

Design of timber structures

Structural aspects of
timber construction

Volume 1

EDITION 2:2016



Introduction to design and design process
Structural properties of sawn timber and engineered wood products
Design of structural timber elements in ULS
Design of timber joints
Composite timber elements
Horizontal stabilization
Design for serviceability
Timber building systems for housing
Structural systems for infrastructure

SWEDISH WOOD™

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Preface

This is the second revised edition of *Design of timber structures Volume 1, Structural aspects of timber construction* published in 2015. Rules and standards change in pace with the development of society, why a publication of this type has to be reviewed regularly.

The book series *Design of timber structures Volume 1–3* has been produced to make it easier for structural designers to calculate timber structures and it is adapted to Eurocodes and to Swedish application rules EKS 10 (BFS 2015:6). It is being used for higher education at universities and institutes.

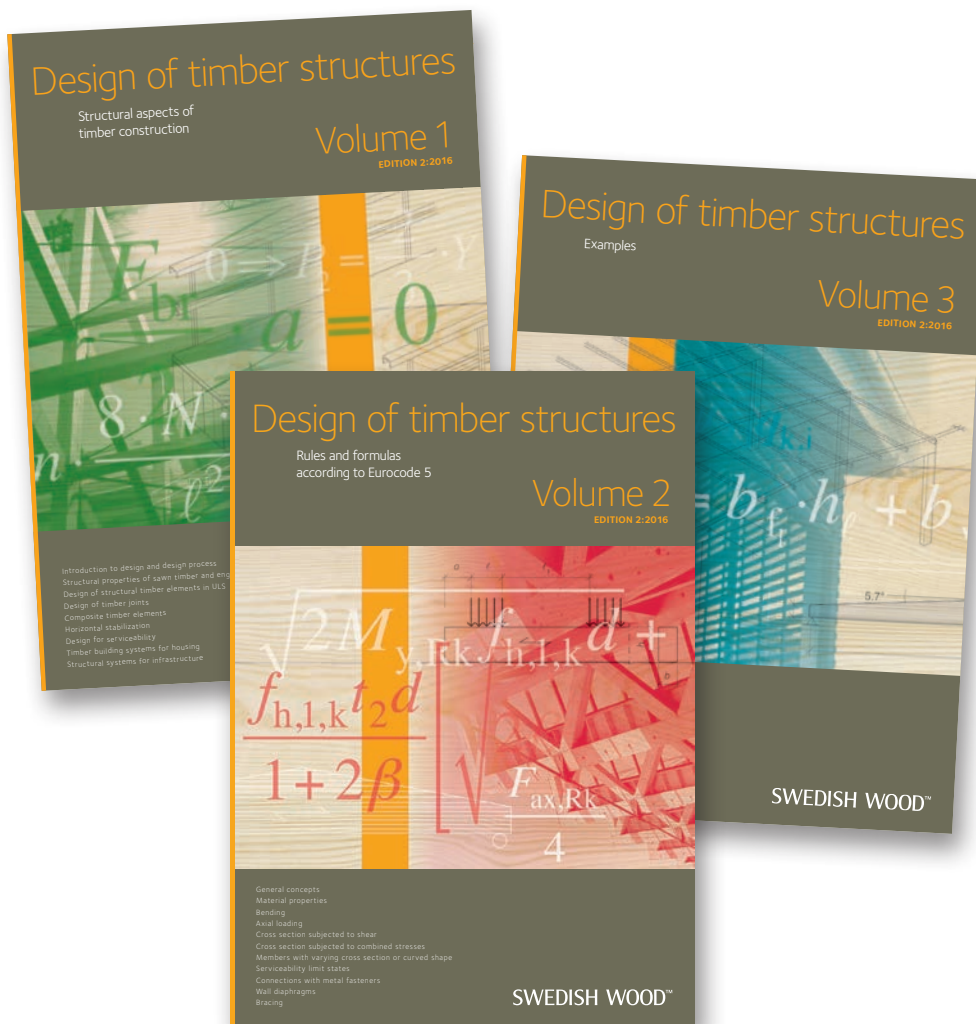
In the book series *Design of timber structures Volume 1–3* also *Volume 2, Rules and formulas according to Eurocode 5* as well as *Volume 3, Examples* are included. All three books are available in English and Swedish. Since the books are available in both languages and due to the nuanced content, our goal is that they will play a role for many users on different skill levels.

The authors of *Volume 1* are active in educating engineers at universities and institutes of higher education in Sweden, as Chalmers University of Technology, Royal Institute of Technology, Linnæus University in Växjö, Luleå University of Technology and Lund University. Also SP Sustainable Built Environment has contributed to the content.

More information about wood, glulam and timber construction can be found at www.swedishwood.com.

Stockholm, October 2016

Eric Borgström
Swedish Wood



Design of timber structures

Design of timber structures Volumes 1–3 are adapted to Eurocode 5 and the Swedish application rules EKS 10 (BFS 2015:6).

- Volume 1: Structural aspects of timber construction
- Volume 2: Rules and formulas according to Eurocode 5
- Volume 3: Examples



Dimensionering av träkonstruktioner

Del 1: Projektering av träkonstruktioner

Del 2: Regler och formler enligt Eurokod 5

Del 3: Dimensioneringsexempel

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Chapter 1

Introduction to design and design process

Robert Kliger

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1 Introduction to design and design process

1.1 General introduction to structural design

Structural design involves choosing the dimensions of load-bearing members and modelling the load-bearing structure 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.

1.2 Life phases of a building project

The life of a building project, irrespective of whether it is a house, industrial site or part of an infrastructure such as a bridge, can be divided into different phases, see Figure 1.1. These phases include design, construction, use of a building (working life) including maintenance and repairs and, finally, at the end of working life of a structure, demolition.

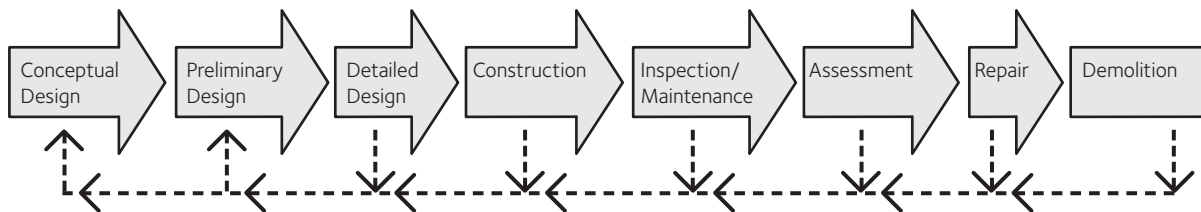


Figure 1.1: Phases of a project life, Niemeyer (2003).

Conceptual design is the first stage of a building project. During this stage, needs identified and examined, requirements for potential solutions are defined, potential solutions are evaluated and a suitable structural concept for further design is developed. Once the concept has been chosen and well defined, the second stage of the design process is reached by verification according to a required design code. The second stage comprises preliminary and detailed design. Forces loading the foundation, for example, are obtained after the preliminary design, which makes it possible to design the foundation at the same time as all the details in the main structure are designed. All possible design problems must be resolved before the construction phase starts. Once the construction phase is completed and a final inspection is made, a structure is ready for use as it was intended. Inspection and maintenance are important phases of the working life of all building projects. Different designs, building methods, materials and environmental exposure require different amounts of maintenance during the working life of a structure. All building components require some form of maintenance. The premature failure of some parts or a change in the intended use of a structure requires structural assessment and creates a need for repair or strengthening.

New structural designs, assessments of remaining life, strengthening and repair are common measures during the working life of a structure. At the end of the working life, a demolition phase is reached. In this phase, it is important that materials used in the structure are easy to dismantle, recycle and/or re-use.

1.2.1 Conceptual design

There are at least two different approaches describing conceptual design; one is by Kroll et al. (2001) and one by Engström (2002). The approach advocated by Kroll et al. is regarded as theoretical, but it is very useful when it comes to understanding problem solving and the creation of innovative solutions. The approach adopted by Engström is more practical and a short summary is presented here. A comparison of both methods is presented in Figure 1.2. The conceptual design approach can be applied both at the beginning of a building process and during the working life of a structure in a process of repair, strengthening and maintenance.

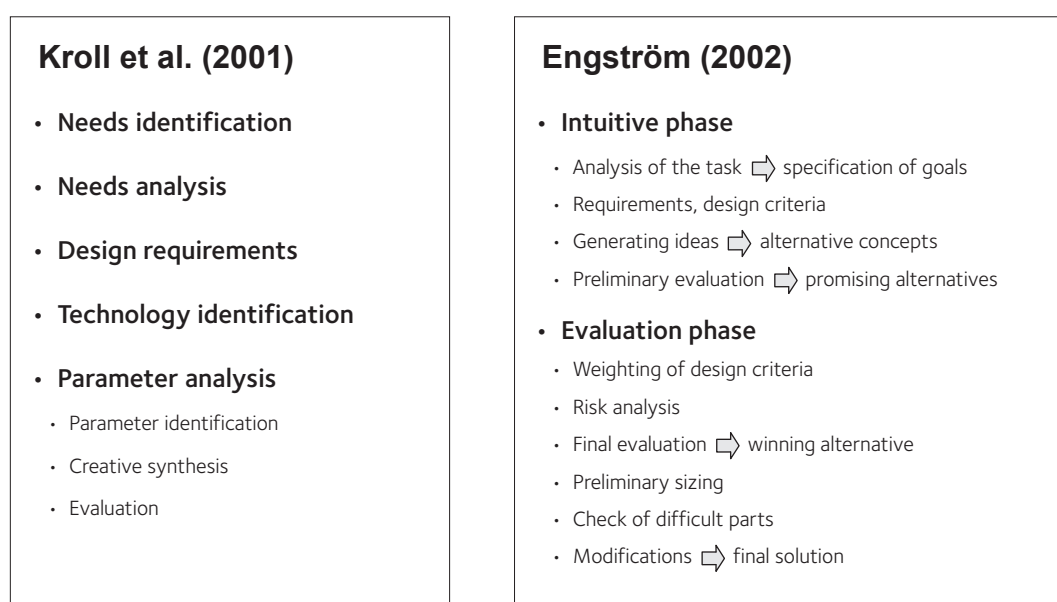


Figure 1.2: Main issues applied in a conceptual design.

Intuitive phase

This phase 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.

Understanding the client's wishes and collecting all the necessary information from possible stakeholders, which may affect the project, is a crucial part of the intuitive phase. Literature studies, a search for inconsistencies and interviews with users and other stakeholders may

provide the information necessary to define the aims and set a list of priorities comprising different requirements. 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. Creative and innovative ideas are based on good theoretical and practical knowledge. Various ideas should be represented by different structural materials or combinations of materials, a variety of structural systems and production methods. Such ideas are created by a systematic search and are produced within the intuitive thinking process.

It is always advantageous to produce too many concept ideas rather than too few. 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 have a well-documented motivation of a solution that was made and perhaps be able to backtrack to this step in the following process.

Evaluation phase and choice of the final concept

The evaluation phase includes ranking different concepts, performing 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 result of the risk analysis should be included in the final evaluation of some of the most promising concepts.

Preliminary sizing

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 thickness of the insulation may be a decisive factor when it comes to the thickness of structural members. The preliminary sizing often includes obtaining relevant load combinations in order to estimate sectional forces acting on a structural part which is being analysed.

Estimations of section sizes are based on:

- experience and recommendations
- preliminary design forces (including interaction)
- preliminary design stresses
- preliminary deflections.

Rough estimations may be based on:

- rule of thumb
- previous experience
- simple analytical tools
- simple design software
- tables, diagrams and charts.

An example of a rule of thumb that applies for a timber beam with a rectangular cross-section is that, if the ratio between depth and width ($h/b \geq 6$), full lateral support must be provided for the compression edge of the beam and the beam must be supported at the bearing points so that rotation is prevented.

1.2.2 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 subsystems 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.

The structural design process is regulated by the pertinent 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 series 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. 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 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.

1.2.3 Construction

Process

The construction process usually involves the translation of designs into the real production of a building. The design usually consists of drawings and specifications, usually prepared by a design team including surveyors, civil engineers, cost engineers (or quantity surveyors), engineers for electrician, structural design, plumbing and fire protection.

The increasing complexity of construction projects is creating a need for design professionals trained in every phase of the project life cycle and the development of an appreciation of the building as an advanced technological system requiring the close integration of many subsystems and their individual components, including sustainability.

Building Information Modelling (BIM) is the process of generating and managing building data during the life cycle of a building. Typically, it uses three-dimensional, real-time, dynamic building modelling software to increase productivity in building design and construction. The process produces a Building Information Model (BIM), which includes building geometry, spatial relationships, geographical information as well as quantities and properties of building components.

A BIM is a way to handle information which is created during the design and construction process but also during management, that is all the way from conceptual design to construction and through the service life. Modelling makes it possible to handle and re-use the relevant information about the building.

1.2.4 Inspection and maintenance

Assessment

Structures are usually designed for a service life of 50 to 100 years. Changes in load characteristics, deterioration with age, environmental effects and random actions may cause local or global damage to the structure. Any structural system in service is subject to age-related deterioration, leading to potential concerns regarding maintenance, health & safety, environmental and economic implications. The condition assessment of aged structures, see for example Paik and Melchers (2008), is an important part of engineering work, which includes:

- inspection and maintenance of aged structures
- residual strength assessment of deteriorated structures
- reliability and risk assessment of aged structures
- models to predict structural deterioration.

The main steps in a condition assessment should be:

- a) to record the damage, if any, and find the causes of distress
- b) to assess the extent of distress and to estimate the residual strength of structural components and the system including the foundation
- c) to plan the rehabilitation and retrofitting/strengthening of a structure when needed.

Detailed visual inspection and Non Destructive Testing (NDT) play an important role in condition assessments of existing structures. If the structure is damaged by a natural disaster or by an accidental load, a great deal of expertise is required to interpret the field observations and test results to make a proper assessment of the condition, as well as analysing and evaluating safety.

Refurbishment and repair

The nature of refurbishment and repair is markedly different from new construction work, as it is necessary to work within the constraints of a pre-determined situation. It is essential for those involved in this type of work to acquire an intimate knowledge of the structure that is going to be repaired. The documentation from the original design, the definition of the alternative use and documentation from the assessment of the structure define the new situation and provide the basis for design of the new construction work.

1.2.5 Demolition

The demolition phase is the final stage in the service life of a structure. Today, there is still a real opportunity for improvement and for reducing the environmental impact of the building sector when it comes to re-using and recycling building materials. The built environment makes an important contribution to the total national environmental impact. Government policies are expected to play an important role in improving the environmental performance of the building sector. The energy and material use in construction and during the service life of a structure represent a large amount of the total energy and material consumption. By using buildings functionally for as long as possible and by aiming during demolition for the maximum re-use of materials and the adequate treatment of the waste streams, a substantial environmental gain may be realised. In this respect, timber structures may have a very large advantage in comparison to other building materials. Demolition projects today are more complicated if sustainability is incorporated into this stage. The preparatory phase from policy to initiative must be well described. Already during the conceptual design, decisions are made which often have a major impact on the entire service life and sustainability of a structure, including the demolition stage.

