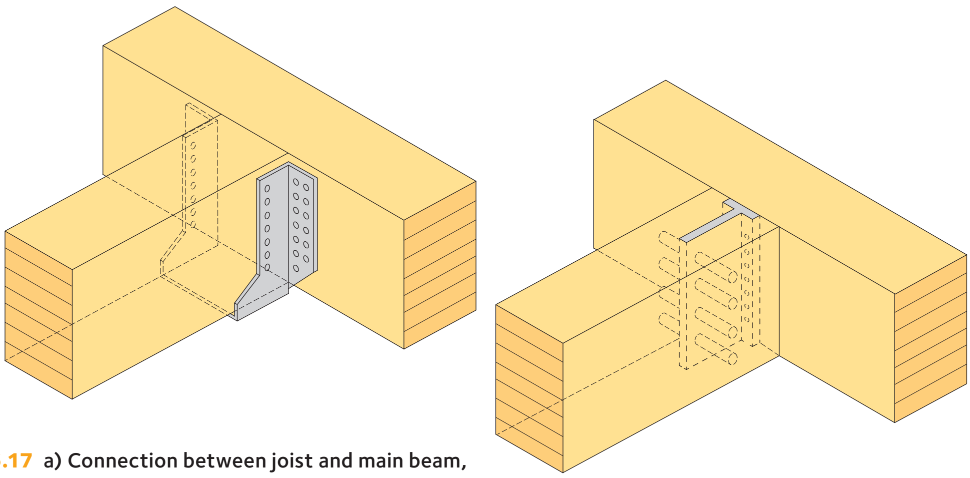


Figure 16.15 **Connection with axially loaded screws, according to EN 1995-1-2**

Figure 16.16 Protection of end-pin-support by means of asbestos–free calcium silicate plates

a)

b)



c)

Figure 16.17 a) Connection between joist and main beam,
b) protected by means of calcium silicate plates,
c) possible alternative to an exposed connection between joist and main beam

16.6 Design considerations

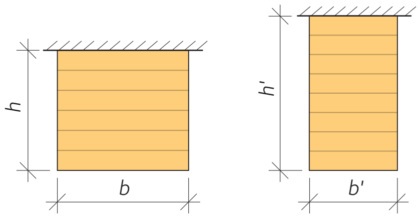


Figure 16.18 "Wide" and "thin" glulam cross-sections

Some considerations can be reported about design and verification of glued laminated timber beams and columns.

First, care must be taken of the basis/height ratio for the cross-section of a fire exposed beam. Let us consider two glulam beam sections exposed to fire on three sides, with different basis/height ratios and with the same section modulus, thus with the same resisting bending moment in normal conditions, *see figure 16.18*. We will hereafter call these two sections "wide" and "thin".

The curves in *figure 16.19* show different behaviour under fire exposure of "wide" sections (with a high basis/height ratio) and "thin" ones (small basis/ height ratio typical for glued laminated wood elements).

These curves show less decreasing slopes for high basis/height ratios, therefore the decay of the beam with the "wide" cross-section is somewhat slower than that observed for "thin" sections, *see figure 16.19, left*. The reported example refers to beams 220 × 240 mm ("wide" section) and 160 × 280 mm ("thin" section), and their behaviour has been calculated according to the effective cross-section method (with a correction of about 1 percent to give both sections the same initial resisting bending moment).

On the other hand, let us assume that a given load-bearing capacity is required at time t of fire exposure. If we consider two sections with the same load-bearing capacity after a given time of exposure to fire (e. g. t = 60 minutes), using a symmetric line of thinking the slim sections at each time before t exhibit a greater value of resisting bending moment. Henceforth designing a slim residual section for time t leads to a better static performance up to time t. The curves shown in *figure 16.19* (right) refer to a residual resistance at t = 60 minutes for softwood glulam of starting cross-sections 200 × 200 mm ("wide" section) and 140 × 280 mm ("thin" section). In this case, similarly, a slight correction (about 1 percent) was made to give both sections the same final resisting bending moment.

It is also of fundamental importance to consider the behaviour of axially compressed glulam columns, for which buckling stability checks during a fire becomes important, since they are usually exposed on all four sides.

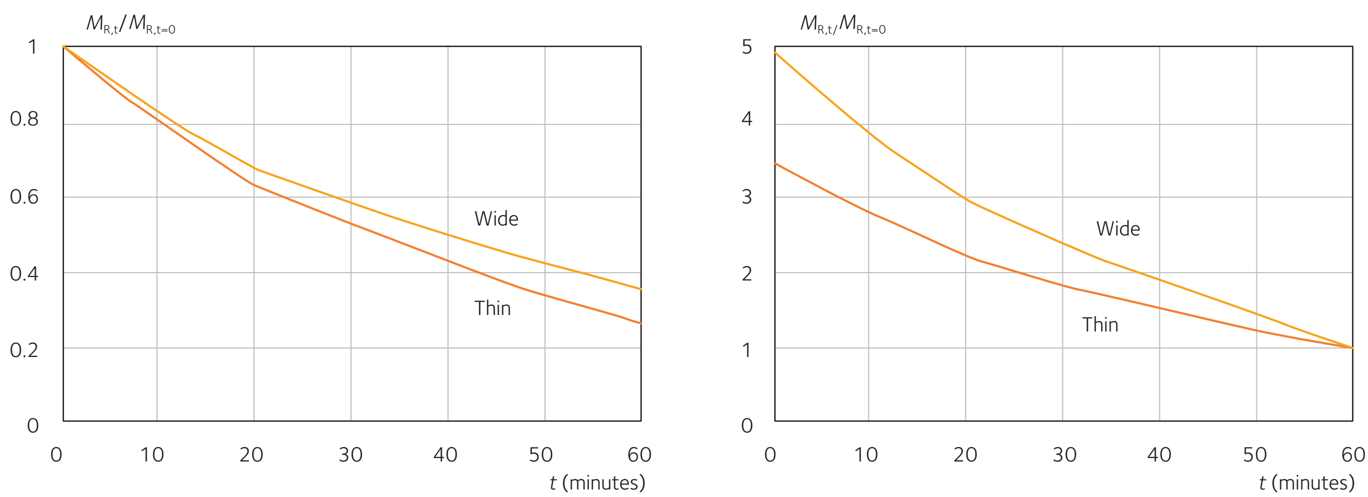


Figure 16.19 Decrease of resistant bending moment for "wide" and "thin" glulam sections

Firstly, when the glulam column is continuous over different storeys (different fire compartments) and it is part of a non-sway frame, more favourable boundary conditions than for normal temperature design may be assumed. In intermediate storeys the column may be assumed as fixed at both ends, whilst in the top storey the column may be assumed as fixed at its lower end, *see figure 16.20*, with the column length L taken as shown in the same figure.