1.3 Various requirements related to timber structures

1.3.1 Stakeholders

The most important parties are: a) society, through codes b) insurance companies c) commissioners of buildings (representing building owners or clients) d) contractors, builders or carpenters e) producers of prefabricated elements f) suppliers of timber products, see Table 1.1. Society, insurance companies and building commissioners set requirements for the components of the entire building, while a building contractor or builder sets requirements for the timber products. Society sets regulations to ensure a sufficiently low risk of collapse during the time the building is in use. The serviceability or functional requirements are often not regulated in detail. The demands are general and it is up to the commissioners of buildings to formulate more precise requirements. The building commissioner gives the contractor the job of constructing a building and then expects a certain function from the building components, often with no regard to the choice of material or design. A floor must fulfil deflection and vibration standards, no matter how this is achieved, for example. The contractor must satisfy both the commissioner's functional requirements and society's safety regulations. The floor joists must have a certain stiffness to meet the deflection requirements. The contractor imposes certain demands on the products. The properties of the products should permit swift and efficient production.

1.3.2 Requirement categories

The requirements set for timber or wood-based products can be divided into the following categories: safety, function, desirable properties and irrelevant characteristics, see Table 1.1. The safety requirements are set by society to prevent failure and damage which might cause personal injury or heavy financial costs. Public perception does not accept fatalities and injuries due to structural failure (in our homes, during working time, during recreation activities and other activities) during the design working life of a structure compared with fatalities arising from other hazards and events. As a result, structural safety is the principal requirement in all codes dealing with structural design. The functional requirements are directly associated with the user's needs and expectations.

Table 1.1 Matrix of requirements from stakeholders with examples at different levels, Johansson et al. (1990).

Interested parties	Requirements based on safety	Requirements based on function	Desirable properties	Irrelevant characteristics
Society, codes and standards	Safety of the structure. Safety from personal injury. Design methods. Eurocodes	Function in general terms	-	-
Insurance companies	Load-bearing capacity during lifetime of the structure. Design methods	Function in general terms. Stiffness, durability, fire, load-bearing capacity. Comply with standards	-	-
Building commissioner User	Comply with the design standards	Function in specific terms. Dimension, shape, durability, smell, load-bearing capacity, stiffness, vibrations	Aesthetic considerations, "warm" surfaces, shape, easy to paint, repaint and dismantle	Choice of material
Contractor, builder, carpenter	Stiffness	Dimension, shape, stiffness, smell, load-bearing capacity, moisture content, durability, mould, rot, wane, weight, location of knots, ability to nail, screw and glue	Right length, tolerant to moisture, no impregnation. Right price	Species, discoloration and appearance of built-in timber

The desirable properties are those that give the product a greater value for the user, apart from the basic needs. The functional requirement limits are often diffuse, since they are often related to comfort and human perception, which are difficult to quantify. The irrelevant characteristics represent those properties that do not affect the function of a product. For example, the building contractor does not generally care about the colour, number or size of knots in wall studs.

1.3.3 Requirements set by society – Eurocodes

General concepts

The fundamental requirements set by the European standard SS-EN 1990 for the reliability of construction works include: structural safety, robustness and serviceability.

Structural safety: A structure should be designed and executed in such a way that it will, during its intended life with appropriate degrees of reliability, and in an economic way, sustain all actions likely to occur during execution and use. This deals with the safety of people, the structure and its contents.

Robustness: A structure should be designed and executed in such a way that it will not be damaged by events such as:

- explosions
- impact
- consequences of human errors to an extent disproportionate to the original cause.

Serviceability: A structure should be designed and executed in such a way that it will, during its intended life with appropriate degrees of reliability, and in an economic way, remain fit for the use for which it is required. This deals with the function, comfort and appearance of the structure.

Durability

The durability of a structure or a part of it assumes to have a certain length of working life based on the fact that appropriate maintenance is given and that it remains fit for use during the design working life. The structural design is suitably adapted to the surroundings of the structure or its parts. Load-bearing parts should be designed in such way that deterioration will not impair the durability and performance of the structure exposed to the anticipated level of maintenance. Guidance for design with regard to working life of a structure is given in Thelandersson et al. (2012).

1.3.4 Requirements set by customers and end users

The requirements set by customers and end users of timber products differ for various products and uses. It is very important to distinguish between indoor and outdoor environmental conditions, between load-bearing and non-load-bearing members and between different construction methods.

Function, buildability, durability and maintenance

These requirements differ between building clients and users on one hand and contractors, carpenters and builders on the other hand, see Table 1.1. For the first group, the types of building material are not important, as long as they serve their function. Dimension, shape, stiffness, vibration, load-bearing capacity, smell and durability are the most important quality parameters to fulfil the function and maintenance of the building, irrespective of the structural material. For the second group, if timber is used as a structural material, the shape of timber at the moisture content used during the construction and the ability to nail, screw and glue, if necessary, are very essential properties to facilitate more rational construction. The moisture content in itself is not a problem for a building contractor. However, the effect of too high moisture content or too much variation in moisture content can have a very negative impact on the function, buildability, durability and maintenance of a structure. The amount of knots in the timber is usually not a problem for a building contractor. However, if the knots are situated in a place where a piece of timber needs to be spliced or connected in a mechanical type of connection, such as nails or screws, this may create a problem for a carpenter.

1.4 Eurocodes – general assumptions, limit states and detailed design

1.4.1 General assumptions and verifications

The general assumptions for any structural design process are well specified in SS-EN 1990. Examples of these assumptions include:

- the choice of the structural system and the design of the structure is made by appropriately qualified and experienced personnel
- the execution is carried out by personnel with the appropriate skills and experience
- adequate supervision and quality control is provided in design offices and during the execution of the work, that is factories, plants and on site
- the construction materials and products are used as specified in SS-EN 1990 to SS-EN 1999, in the relevant execution standards, in reference material and product specifications, see Figure 1.3
- the structure is adequately maintained
- the structure is used in accordance with the design assumptions.

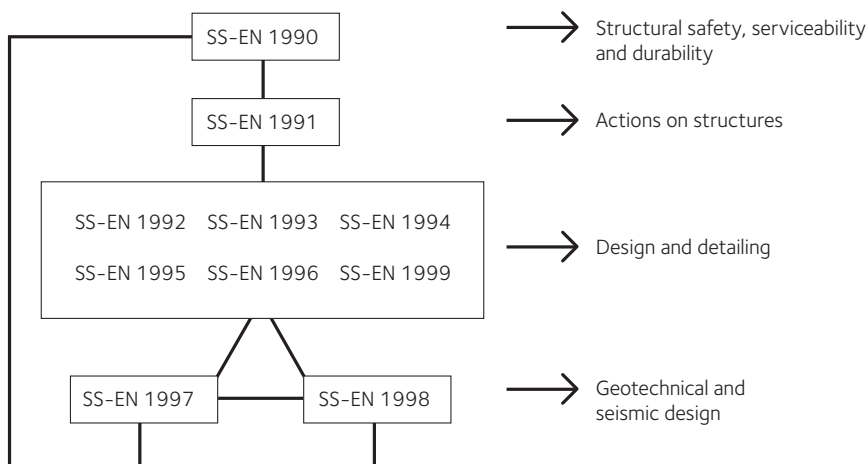


Figure 1.3: Links between Eurocodes.

SS-EN 1990 establishes principles and requirements for safety, serviceability and durability of structures, describes the basis of their design and verification and gives guidelines for related aspects of structural reliability. SS-EN 1990 is also applicable for the structural appraisal of existing structures, developing the design of repairs and alterations or assessing changes of use. An acceptable degree of reliability for the majority of structures is expected to be obtained by design and execution according to Eurocodes 1 to 9, together with appropriate quality assurance measures. The Eurocode for the design and detailing of timber structures is SS-EN 1995.

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 for example due to excessive deformation or vibration. The verification or design procedure is illustrated in Figure 1.4.

To verify a structure or part of it, a designer has to distinguish between design situations, see Table 1.2, agents, actions, combination of actions and their effects, cf. Figure 1.4. 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.

When verification is performed by calculations, this often means using well-established design models which reflect the real mechanical response of the designed structure in the current limit state. However, it is frequently not possible to imitate exactly the real structure or real load magnitude or its variation. As a result, design models are often simplified, depending on the level of accuracy to be achieved or the stage in the design process.

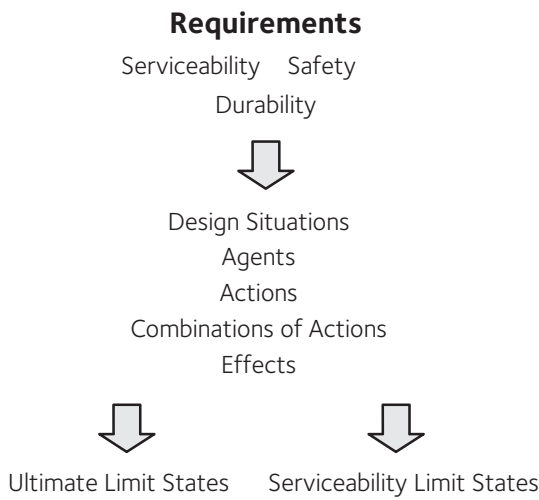


Figure 1.4: Requirements and terminology used in the design procedure.

Table 1.2 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

1.4.2 Principles for limit state design

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 $Z > 0$. 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:

$$P_f = P(Z \leq 0) = \Phi[(0 - \mu_Z) / \sigma_Z] = \Phi(-\beta) \tag{Eq. 1.1}$$

where Φ is the standardised normal distribution function, $\mu_Z = \mu_R - \mu_E$ and $\sigma_Z^2 = \sigma_R^2 + \sigma_E^2$. $\beta = \mu_Z / \sigma_Z$ is the so called safety index.

The safety index β is often used as a measure of safety level and is related to the same reference period T as is used for the load effect E . The relationship between P_f and β is shown in Table 1.3. When β increases, the probability of failure decreases, that is the safety level becomes higher.

Table 1.3 Relationship between probability of failure P_f and safety index β .

β	0,00	1,28	2,32	3,09	3,72	4,27	4,75	5,20	5,60
P_f	0,5	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}

Structural codes based on limit state design usually define a formal safety level in terms of a minimum target safety index β_{target} (or maximum permissible probability of failure). For Eurocode, 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.

1.4.3 Verification based on the partial safety factor method

In normal engineering design, an evaluation based on safety index is not practical. Instead this so called partial safety factor method is used. Referring to the fundamental case in ultimate limit state introduced in Section 1.4.2 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, that is:

$$R_d > E_d \quad \text{Eq. 1.2}$$

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 A . The design value G_d of permanent actions is determined by:

$$G_d = \gamma_G \cdot G_k \quad \text{Eq. 1.3}$$

where γ_G is partial safety factor for permanent load G and G_k is the characteristic value for permanent actions, typically defined as the 50 % quantile or the mean value. Design values for variable actions are defined by:

$$Q_d = \gamma_Q \cdot Q_k \quad \text{Eq. 1.4}$$

where γ_Q is partial safety factor of variable action Q and Q_k is the 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:

$$f_d = \eta \frac{f_k}{\gamma_M} \quad \text{Eq. 1.5}$$

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

The idea behind partial safety factors is that they should reflect the uncertainty of each type of action. The partial safety factor γ_Q is therefore usually larger than γ_G , due to the fact that

variable loads (for example snow, wind and imposed load) display greater uncertainty than permanent loads (for example self weight). In addition, the values of partial safety factors should be chosen so that the desired level of reliability is achieved. For this purpose, the partial safety factors are calibrated and optimised by calculating the reliability indices β obtained when the partial safety factor is used in design for a set of simple and representative design cases. The objective of this calibration procedure is to determine the partial safety factors so that different design situations lead to uniform reliability levels β for different design situations as close as possible to the target reliability level β_{target} .

An important basis for the partial safety factor method is the concept of characteristic value, which should be based on a clear statistical definition. It is obvious that 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 (for example 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 in average once during a 50-year period.

Permanent actions are more or less constant during the whole service life of the structure. In contrast, variable actions change in time and their characteristic value corresponds to rare points in time when extreme levels occur. When more than one variable action contributes to a certain load effect E , combination values of design actions must be determined to account for the reduced probability that extreme values of all variable actions will occur at the same time. 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, which are described by a reduction of the characteristic value Q_k by factors ψ_0 , ψ_1 and ψ_2 . The following load values are introduced in EN 1990:

- 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 actions 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 1.2. These rules define how permanent loads and variable loads shall 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:

$$E_d = \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad \text{Eq. 1.6}$$

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

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

To verify a structure, it is necessary to consider different design situations, see Table 1.2. The selected design situation should be sufficiently severe and represent all the conditions that can reasonably be foreseen as occurring during the execution and use of the structure. For verifications in the ultimate limit state (ULS) or serviceability limit state (SLS), there are a set of prerequisites for calculations.

ULS deals with:

- rupture
- collapse
- loss of equilibrium
- transformation into a mechanism
- failure caused by fatigue.

SLS deals with:

- deformation
- vibration
- cracks
- damage in a negative way affecting use.

The procedure of limit state design can be described as:

- define the relevant limit states for which the structural behaviour is to be checked
- for each limit state, determine the appropriate actions and load combinations to be considered
- using appropriate structural models for design, and taking account of the inevitable variability of parameters such as material properties and geometrical data
- verify that none of the relevant limit states is exceeded.

In ULS, the following types of limit state should be considered when relevant:

- EQU: relating to the static equilibrium of a structure or any part of it which is regarded as a rigid body.
- STR: relating to the internal failure or excessive deformation of a structure or its member.
- GEO: relating to the failure or excessive deformation of the soil or rock (ground).
- FAT: relating to the fatigue failure of a structure or structural members.

1.5 Concepts used for the limit state design of timber and wood-based products

As a building material, timber 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, it has different properties in different directions. It is furthermore hygroscopic, which means that the moisture content in the material constantly changes with the relative humidity of the surrounding environment. When a designer chooses a certain grade of concrete or steel, the manufacturers of these materials produce them according to well-established procedures to obtain a specific grade with more or less controlled variability. When it comes to timber, however, the designer chooses a grade which has been verified by some type of non-destructive strength grading of sawn timber, see Section 2.5. This means sorting existing material, which was formed during a long period of time and often in very different conditions, based on statistical relationships. Sawn timber is produced from a very large variety of logs sawn from many different trees. These trees come often from forests with very different silvicultural conditions and a very long rotation period.

These prerequisites of strength-graded timber and the variable characteristics of wood-based products are taken into account when designing timber structures.

Some of the characteristics which are specific when it comes to various wood-based product design are:

- the duration of load for various types of load
- the moisture content in timber and service classes
- partial factors for material properties and adjustments using various modification factors
- the difference in material response when the loads are applied in various directions in relation to the grain orientation of timber.

1.5.1 Load duration classes

Timber experiences a significant loss of strength over a period of time, see Section 2.4.3. 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 which may occur in practice and associated actions, see Table 1.4.

Table 1.4 Load duration classes.

Load duration classes	Accumulated duration	Examples of loading
Permanent (P)	> 10 years	Self weight
Long-term (L)	6 months – 10 years	Storage
Medium-term (M)	1 week – 6 months	Imposed floor load Snow load
Short-term (S)	< 1 week	Wind load
Instantaneous (I)		Wind gusts Accidental load Single concentrated roof load

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

The influence of load duration on the strength capacity of timber is taken into account by assigning a factor k_{mod} as a function of service class defined in Section 1.5.2 and one of the five load-duration classes in Table 1.4. The load-duration factor k_{mod} is a reduction factor for the characteristic strength of timber and wood-based products, varying between 0,2 and 1,1. Only for instantaneous load and dry wood, this factor is $> 1,0$ and results in an increase in the characteristic strength.

1.5.2 Effect of moisture content and service classes

Moisture content and variations in moisture content play a very significant role for all properties of timber and wood-based products. Moisture content affects both strength and stiffness. To incorporate this effect in design, three service classes have been defined in SS-EN 1995. They are:

- Service class 1** – when the average moisture content in most softwoods does not exceed 12 %
- Service class 2** – when the average moisture content in most softwoods does not exceed 20 %
- Service class 3** – when the average moisture content in most softwoods exceeds 20 %.

1.5.3 Partial factors for material properties and adjustments using various modification factors

In order to take account of uncertainty in the resistance model used for design, together with the unfavourable effects of geometrical deviations of materials, characteristic strength values are divided by the partial factor γ_M in order to obtain the design strength of the material, see Eq. 1.5. The partial factor γ_M varies between 1,3 and 1,2 for most wood-based material in the ULS, but, for accidental combinations and all combinations in the SLS, the partial factor γ_M is equal to 1. There are many different adjustments of strength using modification factors. Commonly used factors in Eurocode 5 are: k_h , $k_{c,90}$, k_{crit} , k_{dis} , k_m , k_{sys} , k_v , k_{vol} .

They correspond to:

- size effect (k_h),
- an increase in compressive strength perpendicular to the grain ($k_{c,90}$)
- a reduction in strength due to the effects of lateral and torsional instability (k_{crit})
- an adjustment in the design tensile stress/strength perpendicular to the grain in the apex zone of a double-tapered, curved or pitched cambered beam (k_{dis})
- stress redistribution for beams loaded in flexure about both major axes (k_m)
- member strength properties when several members or components are equally spaced and connected by a continuous load distribution system (k_{sys})
- a reduction in shear strength due to a notch (k_v)
- an adjustment in the design tensile strength perpendicular to the grain in the apex zone of a double-tapered, curved or pitched cambered beam which exceeds a certain reference volume (k_{vol}).

To describe the effect of creep instantaneous deformations are multiplied with a factor which describes the additional deformation developing over time (k_{def}).

Further background and details about these factors are given in later chapters of this book.

1.5.4 Difference in material response when the loads are applied in various directions in relation to the grain orientation of timber

Both mechanical and physical properties differ significantly in different directions when it comes to fibre orientation, that is the radial, longitudinal and tangential direction of sawn timber result in very different properties. In design, it is very important to recognise whether timber is loaded parallel or perpendicular to the grain. In the first case, timber has very good strength, but, in the latter case, the strength is typically 1/10 of that in the longitudinal direction and often close to zero, see Section 2.4.

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Chapter 2

Structural properties of sawn timber and engineered wood products

Marie Johansson

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2 Structural properties of sawn timber and engineered wood products

2.1 Forestry and the production of sawn timber

In 2009, 3,95 billion hectares of land were classified as forest in the world. Of these forests 25 percent were located in Europe, 21 percent in South America, 16 percent in Africa, 15 percent in Asia, 18 percent in North and Central America and 5 percent in Oceania. Just under half of the wood use in the world is used as industrial wood (pulp, paper and wood products); the other half is used for fuel, the main part of this in developing countries. The countries that produce the largest amounts of sawn timber are USA, Canada, Russia, Germany and Sweden. Canada, Russia and Sweden are also large exporters of sawn timber.

2.1.1 Europe

The forest sector is an important economic sector for Europe. Studying Europe as a whole (including all of Russia) the forest sector accounts for about 1 percent of the GDP and employs almost four million people. Europe has 25 percent of the world's forest resources but only 17 percent of the total land area. The largest part, 81 percent, of these forest resources is within the Russian Federation. This means that 44,3 percent of the European land area is covered with forests. In all countries in Europe the area and the amount of timber in the forest is increasing every year. The differences within Europe is, however, very large. In most of the available statistics, Europe is divided into three regions, Western Europe, Eastern Europe (incl. the Baltic States) and CIS countries (former Soviet republics).

The forests in Western Europe are mostly privately owned (over 70 percent) while in Eastern Europe the share of privately owned forests is increasing from having been mainly state owned before 1990. In the CIS countries the forest is to almost 100 percent publicly owned. The fragmented owner structure in Western Europe has made it complex to manage the forests and in many countries strong private forest owner associations are formed. Forest management are in most European countries highly regulated by the government, meaning strict regulations regarding harvesting and re-planting. The European forests (excl. Russia) are to a large extent a mix of natural and planted forests. The forests in Russia are still to a large extent natural, especially east of the Ural. Another important aspect of forest is the social dimension. More than 90 percent of the European forests are open for public access and the forest area for recreation is increasing.

The countries in Eastern Europe and the CIS countries before 1990 produced about half the sawn wood in Europe. Political changes caused a drastic decrease in production after that. Today the production in these countries is again rising and with a large production of Engineered Wood Products. The demand for bioenergy from the forest has increased in Europe during the last ten years and this demand is expected to increase as more countries try to replace fossil fuel with renewable energy. The demand of renewable energy has increased the amount of material taken from the forests; such as material from thinnings, residues from harvesting, branches and stumps.

The European forest can be classified in boreal forest (The Nordic countries and Northern Russia) while the middle of Europe and the southern parts of Russia can be classified as temperate



a)

b)

Figure 2.1: Example of two softwood species. a) Spruce (*Picea abies*), b) pine (*Pinus Sylvestris*) (Photo Marie Johansson).

broadleaf forest. In the boreal forests softwood species; spruce, pine and larch are dominating, see Figure 2.1. The softwoods are dominating also in the rest of Europe but there are also more hardwoods to be found, as for example beech, chestnut and alder. In production of sawn timber, especially for the construction industry, the softwood species are more commonly used.

2.1.2 Swedish forestry

On the Swedish level the forest industry is a very important industry sector and stands for 9 – 12 percent of the total turnover, added value and employment of the Swedish industry. Slightly over 12 percent of the total export from Sweden is based in the forest industry. The total export value in 2014 was 124 billion Swedish crowns, of this the pulp and paper industry is responsible for approximately 3/4 while sawn timber and engineered wood products is responsible for 1/4.

The Swedish forest area is in total 27,5 million hectares; of this 78 percent is productive forest while the remaining part is areas set aside for conservation and unproductive areas. The Swedish forest is to 50 percent privately owned while private and public companies own 25 percent and 14 percent respectively. The growth in the Swedish forests has been larger than the felling since 1950. The most common species in the Swedish forests are spruce (40 percent) and pine (39 percent). The remaining forest is birch (12 percent) and other deciduous species. The birch is mostly used in the pulp and paper industry.

Spruce and pine are used both in sawmills and in the pulp and paper industry. In 2013, 76 million m³ sub (solid volume under bark) logs was brought to the Swedish industry, of this approximately 50 percent was sent to the sawmilling industry and the other 50 percent to the pulp and paper industry. The sawmilling industry yielded about 70 percent of this into sawn timber and engineered wood products while the remaining 30 percent was wood chips sent to the pulp and paper industry. The total volume of sawn timber from the sawmilling industry in 2013 was 17 million m³ of which 12 million m³ was exported. The largest export market for Swedish sawn timber in 2014 was Great Britain (2,7 million m³) with Egypt, Norway, Germany and Denmark as next largest export markets (Skogsindustrierna 2014).

The use of Swedish sawn timber can be seen in Figure 2.2, which shows that a large part is used for packaging. Of the remaining sawn material 55 percent is used directly in the construction industry.

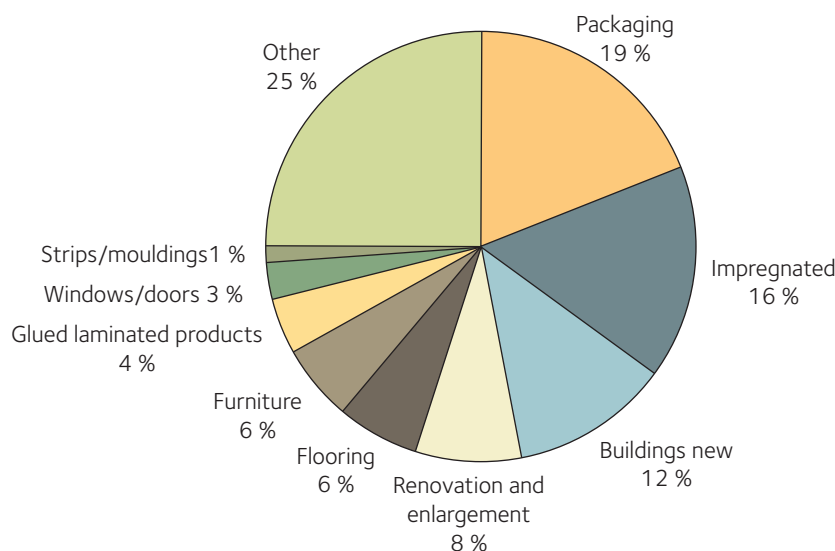


Figure 2.2: Estimated use of sawn timber in Sweden, Dackling (2002).

2.1.3 Production of sawn timber

The following subchapter describes the production of sawn timber in a Swedish (Nordic) context and is concentrated on the production of structural timber from spruce (*Picea abies*). Forests in Sweden are dominated by managed (planted) forests, where the rotation time is approximately 80 years. Most forests are today planted and managed through several thinnings during the growth. At the age of approximately 80 years the trees are harvested. After harvesting the logs are taken to a sawmill or a pulp industry, and the branches are cut into wood chips and taken away to be used as fuel wood. The ground is then replanted with new spruce or pine plants.



Figure 2.3: Harvester and logger in action in the forest
(Photo courtesy of left: Rottne industri AB and right: Gremo AB).

The normal way of harvesting trees in Sweden is to make clear-cuts in the forest of an even-aged stand using a harvester, see Figure 2.3. The harvester has capability to measure the diameter of a log and cross-cut the logs to lengths optimised for the market demand. The logs are cut to pulp logs (smaller diameter) or saw logs. The logs are then transported by a logger to the nearest forest motor road for storing before transport by lorry to a sawmill or pulp industry. For the sawmilling industry the maximum log length in Sweden is normally 5,5 meters with a diameter less than 40 cm.

At the sawmill the logs are normally graded by VMK (Virkesmättningsrådet) into different quality classes to decide the price of the logs for payment to the forest owner. The logs are normally also graded into different length and diameter classes. The logs are often debarked and run through a metal detector before being sent into the saw line. In the beginning of the saw line the logs are scanned to register at least the outer shape of the logs and in more advanced scanners the internal properties are also recorded using x-ray. The result from the scanner is used to optimize the sawing pattern. The logs are sawn to boards of different sizes and sorted into batches of different dimensions (and quality classes). The next step in the process is to stack the boards before kiln drying. The kiln drying is made to reduce the moisture content in the boards to levels between 12 % and 18 % depending on the end use of the material. There are in principle two kinds of kiln driers that can be used, a chamber kiln or a continuous kiln. In the continuous kiln the stacked boards are brought into one end of the kiln and slowly (5 – 7 days) transported through the kiln. In the chamber kiln the stacked boards are placed within the chamber of the kiln and the doors closed. The relative humidity and the temperature in the kiln are possible to control and adapt so that the drying process is performed to minimize the energy needed and to optimize the quality of the sawn timber.

After kiln-drying the boards are un-stacked and run through an adjustment mill where the boards are visually scanned and sorted into different visual grades and unwanted parts of the boards are sawn off. The boards are then often run through a planer to get their final size. After planing, boards to be used as structural timber are strength graded into different strength classes. (More about strength classes and grading can be found in Section 2.5). The sawn timber is now ready to be packed, wrapped and shipped to its final destination.

2.2 Structure of timber

The wood material is built up by trees and the structure of the material is optimised for the circumstances the tree is exposed to. The tree stem is grown round to optimise the tree against wind loads, it grows branches to carry up leaves (or needles) to take part in photosynthesis, it stores chemicals in the centre of the tree to use as weapons in case the tree is exposed to a threat and it forms reaction wood in a leaning tree to optimize its internal stresses. The tree is

also built with internal pre-stressing (growth stresses) to minimize the risk of crushing the wood at high compressive stresses. These are some examples of how trees are optimized for their living. When man harvest the tree not all of these optimizations are beneficial for the use of the sawn wood. To be able to use wood in the best possible way it is necessary to know how wood is built up and the influence this structure have on the behaviour of the material.

2.2.1 Material structure of wood

Wood is a natural composite built up of mainly three elements; 50 percent carbon, 6 percent hydrogen and 44 percent oxygen in the form of cellulose, hemicellulose and lignin. Nordic softwood species have approximately the proportions 40 – 45 percent cellulose, 25 – 30 percent hemicellulose and 25 – 35 percent lignin. Cellulose is a long organic molecule chain built up by glucose units, see Figure 2.4. The number of glucose units can vary from a few up to 15 000 depending on the location of the cellulose molecule within the cell wall. The cellulose molecules have a hydroxyl group (–OH) on the side which can form hydrogen bonds to the next cellulose chain. Hemicellulose is the name for a group of shorter branched molecule chains and lignin is the generic term for a group of three dimensional polymers, Bodig and Jayne (1982).

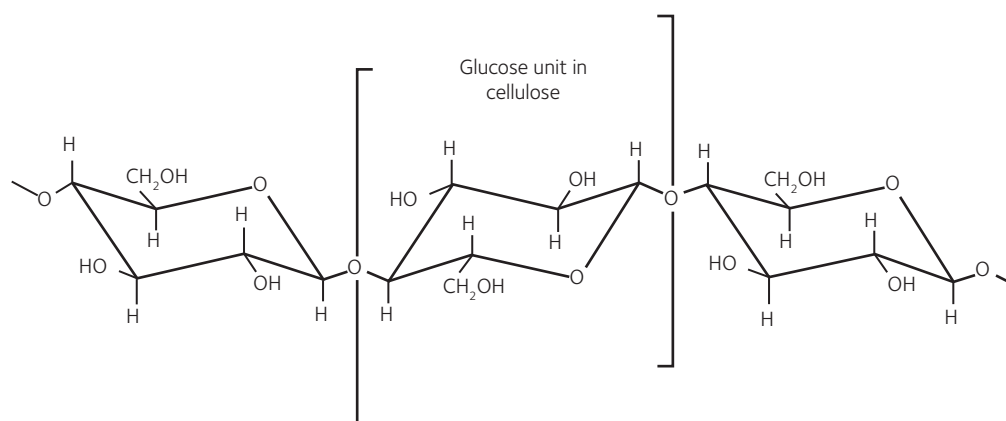


Figure 2.4: Basic structure of a cellulose molecule.