With regard to the problem of column stability, the control under fire conditions becomes crucial even if the slenderness ratios are small and the requested fire resistance is moderate.

Some examples are reported in *figure 16.21*, regarding glulam GL24h columns with square cross-sections of 160, 200 and 240 mm side, and height l_0 = 3,000 mm. The corresponding slenderness ratios λ_y are thus approximately equal to 65, 52 and 43. The reduction factor η_{fi}, *see equation 16.5, page 240*, for the design load in the fire situation was assumed equal to 0.5, 0.6 and 0.7. It must be noted that η_{fi} = 0.6 is the recommended value, according to EN 1995-1-2, except for imposed loads corresponding to category E (areas susceptible to accumulation of goods), where the recommended value is η_{fi} = 0.7.

The abscissa of each plot point with the ordinate 1.0 defines therefore, for any combination of slenderness ratio λ_y and reduction factor η_{fi}, the resistance value in minutes.

Figure 16.21 shows that only columns with slenderness ratios λ_y less than 50 and $\eta_{fi} \leq 0.6$ can ensure a R30, while the same columns can reach R60 only if the initial stress level (normal condition) is suitably reduced (see the values in ordinate). For instance, a column with a starting slenderness ratio $\lambda y \cong 43$ and a reduction factor η_{fi} = 0.7, should be loaded with a 60 percent reduction of its initial carrying capacity to reach R60.

Glued laminated wood columns therefore call for an accurate design starting from R30, since the decrease of the resistant cross-section is coupled with a rapid increase in slenderness. The reduction of the carrying capacity of a glued laminated timber column exposed to fire is therefore important, and the designer must consider this with an appropriate increase of the column cross-section.

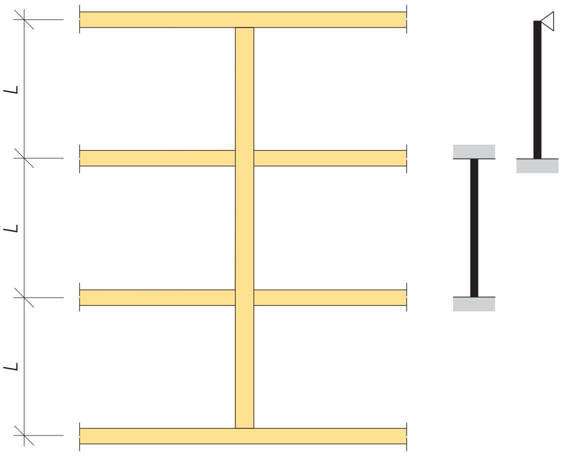


Figure 16.20 Buckling length under fire condition of a continuous column, according to EN 1995-1-2

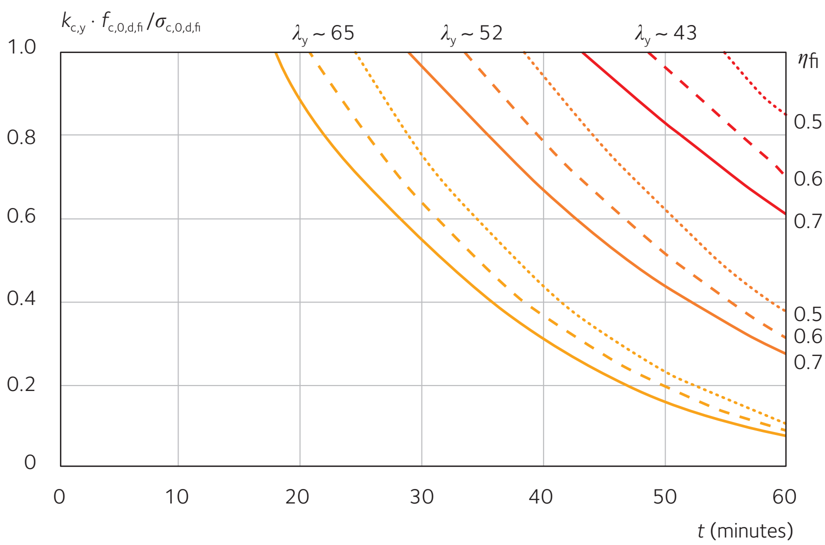


Figure 16.21 Fire resistance of glulam columns with different slenderness ratios

16.7 Fire resistance of fixings and connections – some general solutions

While glulam elements have excellent fire safety properties, steel joints and connections can be weak points that usually need to be fireproofed in order for the structure as a whole to comply with the intended fire resistance class.

The fire safety part of Eurocode 5 (EN 1995-1-2) contains rules for calculating the load-bearing capacity of both protected and unprotected nail and screw joints during a fire, but tested methods are currently lacking. The solutions below are mainly based on German experience (Kordina et al.). The fire resistance of each listed solution must be verified on a case-by-case basis in terms of dimensions and measurements.

16.7.1 Connections

The characteristic load-bearing capacity of steel connectors (nails, wood screws, rods and dowels) decreases as the temperature increases, in the same way as for steel structures, and force transfer between connectors and charred wood thus cannot be relied on. In practice, this means that force transferring connections must as a rule be fireproofed, e.g. as set out in *table 16.6*, in order to achieve a higher fire resistance class than R30.