The cellulose in wood are collected in crystalline strands enclosed by hemicellulose and lignin. The strands are most often called microfibrils. These strands make up the cell wall in the living tube-shaped wood cell. The most common cell type in softwood is the tracheid which is tube-shaped with a size of approximately 2 – 4 mm in length and 0,1 mm in width (diameter). The cell wall in wood consists of four layers. On the outside there is a thin layer usually called the primary wall where the strands are randomly oriented. Inside of this is the proper cell wall consisting of three parts. The inner and outer of these three layers (S1 and S3) have the strands oriented around the tube shaped cell. The main purpose of these layers is

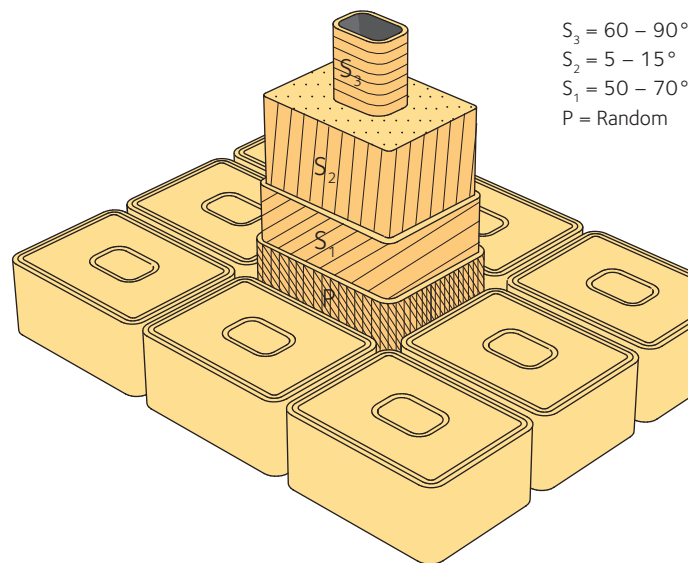


Figure 2.5: The structure of a wood cell. The figure shows the cell wall constructed of 4 layers around the central cell cavity. Typical microfibril angle in these layers is given in the figure, Hoffmeyer (1995).

to maintain the shape of the cell. Between these layers there is a thicker layer (S₂) with most of the strands oriented more or less parallel with the length direction of the cell, see Figure 2.5. This middle cell wall layer (S₂) represents about 85 % of the cell wall thickness and is consequently the layer with the greatest influence on the properties of the cell. The angle between the axial direction of the cell and the microfibrils in the S₂-layer are often called the microfibril angle (MFA). The cells are bound together by a layer called the middle lamella consisting mainly of lignin; this acts as an adhesive layer between the cells.

The wood tissue in softwood is made up from a large number of these tracheids (90 % of the volume), often called fibre or grain. The structure of hardwoods is often more complicated and also includes other types of cells. The tracheids are oriented mainly in the longitudinal direction of the tree. A growing tree forms new cells through cell division in a layer between the wood and the bark; the cambium. During the spring period the tree has a need of cells that can transport water and nutrition up and down the tree. The cells that are formed during this time is called earlywood cells and have thin cell walls (2 µm) and large cell cavities to facilitate transport of liquids in the tree stem. During the summer the need for transport of water and nutrition is less and the tree also needs greater strength to withstand external loads such as wind and snow. During the summer and autumn the tree therefore forms cells with thick cell walls (10 µm) and smaller cell cavity. These cells are normally called latewood, see Figure 2.6. The proportion of latewood cells has significant influence on the mechanical properties of the wood material. The different cell types are often possible to distinguish by their colour; the early wood is seen as a thick lighter coloured band while the late wood is seen as a thinner band of darker wood.

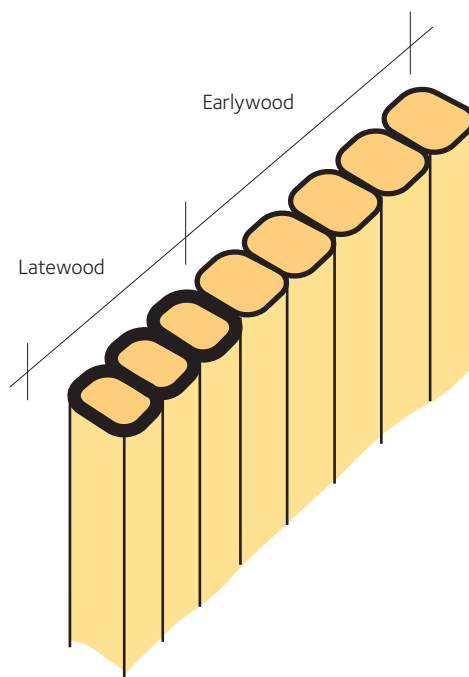


Figure 2.6: Principle of the cell wall growth during one year, note the thin-walled earlywood cells and the thick walled latewood cells. The number of cells in a real tree is higher.

The wood cells have small openings (pits) on the sides that connect them to neighbouring cells. These openings allow for mineral solutions and nutrition to be transported within the tree. There are various types of pits occurring in different tree species. The type of pit is the main factor governing moisture permeability in different species.

The only wood cells that are truly alive, that is have the capacity for cell division, in a tree are the outermost cells close to the bark (cambium). The cells deeper into the tree have the possibility to transport liquids (sapwood). In the cells inside the sapwood zone, the tree starts to store substances that protect the tree against biological attacks that is extractive substances. The wood in this area of the tree is called heartwood. In pine the heartwood can be distinguished by its darker colour, see Figure 2.7. In spruce it is not possible to see the difference between sapwood and heartwood based on the colour. The centre of the tree is often referred to as the pith.

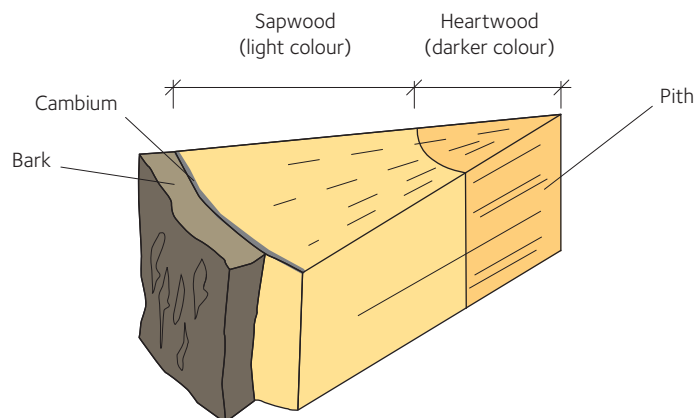


Figure 2.7: Heartwood and sapwood.

2.2.2 Natural characteristics of wood

There are a number of characteristics of wood that for engineering use can be seen as defects, a few of them are described below.

2.2.2.1 Knots

All trees need leaves (hardwoods) or needles (softwoods) for transpiration, respiration and the photosynthesis process. The area needed for this process is so large that it is necessary for the tree to spread the leaves and needles out onto branches. Each year a tree grows in length at the top but also forms new branches at the sides. These branches are each year gradually embedded in the stem wood through the addition of more material on the outside of the trunk and the branches; see Figure 2.8. The material in the branch is the same as in the main stem and the tissue systems of the two structures are interconnected. A longitudinal section through a knot shows that the fibre orientation around the knot is distorted and that the fibres pass in a sweep around the knot. The presence of knots has a negative influence on most mechanical properties of sawn timber. The fibres around the knot are no longer continuous when the wood is sawn and thus affecting the strength, at least in the main fibre direction. The size, shape, number and location of knots influence the strength of the sawn timber. For structural use, sawn timber with smaller and fewer knots are often graded into higher classes.



Figure 2.8: A knot seen from all four sides of a wood specimen 45 x 45 mm in square section (Photo Bertil Enquist).

2.2.2.2 Spiral grain angle

The fibres of wood tend to grow in a spiral around the trunk of the tree. The most accepted explanation, from the perspective of the tree, is that it creates more flexibility in the stem against wind loads. Spiral grain angle is defined as the angle between the fibre direction and the direction of the pith. In the case of Norway spruce, the spiral grain angle is zero close to the pith. A left-handed spiral grain then develops and continues to increase during subsequent growth, often reaching a maximum value of 3 – 4° within the first ten annual growth rings. The grain angle then decreases towards zero. Later in the life of the tree, the grain angle may change direction to become right-handed. The grain angle in this direction increases slowly as long as the stem growth continues.

2.2.2.3 Juvenile wood

Juvenile wood is formed during the first 5-20 years of the growth of a tree. Juvenile wood is formed near the pith. It is characterised by a progressive increase in cell dimensions and changes in cell characteristics and the pattern of cell arrangement. According to most assessments, juvenile wood have less desirable properties than mature wood. The cells in the mature wood may be three to four times longer than those found in juvenile wood. In addition to cell length, there is a difference in cell structure, see Figure 2.9. There are fewer latewood cells in juvenile wood. In softwoods, the density is typically 10 – 15 percent lower in the juvenile core and the strength is normally 15 – 30 percent lower than that of mature wood. Within the cell, the microfibril angle in the S_2 part of the secondary wall is characteristically larger in juvenile wood. Juvenile wood is reported to shrink along the grain up to nine or ten times as much as mature wood.

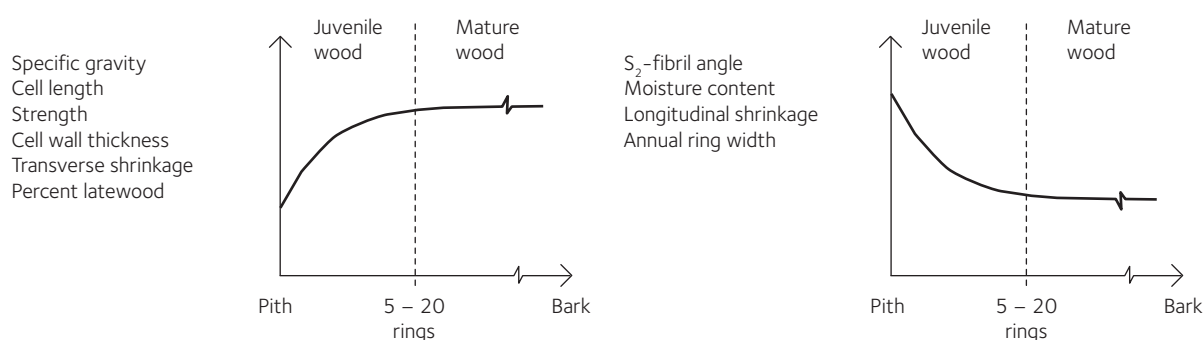


Figure 2.9: Juvenile to mature wood transition of properties.

2.2.2.4 Reaction wood

Reaction wood is formed because that each part of a tree strives to remain in its genetically predetermined, static equilibrium position. Any displacement results in the formation of reaction wood so arranged that the displaced organ is brought back to its original position by the shortest possible route. In this theory, gravity has its place since the displacement must be made in relation to the gravitational field.

The reaction wood present in gymnosperms (softwoods) and angiosperms (hardwoods) is best referred to as compression wood and tension wood, respectively. Compression wood and tension wood are names that have been used for a long time and they also indicate the functions of these tissues in the living tree. Compression wood is formed on the compressed side of a leaning softwood and tension wood on the side subjected to tension. Tension wood is correspondingly formed on the tensioned side of a leaning hardwood. As softwoods are commonly utilised in Sweden for structural purposes the following text is focusing on compression wood. Compression wood fibres are shorter and rounder than for normal wood. They have distorted tips and thick cell walls. Intercellular spaces often form between the rounded cells, which results in less connection between the fibres. The S_2 layer is much thicker in compression wood cells than in normal cells but with a microfibril angle in the S_2 layer in the range of 30 – 50°. In severe compression wood, no S_3 layer is produced, but it is sometimes replaced by a warty layer. Needless to say, every form, between severe compression wood and normal wood, can also be present in wood.

In chemical terms, the same substances are present in both compression wood and normal wood. The differences relate to the amount of the substances. Compression wood contains 30 – 40 percent more lignin and 20 – 25 percent less cellulose than normal wood. The amount of hemicellulose is in general not changed. Compression wood has higher specific gravity, a lower fibre saturation point, reduced permeability, lower radial and tangential shrinkage but substantially larger longitudinal shrinkage, higher compressive strength but lower tensile strength, a lower modulus of elasticity and it is more brittle in terms of fracture than normal wood, Timell (1986).

2.3 Physical properties of wood

2.3.1 Wood and moisture

One of the external factors that have a large influence on the properties of wood and timber is water. The moisture content in wood is always depending on the relative humidity in the surrounding air. When water enters into a dry piece of timber the molecules are firstly bound to the surface of the cellulose strands in the cell walls (to the hydroxyl groups). When all the places in the cell walls are filled the water molecules starts to fill up the cell cavity. The amount of water in the wood is usually defined as moisture content (moisture ratio) u , often expressed in % by weight, see Equation 2.1:

$$u = \frac{m_u - m_{\text{dry}}}{m_{\text{dry}}} \cdot 100 \quad \text{Eq. 2.1}$$

Here u is the moisture content (ratio) in %, m_u is the mass of the wood at moisture content u and m_{dry} is the mass of the wood after drying in 103 °C for 24 hours. The moisture content level where the adsorption and desorption of water is equal is called the equilibrium moisture content (EMC). The relationship between relative humidity and the equilibrium moisture content can be seen in Figure 2.10:

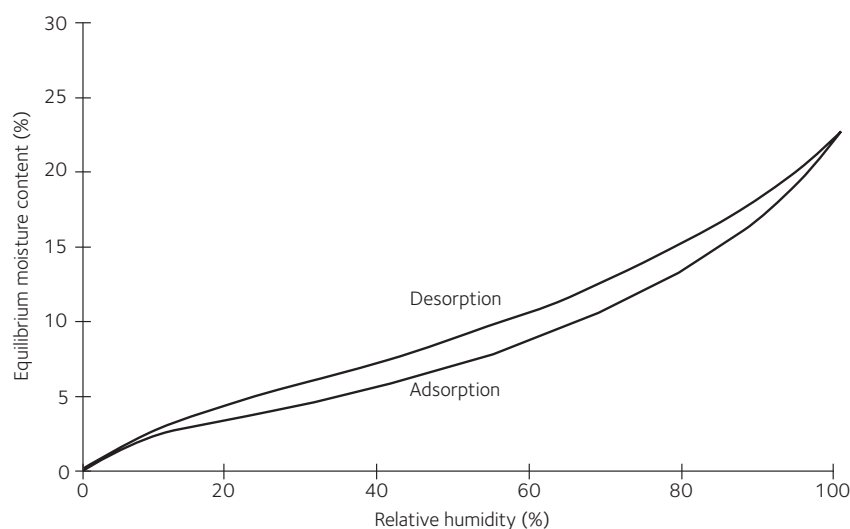


Figure 2.10: Relationship between equilibrium moisture content and the relative humidity.

One concept often used is fibre saturation point (FSP) which is defined as the moisture content when the cell wall is filled with water molecules but the cell cavities are still empty. For softwoods grown in Europe the FSP is often between 27 % and 33 %. At higher moisture content levels the cell cavities are starting to fill up with water. The fibre saturation point is a cut-off point for many properties of the wood material. For example swelling of the wood material occurs when water molecules are bound in the cell wall, water in the cell cavity do not influence the outer dimensions of a piece of wood.

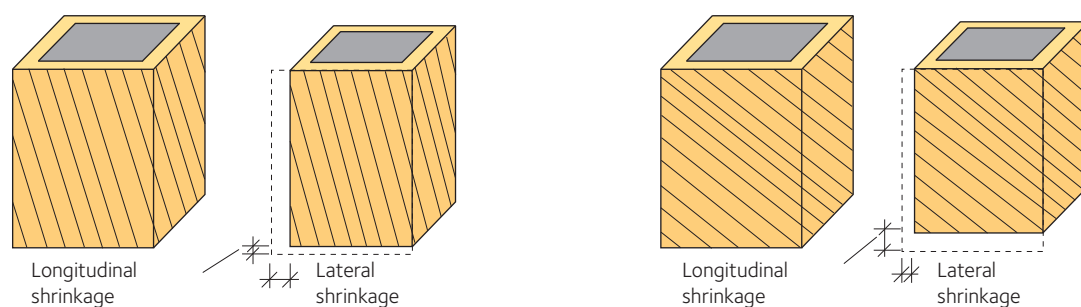
2.3.2 Shrinkage and swelling

In the cell wall, the water is bonded to the surface of the micro fibrils. When the moisture content is reduced, the microfibrils come closer to each other, which make the wood shrink. Since most of the microfibrils are in the S_2 layer, the micro fibril angle of this layer will govern much of the behaviour during shrinkage and swelling.

Since the microfibrils are normally inclined at a small angle to the longitudinal axis, almost all the shrinkage occurs in the transverse directions. Juvenile wood and compression wood have microfibril angles that are much larger than those of normal wood, which results in far larger longitudinal movements, see Figure 2.11. Typical values for shrinkage strain in the different directions are for a change in moisture content of 1 % is:

- Longitudinally 0,0001
- Radially 0,0015
- Tangentially 0,0030

There is a factor of approximately 2 in difference between the shrinkage in radial and tangential direction, this can be explained by the difference in wood structure in these two directions. The shrinkage in the longitudinal direction is very small but for large lengths this can still be necessary to take into account. The presence of compression wood or juvenile wood can have a large influence on, especially, the longitudinal shrinkage which can be a hundred times larger in compression wood than in normal wood.



a) Shrinkage at normal micro fibril angle.

b) Shrinkage at large micro fibril angle
(compression wood or juvenile wood).

Figure 2.11: Diagrammatic sketch of micro fibril arrangement and shrinkage in normal wood and in compression or juvenile wood. Note the difference in micro fibril angles.

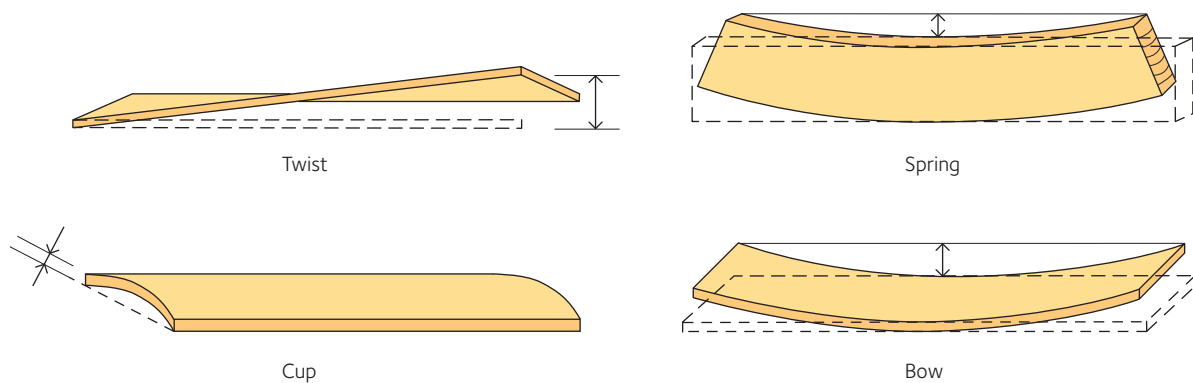


Figure 2.12: Definition of distortion/warp modes.

2.3.3 Distortion of timber

The varying shrinkage within a piece of wood can cause geometrical changes of the wood cross-section that can make them difficult to use (distortion), see Figure 2.12. Distortion can be divided into four different forms: twist, spring, cup and bow. Spring is the edgewise deviation of a piece of timber from a straight line from end to end, while bow is the flat-wise deviation. Twist is defined as lengthwise spiral distortion. Cup is the deviation flat-wise from a straight line across the width of the board. For studs used in the building industry, twist is the distortion mode that causes the largest problems. Spring also causes problems, while moderate bow can be accepted. Cup is not normally a problem for structural timber, but it causes severe material losses during planing.

Twist can be explained by the combined effect of the annual ring orientation, spiral grain angle and shrinkage. The fact that the shrinkage perpendicular to the actual fibre direction is large, combined with a spiral grain angle of some degrees as well as the curvature of the annual ring causes each annual ring to twist and consequently the whole board. The cup deformation is mainly caused by the difference in shrinkage between the radial and tangential direction in combination with the circular shape of the annual rings. The two bending modes bow and spring are caused by both variations in longitudinal shrinkage within a single board and the release of growth stresses in the tree/log when it is sawn.

The size of distortion of sawn timber can be influenced by different sawing patterns and different drying regimes. A good sawing pattern, for example sawing boards that do not include the pith, can reduce the twist. The kiln-drying regime is also important, different drying schedules, air flow and weights on top of the sawn timber can help to minimize the distortion. It can also be beneficial to measure the spiral grain angle and treat boards with large spiral grain angle differently, with for example heavier loads in the kiln.

2.3.4 Density

Density is a very important physical property of wood; it is correlated to almost all mechanical properties. Density (ρ) is defined as:

$$\rho = \frac{m}{V} \quad \text{Eq. 2.2}$$

where m is the mass (kg) and V the volume (m^3). The density is moisture dependent as both the mass and the volume is dependent on the moisture content. Therefore the density for wood has to be defined also in terms of moisture content. The most commonly used definition in timber engineering is the density ρ_{12} , that is based on the mass and volume at 12 % moisture content. This is the moisture content used in all standard tests for wood strength.

The density of the cell wall is about $1\,500\text{ kg/m}^3$. This means that density is a measure of the amount of pores in the wood material. The normal density (ρ_{12}) for softwood grown in the Nordic countries varies between 300 and 600 kg/m^3 .

2.4 Mechanical properties of wood and timber

It is usual to divide wood into small clear specimens (wood) and large specimens that include natural characteristics (timber) such as; knots, spiral grain angle, juvenile wood and reaction wood. Clear wood specimens consist of only straight wood fibres without anomalies and hence all properties are only dependent on the properties of the wood fibre. For timber the effects of the natural characteristics will be large and to a great extent decide the properties and behaviour of the specimen.

2.4.1 Strength and stiffness of wood

The structure of wood, tube shaped cell in a lignin matrix, results in different properties in different directions, that is the material is anisotropic. With a structure of tubes in a matrix it is easy to understand that the compression strength is higher parallel to the fibres than perpendicular to the fibres. Wood thus has different properties in different directions and it is important to keep track of the loading direction. For a complete picture of the properties of wood it is necessary to define all the stresses in Figure 2.13.

Figure 2.13 shows the definition of the different stress in wood. For timber the difference between R and T direction is often disregarded and the directions are named σ_0 or $\sigma_{//}$ and σ_{90} or σ_{\perp} for the parallel and perpendicular to the fibre direction respectively. To describe the “real” behaviour of wood within the elastic range 12 constants are necessary; for example the modulus of elasticity E_L, E_R, E_T , the shear modules G_{LR}, G_{LT}, G_{RT} and the Poisson ratios $\nu_{LR}, \nu_{RL}, \nu_{LT}$,

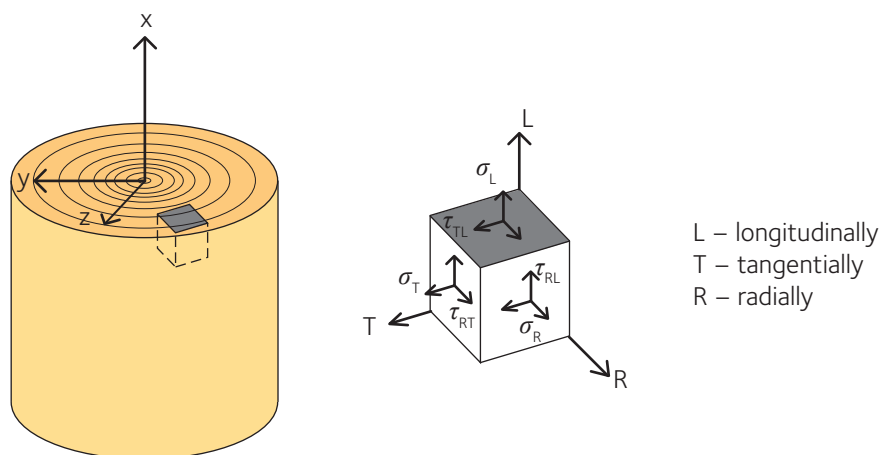


Figure 2.13: Definition of normal- (σ) and shear- (τ) stresses in different directions in wood.

ν_{TL} , ν_{RT} , and ν_{TR} . Normally, the Poisson ratios are assumed to be pair wise equal, which make it possible to eliminate three. By disregarding the difference in tangential and radial direction, the number of variables can be reduced to six, often denoted $E_{//}$, E_{\perp} , $G_{//}$, G_{\perp} , $\nu_{//}$ and ν_{\perp} , where $//$ denotes parallel to the fibre direction and \perp perpendicular to the fibre direction. The modulus of elasticity in wood is often shortened to MOE.

2.4.1.1 Tension parallel to the fibre direction

Loading small clear specimens (that is without knots) it is possible to investigate the effect of loading the wood material in different directions. A pure tension test in the fibre direction show that the stress-strain relationship is almost linear up to failure, see Figure 2.14. On the material level two failure modes are possible (or a combination of them); the middle lamella breaks and the fibres are being pulled out of their matrix material or the fibre can break. The strength of wood in tension parallel to the fibre direction (f_t) is very high; the failure stress is often of the magnitude of 100 MPa. The failure is, however, often very brittle.

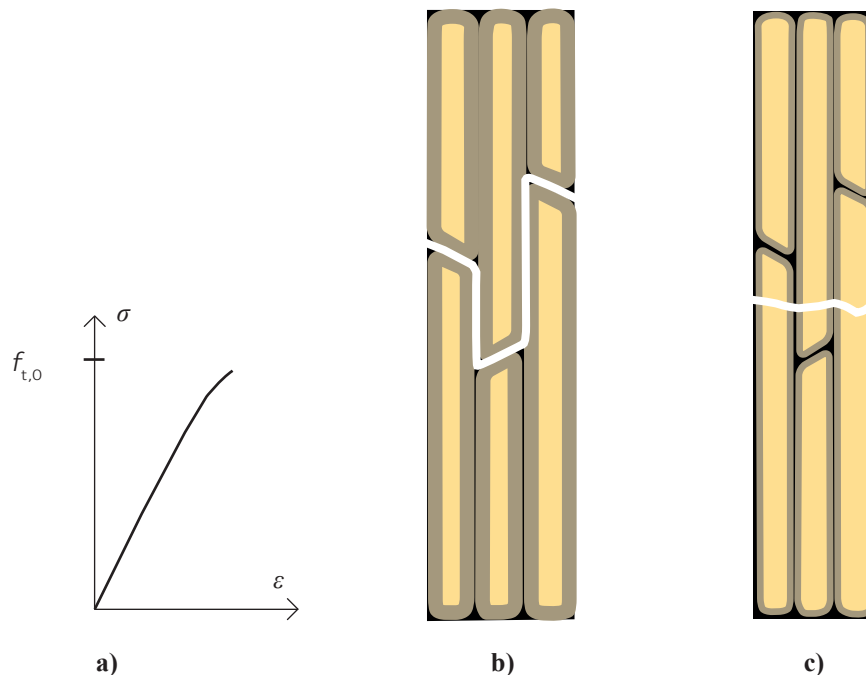


Figure 2.14: a) Stress-strain relationship for clear wood loaded in tension parallel to the fibre direction. Typical failure in pure tension parallel to the fibre direction for b) thick-walled late wood and c) thin walled early wood.

2.4.1.2 Tension perpendicular to the fibre direction

If the wood is loaded in tension perpendicular to the grain the forces to pull apart the fibres or break the fibres are much lower. The stiffness is also lower in this direction. Figure 2.15 shows that the strength of wood loaded perpendicular ($f_{t,90}$) to the fibre direction is very low, in most cases it is not possible to assume a higher strength than 0,5 MPa. It is also possible to see that the stiffness is much lower in this direction.

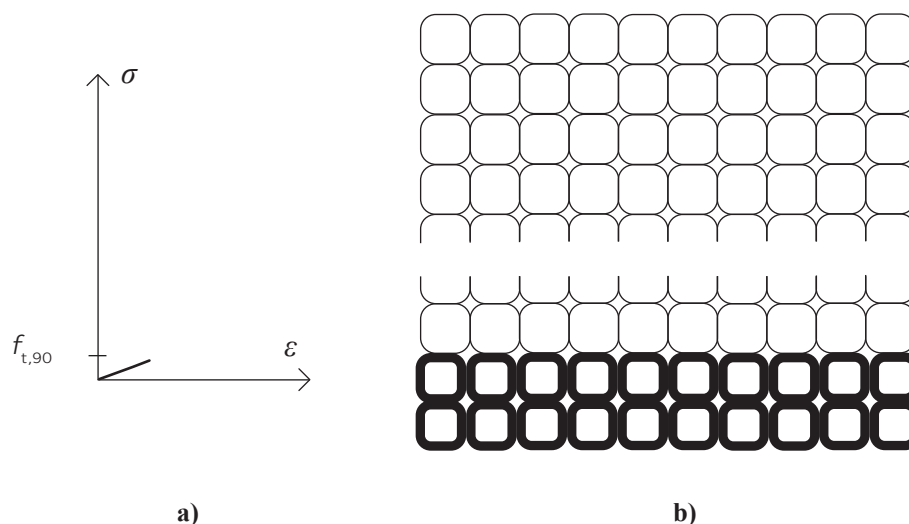


Figure 2.15: a) Stress-strain relationship for clear wood loaded in tension perpendicular to the fibre direction. b) Typical failure in pure tension perpendicular to the fibre direction.