Symmetrical, shear loaded wood-to-wood or wood-to-steel joints can be assumed to meet the requirements for R15 without special measures according to EN 1995-1-2. Intermediate steel members, such as slotted-in plates, are assumed to be at least 2 mm thick and side members at least 6 mm thick.

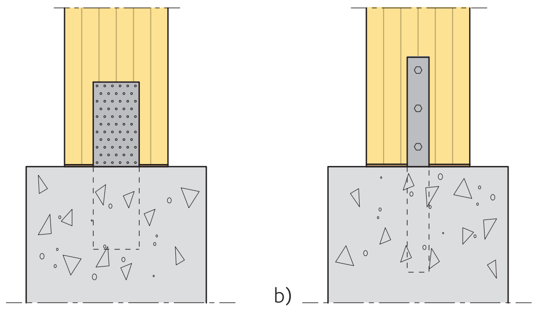


Figure 16.22 **Pinned column base**
a) Cast-in nail plate: R60.
b) Cast-in flat bar and screws: R60.

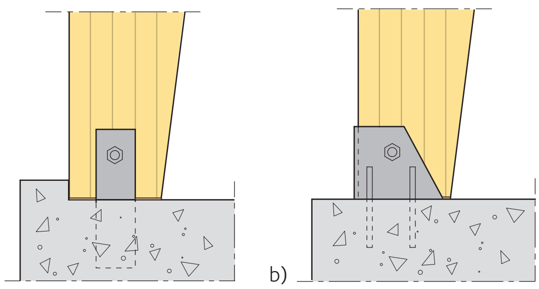


Figure 16.23 **Pinned frame base**
a) With concrete abutment: R60.
b) Steel shoe: R30.

Table 16.6 **Examples of fireproofing insulation for different connectors**
Minimum insulation thickness in mm. [1)]

Material	Minimum density, average value (kg/m³)	Nail, wood screw, screwed joints		Dowel joints	
		R30	R60	R30	R60
Particleboards [2)]	600	19	[3)]	14	[3)]
Plywood, LVL	450	23	[3)]	16	[3)]
Construction timber, glulam	350	16	44	10	40
Normal plasterboard (type A)	–	12.5	3 × 12.5 [4)]	9	2 × 12.5 [4)]
Fire-resistant plasterboard (type F)	–	12.5	2 × 12.5 [4)]	12.5 5)	2 × 12.5 [4)]
Gypsum fibreboard	–	12.5	3 × 12.5 [4)]	9	2 × 12.5 [4)]
Mineral wool board	30	60	140	45	125
Mineral wool board	120	40	80	35	75

[1)] Standard thicknesses may be larger.
[2)] Chipboard, fibreboard, OSB.
[3)] Multiple layers can be calculated according to EN 1995-1-2.
[4)] Nailing or screwing of each board layer.
[5)] Minimum thickness available in Sweden.

16.7.2 Column base

Pinned column base

Pinned attachment of the column base, where the force is transferred mainly via contact pressure, e.g. as shown in *figure 16.22 a)* or *b), page 252*, is deemed to fulfil the requirements for class R60 without special fireproofing. It should also be possible to absorb horizontal forces by abutment against a concrete block, as shown in *figure 16.23 a), page 252,* to achieve R60 without special measures. A column base with a steel shoe as shown in *figure 16.23 b), page 252,* a common solution for three-pin frames, is deemed to meet R30 without fireproofing, but the steel plate must be dimensioned for the fire case in question.

Fixed column base

Slotted-in steel plates and dowels as shown in *figure 16.24 a)* are considered to meet the requirements for R30 and R60, depending on the dimensions and fireproofing. Slotted-in steel plates with bonded-in rods as shown in *figure 16.24 b)* are considered to meet the requirements for R30 and R60, depending on the dimensions. The edge spacing for the rods is chosen so that they are within the effective residual cross-section. When using a steel base, this must be fire protected.

Steel column supports as shown in *figure 16.24 c)* must be fireproofed (see, for example, *table 16.6*) if subject to fire resistance class requirements.

When fixing with steel cover plates, these must be fireproofed (see, for example, *table 16.6*) if subject to fire resistance class requirements. Edge spacing for nails or screws with any washers should be chosen so that they are within the residual cross-section.

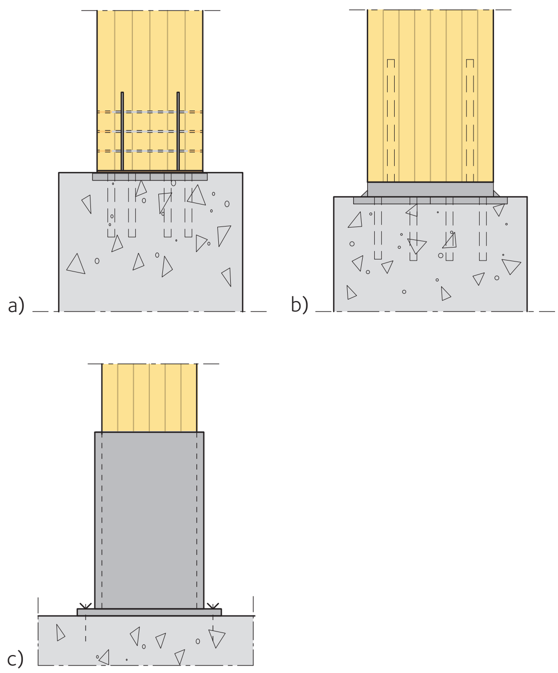


Figure 16.24 Fixed column base
a) Slotted-in steel plates and dowels: R30 or R60 depending on dimensions.
b) Bonded-in rod: R30 or R60 depending on dimensions.
c) Steel column support: Requires fireproofing.

16.7.3 Column top

Slotted-in steel plates and dowels as shown in *figure 16.25 a)* are considered to meet the requirements for R30 and R60, depending on the dimensions and fireproofing. Glulam forks as shown in *figure 16.25 b)* are considered to meet the requirements for R30 if the fork is over 25 mm thick and R60 if the thickness is over 40 mm. For beams with a height greater than 4 times the width, the requirement for the thickness of the fork is increased to 80 and 140 mm, respectively, due to the risk of buckling. The design is not suitable if horizontal forces are to be transferred in a fire.