2.4.1.3 Compression parallel to the fibre direction

In compression, the wood material will behave differently. In compression parallel to the fibre direction the main stress will be parallel with the axial direction of the fibres (tubes). Tubes loaded axially are very stable and can withstand a high load level. When the load is too high some fibres will start to buckle and be driven into the other fibres. When the buckling behaviour in the wood starts the possibility to take higher load will diminish and the behaviour can be classified as plasticising. Figure 2.16 shows the stress-strain relationship for clear wood loaded parallel to the fibre direction. The compression strength for wood in pure compression parallel to the grain (f_c) is around 80 MPa. Typical values for ϵ_c are 0,8 – 1,2 % with an ultimate strain level $\epsilon_u \approx 3\epsilon_c$.

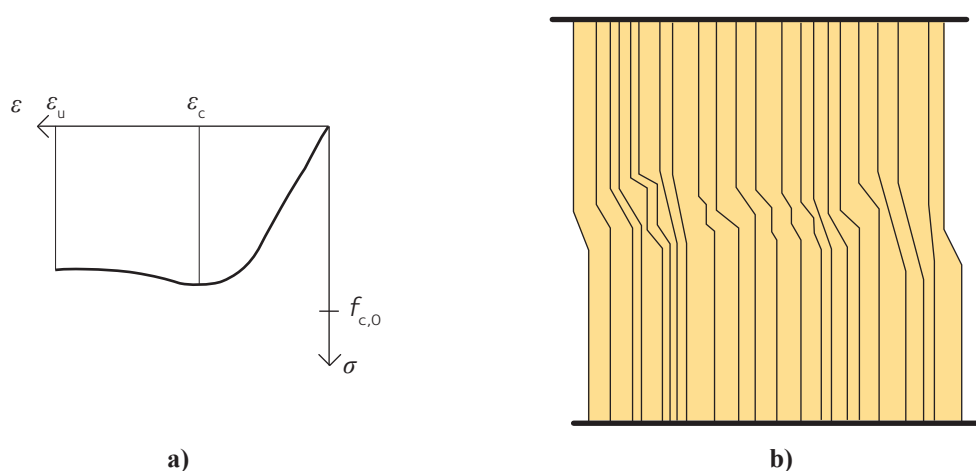


Figure 2.16: a) Stress-strain relationship for clear wood loaded in compression parallel to the fibre direction. b) Typical failure in pure compression parallel to the fibre direction.

2.4.1.4 Compression perpendicular to the fibre direction

In compression perpendicular to the fibre direction the tube shaped wood cells will be crushed. Crushing a tube shaped structure from the side requires low force and consequently both the stiffness and the strength for this form of loading are low. It is however not possible to define a true failure in compression perpendicular to the grain. When all the cells are crushed the stress level can increase again. Figure 2.17 shows the stress-strain relationship for wood loaded perpendicular to the grain. The maximum stress level in compression loading perpendicular to the grain (f_{c90}) is 3 – 5 MPa. The failure in compression perpendicular to the grain is then defined as the stress level that gives 10 percent remaining deformations.

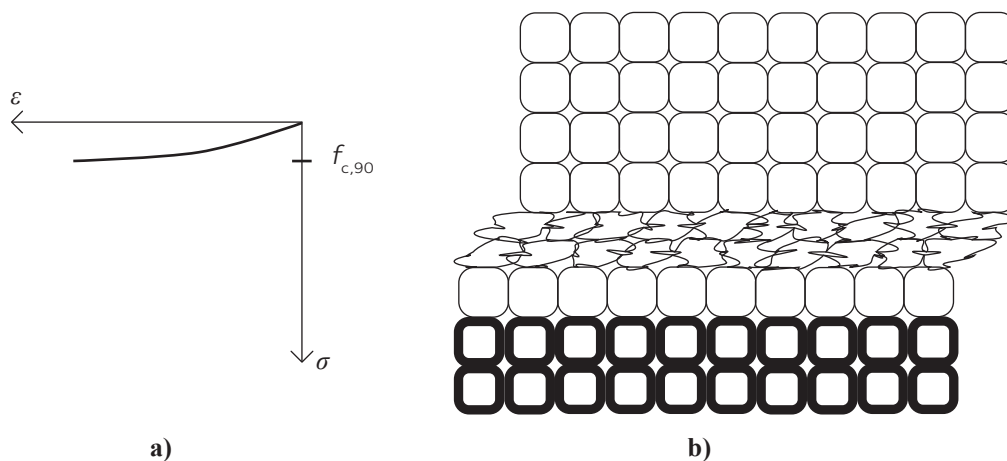


Figure 2.17: a) Stress-strain relationship for clear wood loaded in compression perpendicular to the fibre direction. b) Typical failure in pure compression perpendicular to the fibre direction.

There is a difference in the amount of deformation for the same load level depending on the orientation of the annual ring. The modulus of elasticity perpendicular to the grain is higher in the thick-walled latewood than in the thin-walled earlywood. This is, however, not taken into account when designing timber structures.

2.4.1.5 Loading in an angle to the grain direction

If the material is loaded in an angle between perpendicular to grain and parallel to grain the strength is somewhere between the extreme cases showed above. One equation for calculating the compression strength depending on the angle (α), see Figure 2.18, between the load direction and the fibre direction was proposed by Hankinson (see for example Dinwoodie 2000), see Equation 2.3.

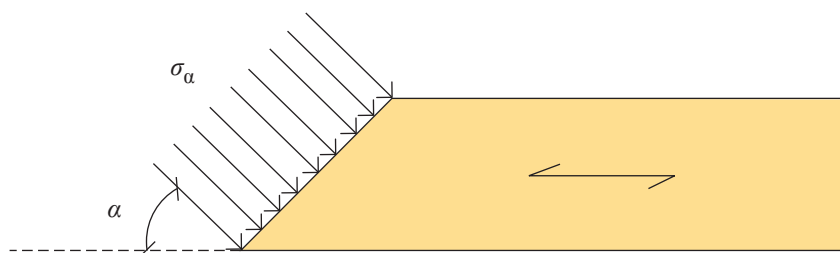


Figure 2.18: Definition of the angle α between the stress (force) and the fibre direction.

$$f_{\alpha} = \frac{f_0 \cdot f_{90}}{f_0 \sin^2 \alpha + f_{90} \cos^2 \alpha} \quad \text{Eq. 2.3}$$

This equation has showed to work well also in tension. In graphic form, see Figure 2.19, it is possible to see that already a small deviation from the parallel direction gives a considerable reduction in strength.

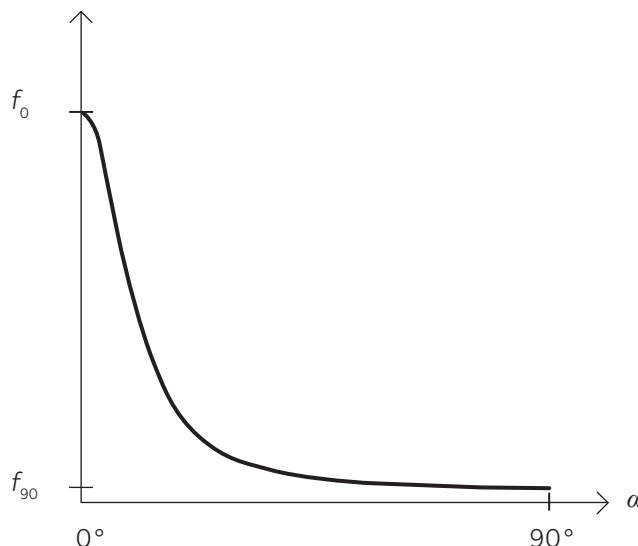


Figure 2.19: Relationship between the failure strength and the angle (α) between the fibre direction and force direction according to Hankinson, Dinwoodie (2000).

2.4.1.6 Shear strength

The shear strength of timber is highest in planes parallel to the fibre direction, the strength varying between 5 and 8 MPa. The shear strength perpendicular to the fibre direction τ_{RT} (rolling shear) is often between 3 and 4 MPa (that is half the strength).

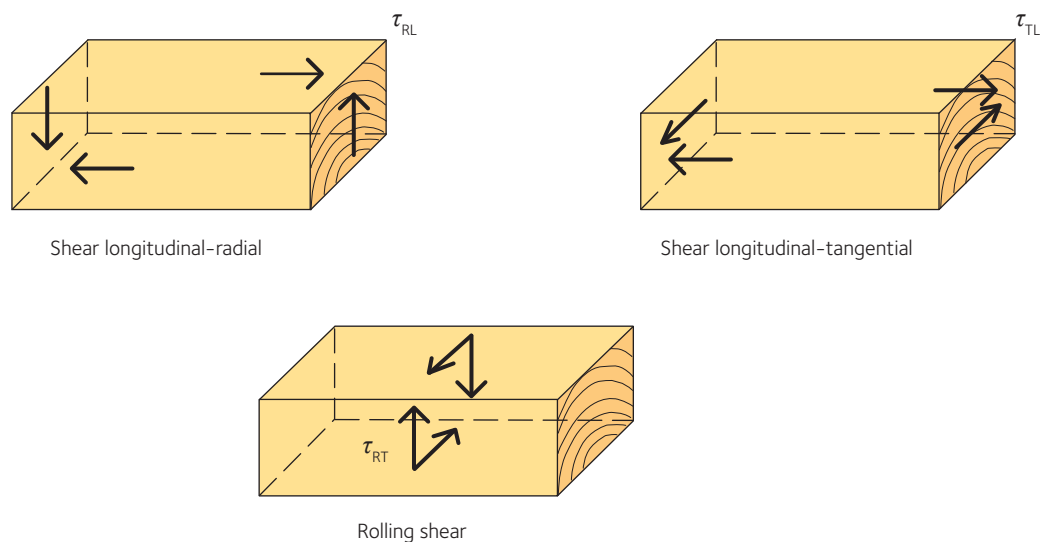


Figure 2.20: Shear in different directions, Carling et al. (1992).

In timber structures shear in the τ_{RL} and τ_{TL} direction are the most common. The shear strength in the τ_{TL} direction is lower since the failure line is going through the weaker early wood layer compared with the τ_{RL} direction where the failure area has to go through both the early and late wood. For the structural engineer it is however not possible to distinguish between these two and the lower of the two values are used in the codes. For the rolling shear failure the tube shaped fibres are rolling against each other instead of cutting the cells in the longitudinal direction. Rolling shear is not so common in normal square beams, but must be considered in for example glulam I-beams. The shear strength as well as the tension perpendicular to grain strength is highly influenced by drying cracks in the material.

2.4.2 Strength and stiffness of structural timber

Larger pieces of wood (sawn timber) do not only contain straight wood fibres but also anomalies such as knots, compression wood, spiral grain and so on. It is therefore not possible to predict the mechanical behaviour of sawn timber directly from the properties of clear wood. For example the fibre deviations around a knot can give large local stresses due to a loading in an angle to the grain.

The effect of a knot on the strain (and stress) field can be seen in Figure 2.21. This specimen, cross-section 45×45 mm including a knot, see Figure 2.8, was loaded in tension up to a level of 20 kN (average stress 10,0 MPa). The strains were recorded with the measurement system ARAMIS™ manufactured by the company GOM during loading. The system is based on evaluation of a random pattern applied to the surface that deforms along with the material. Two CCD cameras are placed at different angles in front of the specimen which enables stereoscopic pictures to be taken of the specimen. On the basis of the movement in the random pattern it is possible to evaluate 3D displacement, rotations and strains.

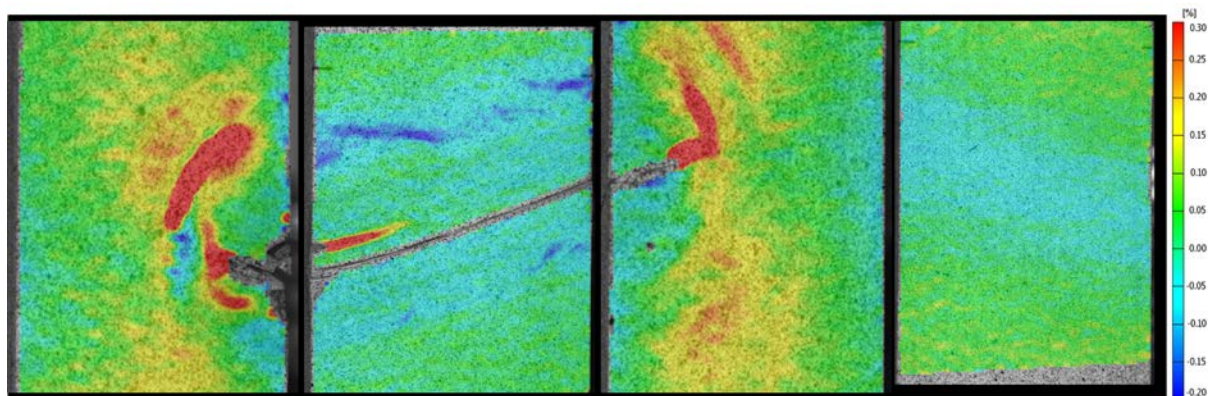


Figure 2.21: Longitudinal surface strains (ϵ_y) for maximum load 20 kN.

The resulting strain shows that the strain distribution around a knot is far from even. Areas far from the knot, with straighter grain, exhibit less strain than areas close to the knot.

Strength values used for structural timber are therefore determined by testing with a standardised methodology. This means that the strength values more reflect a strength value on the element level than an actual strength value on the material level. The strength value for bending is according to the European standard SS-EN 384 and SS-EN 408 determined from

a bending test performed with four-point loading. The beam shall have a length of 19 times the beam height (H), and be loaded with a clear span of $18H$ with a distance between the inner loading points of $6H$. The values for stiffness and strength determined from a loading test to failure are defined using the elastic section modulus. The determined strength values have to be recalculated to a height of 150 mm, which is the standard height for sawn timber. This manner of testing results in strength values including the effect of defects in the material (as well as possibly a small effect of instability). The method of utilizing the elastic section modulus when determining the strength also means that the strength values can only be used in design models that are based on elastic theory. The strength values in tension and compression are determined in the same manner with testing of a nominal dimension and determining a strength value based on this dimension.

The method of determining strength and stiffness values through testing results in different strength values for different loading modes such as bending, tension and compression.

2.4.3 Influence of moisture

The mechanical properties of wood are affected by the moisture content. The lower the moisture content the higher the strength and stiffness. Often the strength and stiffness of timber are said to increase linearly with decreasing moisture content below the fibre saturation point. Above the fibre saturation point there are no great changes in strength and stiffness with increasing moisture content.

Tests have shown that the influence of moisture content is different in different loading directions. For clear wood the change in properties for 1 % change in moisture content can be seen in Table 2.2. This relationship is valid for moisture contents varying from 8 % to 20 %. Tests have shown that the influence of moisture content on full size timber is not as pronounced as on small clear wood specimens, especially in some loading directions. The tension strength is almost independent of the moisture content for full size timber while the compression strength is very much affected by moisture content.

The influence of moisture content is normally taken into account in design codes by reducing the strength values for timber used in environments where high moisture content can occur. In Eurocode 5 all structural elements are assigned into different service classes depending on the highest moisture content the element can be subjected to during its lifetime, c.f. section 1.5.2.

Table 2.2 Approximate change in mechanical properties of clear wood for a 1 % change in moisture content, Hoffmeyer (2003).

Property	Change (%)
Compression strength parallel to the fibre direction	5
Compression strength perpendicular to the fibre direction	5
Bending strength parallel to the fibre direction	4
Tension strength parallel to the fibre direction	2,5
Tension strength perpendicular to the fibre direction	2
Shear strength parallel to the fibre direction	3
Modulus of elasticity parallel to the fibre direction	1,5

2.4.4 Influence of time

Another factor with a great influence on the mechanical properties of wood and timber is time. Tests have shown that the bending strength decreases with an increased loading time, that is duration of load (DOL). This relationship seems to be valid in all loading modes but is especially important in bending. The original curve for strength versus time was established at the Forest Products Lab in Madison, USA in the 1950's based on small clear specimens and is known as the Madison curve. The equation for the Madison curve is nearly proportional to the logarithm of the time over which the load is applied, see Equation 2.4:

$$f = 90,4 - 6,3 \log t \quad \text{Eq. 2.4}$$

where f is the stress level (percent) and t is the effective duration of the load in hours. The curve shows that the strength after one year has decreased to approximately 60 percent of the original short term strength, see Figure 2.22. The Madison curve shows a critical stress level (50 percent) under which there is no duration of load effects.

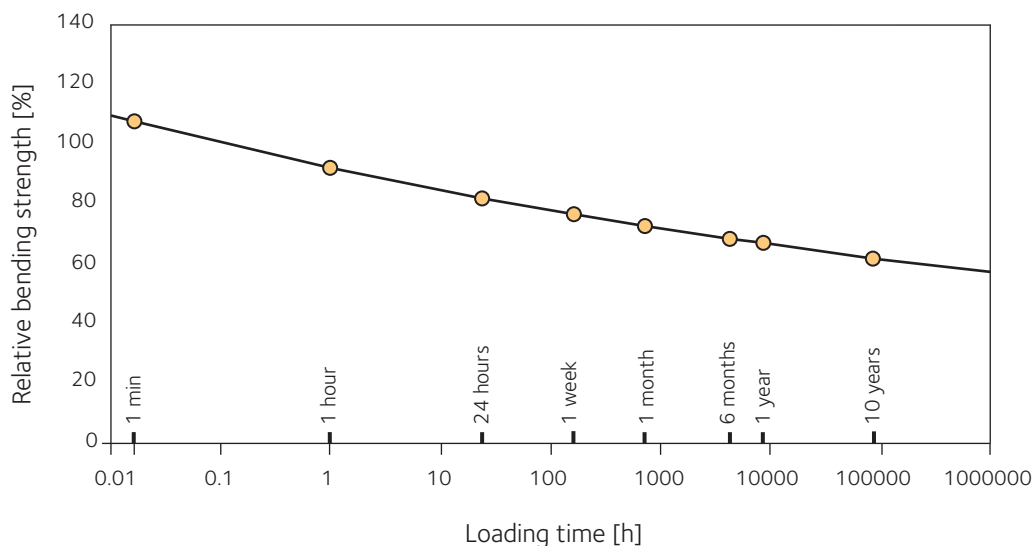


Figure 2.22: The relationship between relative bending strength and loading time. The strength at 5 minutes has been set to 100 percent.

Further studies have shown that for full size timber this curve may be different. Some research has indicated that the duration of load effect is less for low-strength timber than for high-strength timber, and that the effect is smaller than for clear wood, Hoffmeyer (2003). Other studies indicate that the DOL-effect for structural timber is of the same order of magnitude as for clear wood. There is also an effect of moisture content and temperature on the DOL curve. Higher moisture content and varying moisture gives larger DOL-effect, the same has been found for increasing temperatures.

To incorporate the effect of DOL in the design all loads are normally classified into classes depending on their duration. Depending on the load duration class of the specified load, the strength is reduced. The reduction is larger for long term loads than for short-term loads. In Eurocode 5 the effect of duration of load and moisture content is combined into one modification factor k_{mod} , c.f. section 1.5.1.

2.4.5 Influence of temperature

Both the strength and stiffness of wood is decreasing with increasing temperature. The effect is, however, relatively small for normal temperatures (−30 °C to +90 °C). At temperatures above 95 °C (or 65 °C for long-term loads) there is a thermal degradation of the wood material. At higher temperatures the cellulose chains are shortened and the structure of the hemicellulose is changing. The effect of temperature is normally not taken into account in design codes.

2.4.6 Influence of size

Strength testing of timber has shown that there is a considerable influence of the volume of the tested specimens. Larger specimens break at a lower average tension stress level than smaller specimens. This phenomenon is normally explained by the weakest link theory. This theory says that “a chain subjected to tension is never stronger than its weakest link”. For wood it is shown that the probability of a large weakness occurring in the most loaded section is larger for a large specimen than for a small specimen. Wood is a brittle material and the Weibull theory has been used to explain the behaviour. The Weibull theory assumes that the material is brittle and that the defects are of random size and randomly distributed within the specimen. If V_1 and V_2 are the volumes of two wood specimens respectively and f_1 and f_2 are their respective strengths, Equation 2.5 can be used to describe the relationship between these parameters. The factor k is the shape parameter of the Weibull distribution:

$$\left(\frac{f_2}{f_1}\right) = \left(\frac{V_1}{V_2}\right)^{1/k} \quad \text{Eq. 2.5}$$

The volume is here the product of the width, depth and length, that is b , h , and ℓ of the specimens why the equation may be rewritten as Equation 2.6 with exponents associated with each dimension:

$$\left(\frac{f_2}{f_1}\right) = \left(\frac{b_1}{b_2}\right)^{1/k_b} \cdot \left(\frac{h_1}{h_2}\right)^{1/k_h} \cdot \left(\frac{\ell_1}{\ell_2}\right)^{1/k_\ell} \quad \text{Eq. 2.6}$$

In practice the width of a timber beam has a relatively small variation and the effect of the width is generally ignored. The strength values for bending strength as well as for tension strength are tested in a standard configuration with a certain ratio between depth and length. This makes it possible to reduce Equation 2.6 to only the depth but with a combined factor for the shape function taking both depth and length into account.

The size (height) effect is often taken into account in design codes by having different strength values for different sizes of the loaded element. Eurocode 5 includes a modification factor k_h that increases the bending and tension strength for small dimensions, for sawn timber for a height less than 150 mm in bending and a width less than 150 mm in tension.

The volume effect is also very large in tension perpendicular to the grain. Eurocode 5 therefore includes a modification factor k_{vol} in areas with high tension stresses perpendicular to the grain as for example in curved elements or in the apex zone of double tapered beams, see Chapter 3.

2.4.7 Long-term deformations

The loading time is affecting not only the strength but also the deformation of wood. A piece of timber subjected to a constant load will show increasing deformation with time, this effect is called creep, see Figure 2.23. When the load is removed most of the deformation will be recovered, although there is almost always a small permanent deformation remaining.

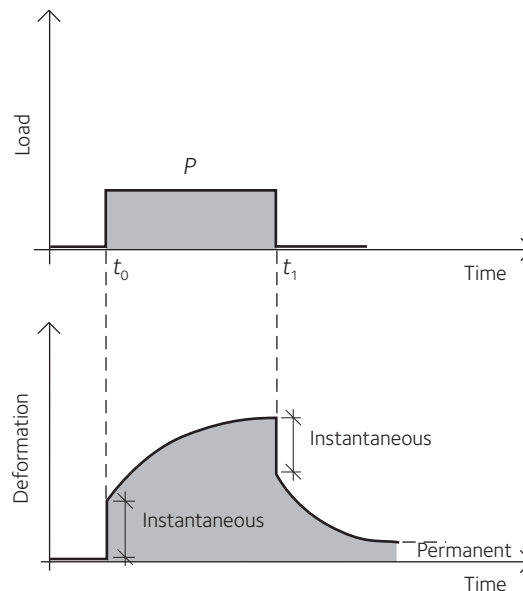


Figure 2.23: Creep curve, above the load-time curve and below the deformation load-time curve.

The creep curve in Figure 2.23 shows the deformation for a relatively low load/stress level. For low load levels the creep curve is convex. Higher load/stress levels can lead to a rapidly increasing deformation and lead to creep rupture, see Section 2.4.4.

The deformation can be divided into three components; elastic, delayed elastic and viscous deformation. The elastic deformation appears directly after loading. After that the deformation will increase slowly under a constant load. This increase in deformation is made up of both the delayed elastic and the viscous deformation. The difference between these two components is visible first after unloading. The delayed elastic deformation is reversible with time while the viscous deformation is permanent.

There are several external factors that influence the magnitude of the creep; temperature, load direction, stiffness, knots and moisture content:

- Load direction – the amount of creep is different in different load directions.
- Stiffness – tests have shown that material with higher modulus of elasticity not only get less instantaneous deformation; the additional creep deformation is also smaller for material with a higher modulus of elasticity.
- Knots – timber beams with more knots (and lower MOE) exhibit larger creep deformation under load.
- Moisture content – tests have shown that wood with higher moisture content exhibit larger creep deformations than wood with lower moisture content. Varying moisture content in combination with loading have, however, been shown to give a significant increase of creep deformations.

- Temperature – increased temperature leads to increased creep deformations. This is especially noticeable at temperatures over 100 °C when the lignin is softening.

Increasing deformations for timber under varying climate conditions are known as the mechano-sorptive effect. This effect was first described by researchers from Australia during the 1950's. This effect is illustrated in Figure 2.24 where the results from a study of creep in small beech specimens loaded in bending are shown.

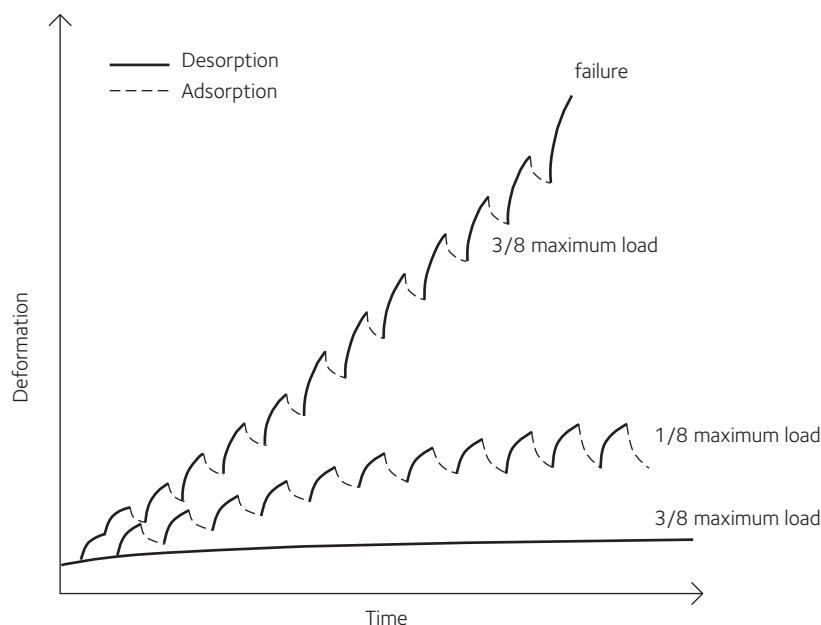


Figure 2.24: The deflection of small beech specimens loaded in bending to 1/8 and 3/8's of the short term strength in constant and varying climatic conditions, Hearmon and Paton (1964).

The influence of moisture content and load duration is important when designing a timber structure. Engineered wood products, produced of wood and some kind of adhesive, may have different creep properties than sawn timber. In these types of material the creep behaviour may be governed by the properties of the adhesive. The creep is larger for some engineered wood products than for sawn timber.

Creep is often taken into account in design, it is for example important in the analysis of stresses in composite structures but maybe most important in the calculation of long-term deformations in the serviceability limit state. The increase in deformation due to creep (u_{creep}) after an infinite time is often defined as a creep factor, denoted (k_{def}) in Eurocode 5, times the instantaneous deformation (u_{inst}), see Equation 2.7:

$$u_{\text{creep}} = k_{\text{def}} u_{\text{inst}} \quad \text{Eq. 2.7}$$

With this definition it is possible to calculate the final deformation, see Equation 2.8:

$$u_{\text{fin}} = u_{\text{inst}} + u_{\text{creep}} = u_{\text{inst}} (1 + k_{\text{def}}) \quad \text{Eq. 2.8}$$

With the help of the creep coefficient it is possible to estimate how large the deformation of a structure will be after a long time with a constant load. For normal sawn timber the creep factor k_{def} is 0,6 – 0,8 for normally dry conditions and as large as 2,0 for timber subjected to high and varying moisture content.

2.5 Strength grading

The strength, stiffness and density of wood vary to a great extent. For Norway spruce the variation in bending strength can be between 10 and 90 MPa. To be able to use this material in load-bearing structures it is necessary to have a better knowledge of the properties. The wood material is produced by nature and it is therefore not possible to control the variation in properties by changing the manufacturing process. For wood it is instead necessary to get an estimate of the properties and grade the material into different strength classes.

The characteristic strength value for all materials is normally defined as the 5 %-fractile in the distribution of strength, see Figure 2.25.

By grading the material into different strength classes it is possible to:

- improve the control of timber characteristics such as strength and stiffness
- have a common classification within a market
- optimise the yield from the raw material
- optimise the use (good enough quality).

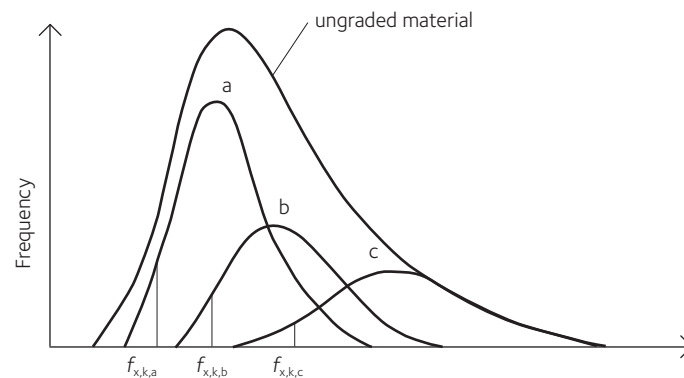


Figure 2.25: Principle of the variation in strength for timber and the material split into three strength classes.

According to most standards the material is graded based on its bending strength. The mean modulus of elasticity in bending and density also have to be checked so that it is within the limits of the grade. All other parameters are estimated based on these values.

There are currently two types of grading: visual strength grading and machine strength grading.

Visual strength grading

The visual strength grading technique relies on the assumed correlation between visually detected defects and strength. In principle there are grading rules establishing which type and size of defects can be allowed in each strength class. The first visual strength grading rules were established in the US in the 1920's and visual grading rules were also established in several European countries during the 1930's. The grading rules are of course different for different wood species, number of grades and so on. Traditionally visual strength grading has been conducted by human inspectors looking at each piece of timber as it passes them on a conveyor. This of course gives problem in accuracy since the grader only has a couple of seconds to assign the board to a certain grade. It also has the natural drawback of only seeing the visual defects, defects within the material is not possible to detect. The method only allows for the use of simple combinations of defects. Today, there are scanning techniques possible to use together with the visual strength grading rules that improve the technique.