Fixing with steel flat bars as shown in *figure 16.25 c)* is considered to fulfil the requirements for R30 if only downward vertical loads are to be transferred and the lateral stability of the beam in the event of a fire is ensured by special measures, for example using the connecting roof or wall structure. R60 requires fireproofing insulation as set out in *table 16.6, page 252*.

A corresponding connection with nail plates requires fireproofing insulation even in class R30.

Joints with bonded-in rods as shown in *figure 16.25 d)* are considered to meet the requirements for R30 and R60, depending on the dimensions. The edge spacing of the rods is chosen so that they fall within the residual cross-section. The lateral stability of the beam in the event of a fire must be ensured by special measures, for example using the connecting roof or wall structure.

A connection with a slotted-in T-profile as shown in *figure 16.26* can be classified as R30 if the beam width is at least 120 mm, and as R60 if it is at least 230 mm. In both cases, the width of the slot must not

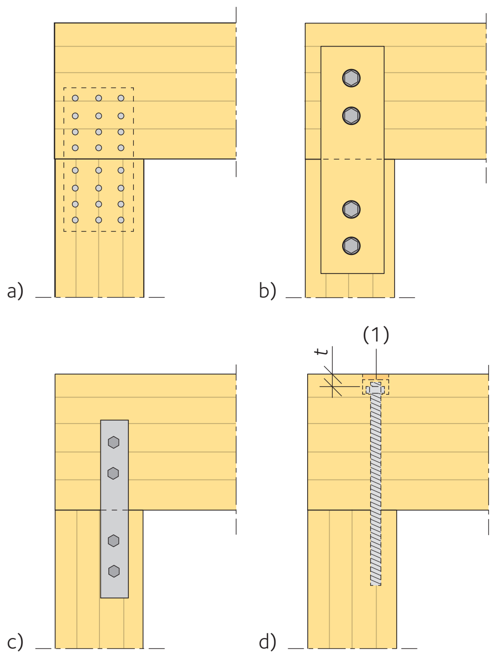


Figure 16.25 Column-beam connection
a) Slotted-in steel plates and dowels: R30 or R60 depending on dimensions.
b) Glulam fork: R30 or R60 depending on dimensions.
c) Steel flat bars: R30.
d) Bonded-in rod: R30 or R60 depending on dimensions.

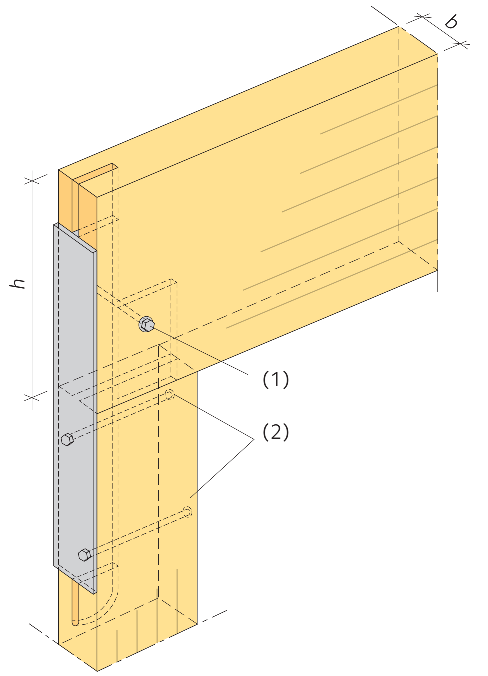


Figure 16.26 Column-beam connection with slotted-in T-profile: R30 or R60 depending on dimensions
1) Through bolt.
2) Through bolt with washer.

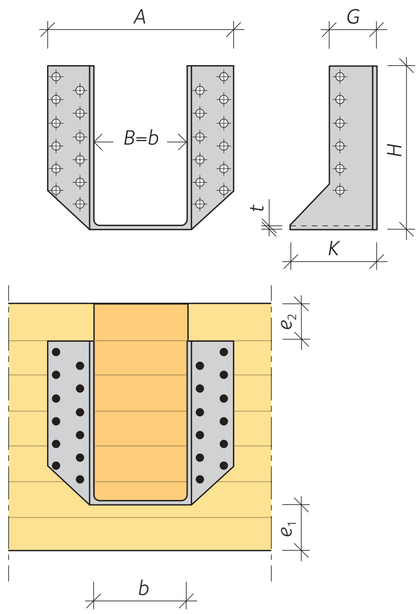


Figure 16.27 Connection of secondary beam with standard beam hanger: R30

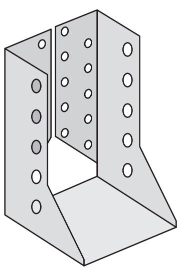


Figure 16.28 Hanger with internal flanges

Table 16.7 Minimum measurements for class R30 when connecting to a secondary beam using a standard hanger. Designations according to *figures 16.27* and *16.29*.

(mm)	Capacity utilisation [1] 33 percent	Capacity utilisation [1] 75 percent
B	100	120
A	170	200
G	40	44
K	75	85
t	2	2
e_1	50	100
e_2	20	30
Nail length	75	75
No. of nails		
in primary beam	2 × 6	2 × 7
in secondary beam	2 × 12	2 × 13

[1] The load effect in the event of a fire as percentage of the design load capacity when designing without taking fire into account.

Source: Kordina et al.

exceed 10 mm. For slender beams ($h/b \geq 4$) in class R60, the back of the T-profile requires fireproofing insulation, for example as per *table 16.6*, to avoid stability problems caused by charring in the slot.

16.7.4 Connecting a secondary beam

Beam hangers with no fireproofing insulation, *see figure 16.27*, may be classified as R30 if the capacity utilisation is no more than 75 percent. However, the minimum dimensions according to *table 16.7* must be met and extra long nails are required: 75 mm instead of the usual 40 mm.

The same fire resistance class is assigned to beam hangers with internal flanges as shown in *figure 16.28* if the minimum dimensions in *table 16.7* are met.

When connecting to columns as shown in *figure 16.29*, edge spacing e_1 must be maintained as prescribed in *table 16.7* in order to achieve fire resistance class R30.

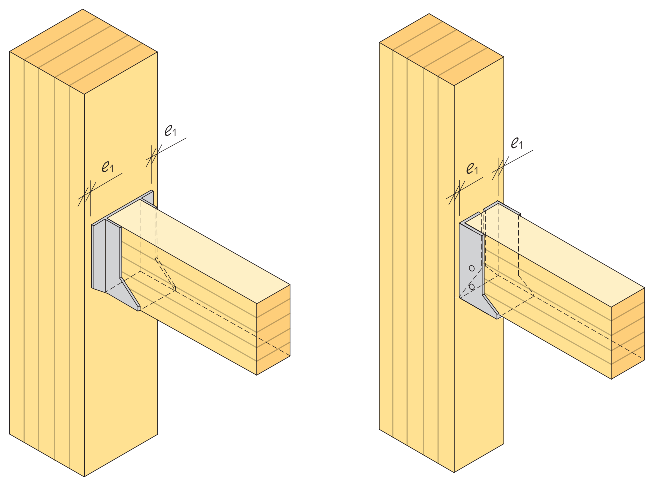


Figure 16.29 Edge spacing for beam-column connection with standard beam hanger: R30 under certain conditions, *see table 16.7*

16.7.5 Beam joint

Bolted beam joints as shown in *figure 16.30* fulfil the requirements for R30 without fireproofing insulation if the load effect in a fire is no more than 65 percent of the design load capacity when designing without taking fire into account.

With fireproofing insulation as per *table 16.6*, the requirements for R30 are mwwss. With insulation as per *table 16.6* the requirements for R30 are met if the capacity utilisation in a fire is no more than 65 percent.

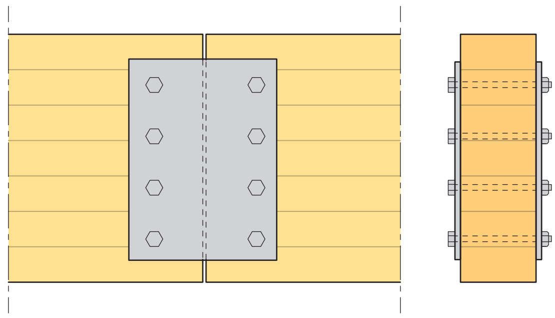


Figure 16.30 Bolted beam joint: R30

16.7.6 Ridge joint

Ridge joints that use nail plates as per *figure 16.32 a)* require fireproofing insulation as set out in *table 16.6* to achieve fire resistance class R30 or R60.

In the event of higher appearance requirements, the ridge fixings in *figure 16.32 b)* may be a suitable alternative. Without fireproofing insulation, the design is deemed to meet the requirements for class R30. If the space between the beams is filled with mineral wool ($\rho \geq 50$ kg/m^3) and the fixing is placed within the effective residual cross-section, the fire resistance is considered to correspond to class R60, depending on the dimensions.

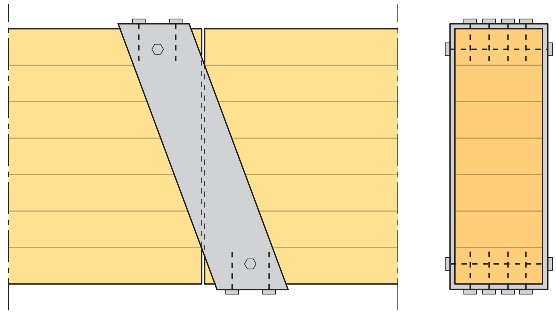


Figure 16.31 Gerber joint: Requires fireproofing

16.7.7 Steel tie

An uninsulated steel tension tie will generally not meet the requirements for R30. Mineral wool pipe wrap is the simplest way of providing fireproofing insulation. If higher appearance standards are required, these can be encased.

Note that the elongation of the tie due to temperature increase is significant even in a design with fireproofing insulation and the supports must be designed accordingly.

Tie fixings as per *figure 16.33* must be furnished with fireproofing insulation in order to achieve a fire resistance class. The material and insulation thickness can be selected in line with *table 16.6*.

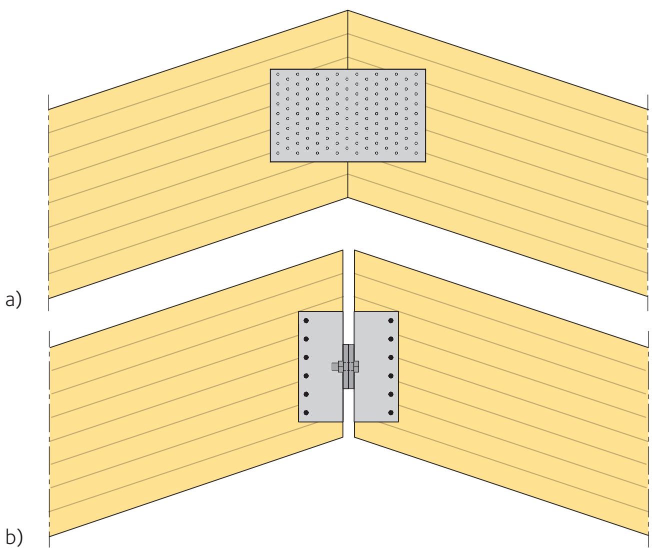


Figure 16.32 Ridge joints
a) Nail plates: Require fireproofing.
b) Welded steel fixing: R30.

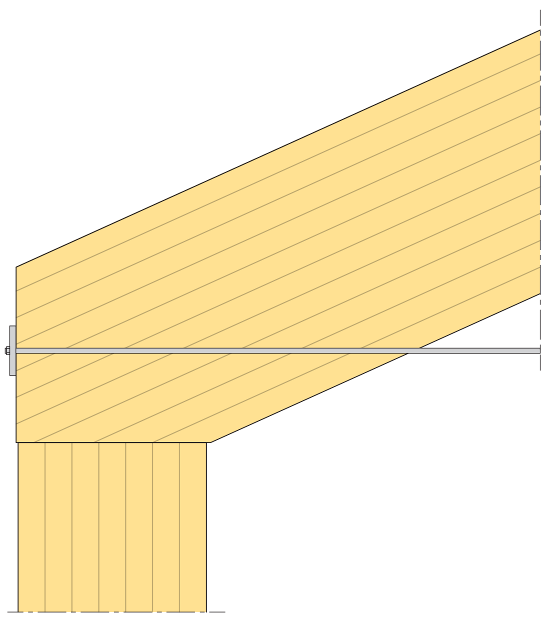


Figure 16.33 Steel tension tie fixing: Requires fireproofing

Symbols

Symbols in EN 1995-1-1.

Symbol	Explanation
Latin upper case letters	
A	Cross-sectional area
A_{ef}	Effective area of the total contact surface between a punched metal plate fastener and the timber; effective total contact surface perpendicular to the grain
A_f	Cross-sectional area of flange
$A_{net,t}$	Net cross-sectional area perpendicular to the grain
$A_{net,v}$	Net shear area parallel to the grain
C	Spring stiffness
$E_{0,05}$	Fifth percentile value of modulus of elasticity
E_d	Design value of modulus of elasticity
E_{mean}	Mean value of modulus of elasticity
$E_{mean,fin}$	Final mean value of modulus of elasticity
F	Force
$F_{A,Ed}$	Design force acting on a punched metal plate fastener at the centroid of the effective area
$F_{A,min,d}$	Minimum design force acting on a punched metal plate fastener at the centroid of the effective area
$F_{ax,Ed}$	Design axial force on fastener
$F_{ax,Rd}$	Design value of axial withdrawal capacity of the fastener
$F_{ax,Rk}$	Characteristic axial withdrawal capacity of the fastener
F_c	Compressive force
F_d	Design force
$F_{d,ser}$	Design force at the serviceability limit state
$F_{f,Rd}$	Design load-carrying capacity per fastener in wall diaphragm
$F_{i,c,Ed}$	Design compressive reaction force at end of shear wall
$F_{i,t,Ed}$	Design tensile reaction force at end of shear wall
$F_{i,vert,Ed}$	Vertical load on wall
$F_{i,v,Rd}$	Design racking resistance of panel *i* or wall *i*
F_{la}	Lateral load
$F_{M,Ed}$	Design force from a design moment
F_t	Tensile force
$F_{t,Rk}$	Characteristic tensile capacity of a connection
$F_{v,0,Rk}$	Characteristic load-carrying capacity of a connector along the grain
$F_{v,Ed}$	Design shear force per shear plane of fastener; Horizontal design effect on wall diaphragm
$F_{v,Rd}$	Design load-carrying capacity per shear plane per fastener; Design racking load capacity
$F_{v,Rk}$	Characteristic load-carrying capacity per shear plane per fastener
$F_{v,w,Ed}$	Design shear force acting on web
$F_{x,Ed}$	Design value of a force in *x*-direction
$F_{y,Ed}$	Design value of a force in *y*-direction
$F_{x,Rd}$	Design value of plate capacity in *x*-direction
$F_{y,Rd}$	Design value of plate capacity in *y*-direction
$F_{x,Rk}$	Characteristic plate capacity in *x*-direction
$F_{y,Rk}$	Characteristic plate capacity in *y*-direction
$G_{0,05}$	Fifth percentile value of shear modulus
G_d	Design value of shear modulus
G_{mean}	Mean value of shear modulus
H	Overall rise of a truss
I_f	Second moment of area of flange
I_{tor}	Torsional moment of inertia
I_z	Second moment of area about the weak axis
K_{ser}	Slip modulus
$K_{ser,fin}$	Final slip modulus
K_u	Instantaneous slip modulus for ultimate limit states
$L_{net,t}$	Net width of the cross-section perpendicular to the grain
$L_{net,v}$	Net length of the fracture area in shear
$M_{A,Ed}$	Design moment acting on a punched metal plate fastener
$M_{ap,d}$	Design moment at apex zone
M_d	Design moment
$M_{y,Rk}$	Characteristic yield moment of fastener
N	Axial force
$R_{90,d}$	Design splitting capacity
$R_{90,k}$	Characteristic splitting capacity
$R_{ax,d}$	Design load-carrying capacity of an axially loaded connection
$R_{ax,k}$	Characteristic load-carrying capacity of an axially loaded connection
$R_{ax,\alpha,k}$	Characteristic load-carrying capacity at an angle α to grain
R_d	Design value of a load-carrying capacity
$R_{ef,k}$	Effective characteristic load-carrying capacity of a connection
$R_{iv,d}$	Design racking capacity of a wall
R_k	Characteristic load-carrying capacity
$R_{sp,k}$	Characteristic splitting capacity

$R_{to,k}$	Characteristic load-carrying capacity of a toothed plate connector
$R_{v,d}$	Design racking capacity of a wall diaphragm
V	Shear force; volume
V_u, V_l	Shear forces in upper and lower part of beam with a hole
W_y	Section modulus about axis y
X_d	Design value of a strength property
X_k	Characteristic value of a strength property

Latin lower case letters

a	Distance
a_1	Spacing, parallel to grain, of fasteners within one row
$a_{1,CG}$	Minimum end distance to the centre of gravity of the screw in each timber member
a_2	Spacing, perpendicular to grain, between rows of fasteners
$a_{2,CG}$	Minimum edge distance to the centre of gravity of the screw in each timber member
$a_{3,c}$	Distance between fastener and unloaded end
$a_{3,t}$	Distance between fastener and loaded end
$a_{4,c}$	Distance between fastener and unloaded edge
$a_{4,t}$	Distance between fastener and loaded edge
a_{bow}	Maximum bow of truss member
$a_{bow,perm}$	Maximum permitted bow of truss member
a_{dev}	Maximum deviation of truss
$a_{dev,perm}$	Maximum permitted deviation of truss
b	Width
b_i	Width of panel i or wall i
b_{net}	Clear distance between studs
b_w	Web width
d	Diameter; outer thread diameter
d_1	Diameter of centre hole of connector; inner thread diameter
d_c	Connector diameter
d_{ef}	Effective diameter
d_h	Head diameter of connector
$f_{h,i,k}$	Characteristic embedment strength of timber member i
$f_{a,0,0}$	Characteristic anchorage capacity per unit area for α = 0° and β = 0°
$f_{a,90,90}$	Characteristic anchorage capacity per unit area for α = 90° and β = 90°
$f_{a,\alpha,\beta,k}$	Characteristic anchorage strength
$f_{ax,k}$	Characteristic withdrawal parameter for nails
$f_{c,0,d}$	Design compressive strength along the grain
$f_{c,w,d}$	Design compressive strength of web
$f_{f,c,d}$	Design compressive strength of flange
$f_{c,90,k}$	Characteristic compressive strength perpendicular to grain
$f_{f,t,d}$	Design tensile strength of flange
$f_{h,k}$	Characteristic embedment strength
$f_{head,k}$	Characteristic pull-through parameter for nails
f_1	Fundamental frequency
$f_{m,k}$	Characteristic bending strength
$f_{m,y,d}$	Design bending strength about the principal y-axis
$f_{m,z,d}$	Design bending strength about the principal z-axis
$f_{m,\alpha,d}$	Design bending strength at an angle α to the grain
$f_{t,0,d}$	Design tensile strength along the grain
$f_{t,0,k}$	Characteristic tensile strength along the grain
$f_{t,90,d}$	Design tensile strength perpendicular to the grain
$f_{t,w,d}$	Design tensile strength of the web
$f_{u,k}$	Characteristic tensile strength of bolts
$f_{v,0,d}$	Design panel shear strength
$f_{v,ax,\alpha,k}$	Characteristic withdrawal strength at an angle α to grain
$f_{v,ax,90,k}$	Characteristic withdrawal strength perpendicular to grain
$f_{v,d}$	Design shear strength
h	Depth; height of wall
h_{ap}	Depth of the apex zone
h_d	Hole depth
h_e	Embedment depth; loaded edge distance
h_{ef}	Effective depth
$h_{f,c}$	Depth of compression flange
$h_{f,t}$	Depth of tension flange
$h_{r\ell}$	Distance from lower edge of hole to bottom of member
h_{ru}	Distance from upper edge of hole to top of member
h_w	Web depth
i	Notch inclination
$k_{c,y}$, $k_{c,z}$	Instability factor
k_{cr}	Crack factor for shear resistance
k_{crit}	Factor used for lateral buckling
k_d	Dimension factor for panel
k_{def}	Deformation factor
k_{dis}	Factor taking into account the distribution of stresses in an apex zone
$k_{f,1}$, $k_{f,2}$, $k_{f,3}$	Modification factors for bracing resistance
k_h	Depth factor
$k_{i,q}$	Uniformly distributed load factor
k_m	Factor considering re-distribution of bending stresses in a cross-section
k_{mod}	Modification factor for duration of load and moisture content
k_n	Sheathing material factor

k_r	Reduction factor
$k_{R,red}$	Reduction factor for load-carrying capacity
k_s	Fastener spacing factor; modification factor for spring stiffness
$k_{s,red}$	Reduction factor for spacing
k_{shape}	Factor depending on the shape of the cross-section
k_{sys}	System strength factor
k_v	Reduction factor for notched beams
k_{vol}	Volume factor
k_y or k_z	Instability factor
$l_{a,min}$	Minimum anchorage length for a glued-in rod
l	Span; contact length
l_A	Distance from a hole to the centre of the member support
l_{ef}	Effective length; Effective length of distribution
l_V	Distance from a hole to the end of the member
l_Z	Spacing between holes
m	Mass per unit area
n_{40}	Number of frequencies below 40 Hz
n_{ef}	Effective number of fasteners
p_d	Distributed load
q_i	Equivalent uniformly distributed load
r	Radius of curvature
s	Spacing
s_0	Basic fastener spacing
r_{in}	Inner radius
t	Thickness
t_{pen}	Penetration depth
u_{creep}	Creep deformation
u_{fin}	Final deformation
$u_{fin,G}$	Final deformation for a permanent action G
$u_{fin,Q,1}$	Final deformation for the leading variable action Q_1
$u_{fin,Q,i}$	Final deformation for accompanying variable actions Q_i
u_{inst}	Instantaneous deformation
$u_{inst,G}$	Instantaneous deformation for a permanent action G
$u_{inst,Q,1}$	Instantaneous deformation for the leading variable action Q_1
$u_{inst,Q,i}$	Instantaneous deformation for accompanying variable actions Q_i
w_c	Precamber
w_{creep}	Creep deflection
w_{fin}	Final deflection
w_{inst}	Instantaneous deflection
$w_{net,fin}$	Net final deflection
v	Unit impulse velocity response

Greek lower case letters	
α	Angle between the x-direction and the force for a punched metal plate; Angle between the direction of the load and the loaded edge (or end)
β	Angle between the grain direction and the force for a punched metal plate
β_c	Straightness factor
γ	Angle between the x-direction and the timber connection line for a punched metal plate
γ_M	Partial factor for material properties, also accounting for model uncertainties and dimensional variations
λ_y	Slenderness ratio corresponding to bending about the y-axis
λ_z	Slenderness ratio corresponding to bending about the z-axis
$\lambda_{rel,y}$	Relative slenderness ratio corresponding to bending about the y-axis
$\lambda_{rel,z}$	Relative slenderness ratio corresponding to bending about the z-axis
ρ_k	Characteristic density
ρ_m	Mean density
$\sigma_{c,0,d}$	Design compressive stress along the grain
$\sigma_{c,\alpha,d}$	Design compressive stress at an angle α to the grain
$\sigma_{f,c,d}$	Mean design compressive stress of flange
$\sigma_{f,c,max,d}$	Design compressive stress of extreme fibres of flange
$\sigma_{f,t,d}$	Mean design tensile stress of flange
$\sigma_{f,t,max,d}$	Design tensile stress of extreme fibres of flange
$\sigma_{m,crit}$	Critical bending stress
$\sigma_{m,y,d}$	Design bending stress about the principal y-axis
$\sigma_{m,z,d}$	Design bending stress about the principal z-axis
$\sigma_{m,\alpha,d}$	Design bending stress at an angle α to the grain
σ_N	Axial stress
$\sigma_{t,0,d}$	Design tensile stress along the grain
$\sigma_{t,90,d}$	Design tensile stress perpendicular to the grain
$\sigma_{w,c,d}$	Design compressive stress of web
$\sigma_{w,t,d}$	Design tensile stress of web
τ_d	Design shear stress
$\tau_{F,d}$	Design anchorage stress from axial force
$\tau_{M,d}$	Design anchorage stress from moment
$\tau_{tor,d}$	Design shear stress from torsion
ψ_0	Factor for combination value of a variable action
ψ_1	Factor for frequent value of a variable action
ψ_2	Factor for quasi-permanent value of a variable action
ζ	Modal damping ratio

Source: EN 1995-1-1:2004, 1.6

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Glulam of Sweden AB, pages 81, 101, 103, 107, 160, 185
Thomas Harrysson, pages 112, 150, 176, 239
Sam Hedman, page 59
Sören Håkanlind, pages 7, 8, 9 (lower), 21, 22, 24, 29, 30, 32, 33, 35, 37, 46, 50, 64, 71, 74, 76, 85, 86, 87, 90, 95—98, 105, 106, 108, 114, 118, 124, 136, 142, 144, 147, 156, 157, 163-165, 174, 175, 188, 201, 206, 208, 218, 223, 225
Ola Högberg, page 100
Kerstin Jonsson, pages 11, 100
Joakim Kröger, page 65
Åke E:son Lindman, pages 1, 4, 204
Christian Ljung, pages 68, 69, 99, 205
Martinsons Byggsystem AB, pages 16, 135, 227
Moelven Töreboda AB, pages 9 (top), 20, 88, 89, 158, 195
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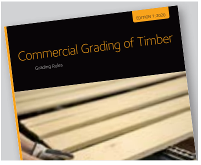

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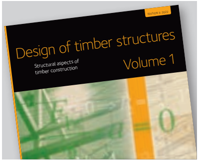

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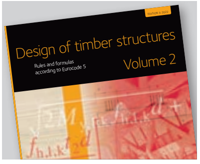

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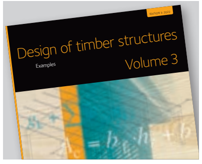

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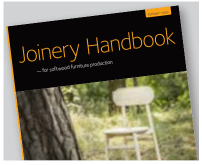

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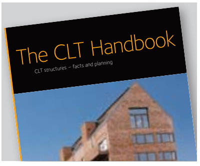

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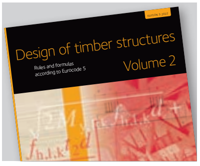

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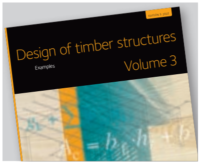

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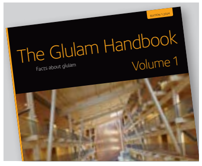

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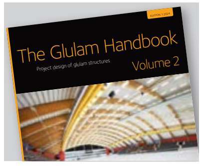

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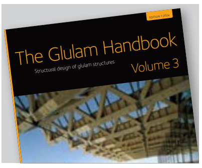

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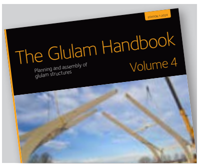

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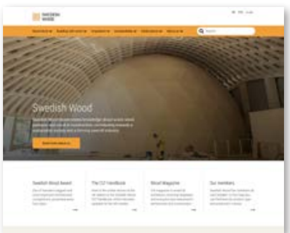

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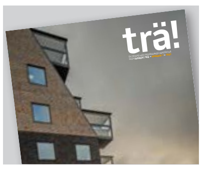

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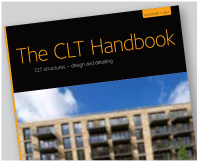

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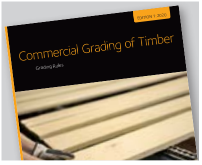

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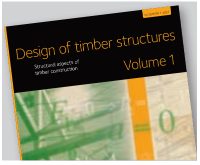

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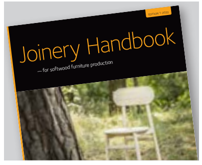

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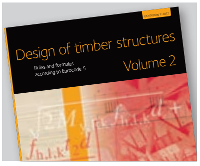

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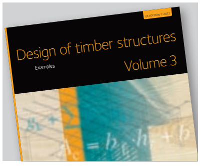

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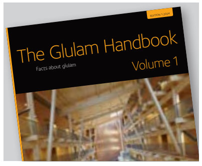

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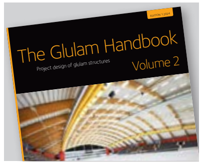

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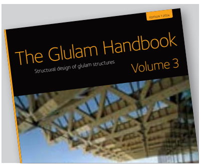

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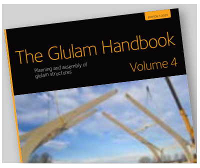

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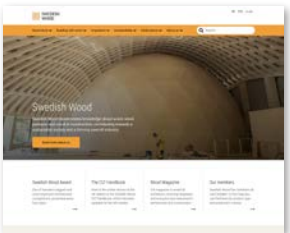

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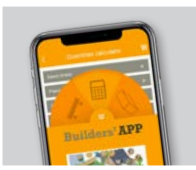

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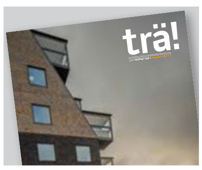

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Swedish Wood represents the Swedish sawmill industry and is part of the Swedish Forest Industries Federation. Swedish Wood represents the Swedish glulam, CLT and packaging industries, and collaborates closely with Swedish builders' merchants and wholesalers of wood products.

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