Machine strength grading

The machine strength grading technique relies on running the pieces of timber through a machine that measures one or several parameters non-destructively. These parameters are then used to predict the strength and possibly stiffness. The idea to use non-destructively measurements as a basis for prediction of strength and stiffness was presented in the 1950's in both USA and Australia. The reason was a wish to improve the accuracy of the grading. The most commonly used parameters to measure non-destructively are modulus of elasticity and density.

2.5.1 Relationship between strength, stiffness and other parameters

The strength grading principles are built on measuring one or more parameters non-destructively and use this measure to predict the strength (and possibly stiffness and density). This means that to use the grading principles it is necessary to have a good knowledge about the relationship between these parameters and strength and the influence of different kind of natural characteristics on these relationships. For clear wood the following relationships have been found between strength, density and modulus of elasticity (MOE):

- Bending strength – modulus of elasticity $R^2 = 0,70 - 0,75$
- Bending strength – density $R^2 = 0,60 - 0,65$
- Modulus of elasticity – density $R^2 = 0,60 - 0,65$

The properties in timber, however, vary to a large degree between different trees, logs and even within a tree or log. The natural characteristics of wood also influence the parameters and their relationships, and it is not only the size of the defects but also their location that has an influence. In visual grading the occurrence of knots is the single most important parameter for assigning boards into different strength classes. Knots have also shown to be the most important factor in true strength tests. In strength tests of Norway spruce it has been shown that approximately 90 percent of the failures start at the position of a knot. Spiral grain angle is the cause of failure in some cases. Compression wood has a pronounced effect on the relationships between these parameters, it has high density and low MOE but the strength is only marginally decreased.

There are a number of different non-destructively measurable parameters that can be used in strength grading. Numerous tests have been done to establish the relationship between different parameters and the bending or tension strength of sawn timber. Studying a few of these test series gives the following coefficient of determination between bending strength and other parameters, see Table 2.3.

Table 2.3 Coefficient of determination between bending strength and a non-destructively measured parameter based on a number of studies on sawn timber (Johansson 2003)

Measured parameter	R^2
Knots	0,16 – 0,27
Annual ring width	0,20 – 0,44
Density	0,16 – 0,40
MOE, bending or tension	0,53 – 0,72
Knots and density	0,38
Knots and MOE	0,58 – 0,73

The best predictor for strength is the stiffness or modulus of elasticity. The coefficient of determination varies between 0,5 and 0,7 between different studies; the difference is mainly due to different measurement methods for the MOE. A typical result from a test of bending strength and stiffness can be seen in Figure 2.26.

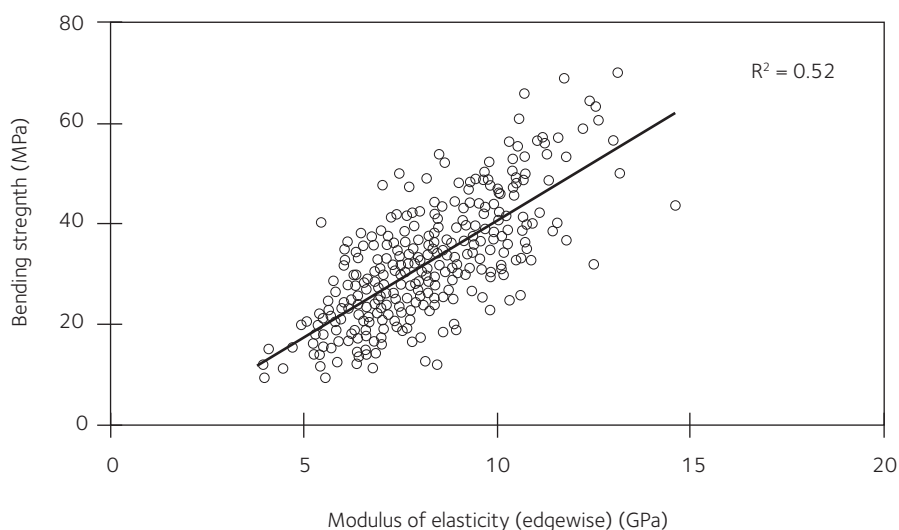


Figure 2.26: The relationship between the modulus of elasticity (edgewise) and bending strength for 380 studs of the dimension 45 x 70 mm.

The difference can also be attributed to different knot sizes in the material in different studies. The modulus of elasticity includes information about knots (lower MOE at the position of the knot), spiral grain angle (lower MOE) and compression wood (lower MOE) which is why it works better than the other parameters. It is of course also possible to combine different parameters when grading timber. Adding information about knots together with information about the average density strengthens the prediction of strength. Adding more information to the MOE does not strengthen the prediction to any higher degree, probably because the MOE already includes this information.

2.5.2 Machine strength grading principles

There are many techniques used in strength grading machines; static flat-wise bending, MOE determined from vibration or measurement of density by radiation are the most commonly used techniques.

2.5.2.1 Static flat-wise bending

Flat-wise bending of the timber is the principle that has been used the longest and was the dominating technique among the grading machines up until the early 21st century. The technique is based on measuring the flat-wise MOE in three-point bending. The boards are continuously feed flat-wise through a machine with three rollers and the MOE is evaluated based on either 1) measurement of deformation for a constant load or 2) measurement of the load necessary to create a pre-set deformation, see Figure 2.27. This gives a measure of the variation of the MOE along the board. With a known MOE it is possible to estimate the strength of the board and assign it into a specific strength classes.

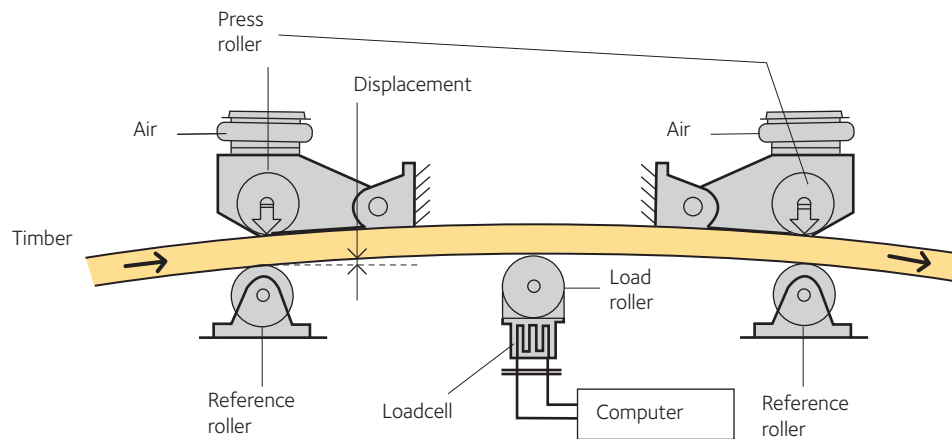


Figure 2.27: Principle of a static bending machine.

2.5.2.2 MOE determined from vibration

The dynamic modulus of elasticity of a board can be determined by introducing vibrations into it and evaluate its resonance frequency. The principle is built on theory for vibration of prismatic beams and the equation for determining the MOE from the first resonance frequency for free-free support conditions in the axial direction can be expressed as:

$$E_{A1} = 4 f_{A1}^2 \ell^2 \rho \quad \text{Eq. 2.9}$$

where E_{A1} is the modulus of elasticity, ρ is the density at the measurement time, f_{A1} is the first resonance frequency in the axial direction and ℓ is the beam length. For other support conditions than the free-free and for higher order frequencies the equation has to be modified. It is also possible to use transverse vibrations. For clear wood specimens the dynamic MOE correlates well with the true MOE, the same has been seen in several studies also for timber. The method, however, has the drawback of measuring an “average” MOE of the board while the static bending machines have, at least in theory, the capacity to localise the position with the minimum MOE along the board. Several studies have shown that for material with limited knot size the techniques works as well as or better than the technique using static bending.

2.5.2.3 Other measurement techniques

With x-ray, γ -rays or microwaves it is possible to measure the density variation within a piece of timber. Several of the natural characteristics of wood imply a density that is different from normal wood. Knots and compression wood show for example a higher density and juvenile wood has a lower density. Using enough accuracy in the measurement procedure it is possible to characterise, for example the size, position and shape of the knots within a piece of timber. With known knot structure and threshold values for knot areas it is possible to create a machine that can grade timber into different strength classes.

There are also other types of techniques that can be used. One is based on the wave velocity; this technique is based on measuring the time it takes for a pulse, generated by a hammer, or ultrasonic sound, to pass longitudinally through a board. The speed of the sound wave through the material can be used as a basis to calculate the average MOE of the board. It is of course also possible to combine different types of techniques; for example combine a technique for

measuring MOE with a radiation technique to find the size and location of knots. There are a number of grading machines on the market; most of them are built on vibration techniques but almost as many are built on flat-wise bending and x-ray. Some of these machines also have the capability to use two of the techniques in combination.

2.6 Modified wood

It is possible to change the properties of wood by modifying the cell wall polymers and the matrix they are in. Modification of wood has been the subject of years of research mainly focussed on increasing dimensional stability and biological resistance. These modifications have been studied to replace toxic wood preservation chemicals such as copper, chrome and arsenic and creosote.

Over the past 15 years, a number of alternatives, more environmentally friendly wood modifications have been reinvestigated, patented and commercialized. There are three basic technologies, some with variations: acetylation, furfurylation and heat treatments. These modifications have in common that they all increase dimensional stability, reduce wood moisture content and reduce attack by wood destroying fungi, see Hon and Shiraishi (1991) and Hill (2006).

2.6.1 Processes

Acetylation is a reaction between acetic anhydride and hydroxyl groups that are located mainly on the hemicellulose and lignin polymers located in the cell wall. The bonded acetyl bulks the cell wall almost back to the green dimensions of the original wood. The reaction is carried out in a vacuum/pressure cylinder after the wood is dried to below 10 % moisture.

Furfurylation is a method where the wood is pressure impregnated with monomeric furfuryl alcohol. The key reaction is a polymerization of the alcohol bulking the cell wall with or without crosslinking. The process leads to a permanent swelling of the wood. Some of the added chemical can be located in the lumens.

Heat treatments are done by heating the wood to high temperatures (190 °C to 220 °C) in the presence or absence of oxygen and sometimes heated in oil. The high temperature leads to a loss of mainly hemicellulose polymers resulting in a loss of mechanical properties. The degradation is more severe if the process is carried out in oxygen.

2.6.2 Properties of modified wood

Acetylation results in a slight darkening of light woods and a mild bleaching of dark woods. The best results are seen with an acetyl weight gain of 18 percent to 20 percent. Dimensional stability is greatly improved (80 percent +), decay resistance against brown-, white- and soft-rot fungi is greatly increased (less than 2 percent in a standard soil block test) and the equilibrium moisture content is greatly reduced (reduced from 21 % to 5 %). The mechanical properties (modulus of rupture and elasticity) are unchanged over controls in dry tests but wet strength and wet stiffness of acetylated wood are much higher than controls. Acetylated wood is used in doors, windows, decking, outdoor furniture and claddings.

Furfurylation changes the color of the wood depending on the level of treatment. Wood treated to a weight gain of 30 percent is dark red to brown and wood treated to 100 percent is

dark brown. Mechanical properties are improved, decay resistance is increased and the equilibrium moisture content is reduced 30 to 60 % compared to untreated wood. Furfurylated wood is often used in decking and cladding but can also be seen in outdoor furniture.

Heat treatments result in major losses in mechanical properties depending on the temperature, time and method of treatment. Mechanical properties can be reduced more than 50 percent. Heat treated wood is generally darker and can smell somewhat burned. There is some increase in decay resistance, the equilibrium moisture content is reduced (40 % to 50 %) and the wood is more brittle as compared to untreated wood. Heat-treated wood is used in non-structural applications such as decking and cladding.

2.7 Engineered wood products

Normal sawn timber can be found only up to certain dimensions, due to the size of the trees and the industrial process. In Sweden the maximum size of sawn timber is a depth of 245 mm and a length of less than 5,5 meters. For larger dimensions it is necessary to use some kind of Engineered Wood Product or EWP for short. EWPs are made of wood in the form of sawn timber boards, veneers, particles or fibres held together with, in most cases, some type of adhesive.

During the 1900's a large number of different EWPs both in the form of beams and in the forms of panels have been invented. Figure 2.28 shows a timeline for the invention of different types of EWPs.

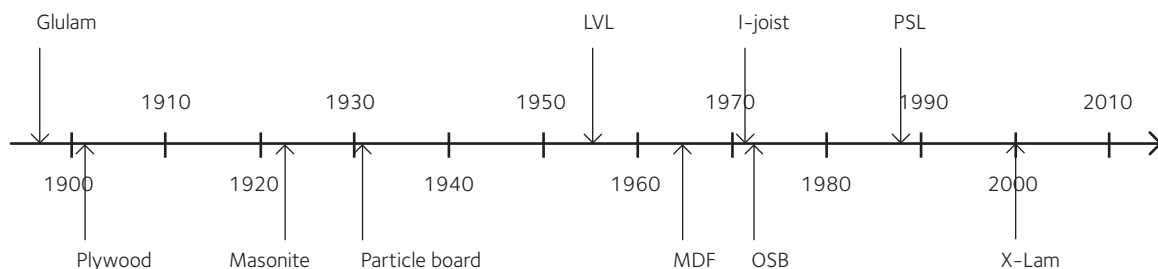


Figure 2.28: Approximate year of development of different types of EWPs, beam type products above the time line and panel type products below the time line. LVL – laminated veneer lumber, MDF – medium density fibreboard, OSB – oriented strand board, PSL – parallel strand lumber, X-Lam – cross-laminated timber.

The majority of these products were invented in North America where lack of material from old growth timber has led to an increased use of material from smaller diameter trees, lower quality logs and new tree species. In a controlled production process it is possible to develop new value added products with good structural properties.

2.7.1 Engineered wood products based on sawn timber boards

EWPs based on gluing together pieces of sawn timber can be produced in the form of beams – glulam beams, or in the form of thick panels, often glued with the sawn timber boards oriented in different directions in different layers, referred to as cross-laminated timber – X-Lam.

2.7.1.1 Glued laminated timber – glulam

The oldest of the engineered wood products is glued laminated timber. The first patent for a glue lamination process for sawn boards was awarded to Otto Hetzer in 1906. Glulam consists of several (at least four) laminations (finger jointed sawn boards) bonded together with adhesives. The boards are all oriented with their main fibre direction in the axial direction of the finished glulam beam.

Normal straight glulam in Sweden consists of 45 mm thick laminations with a width of up to 215 mm. Wider beams can be produced by gluing two or more glulam beams together. Glulam can either be homogeneous or made to match the design stress levels in the beam with higher strength laminations in the outer higher stressed regions of the beam. This type of glulam is called combined glulam, see Figure 2.29.

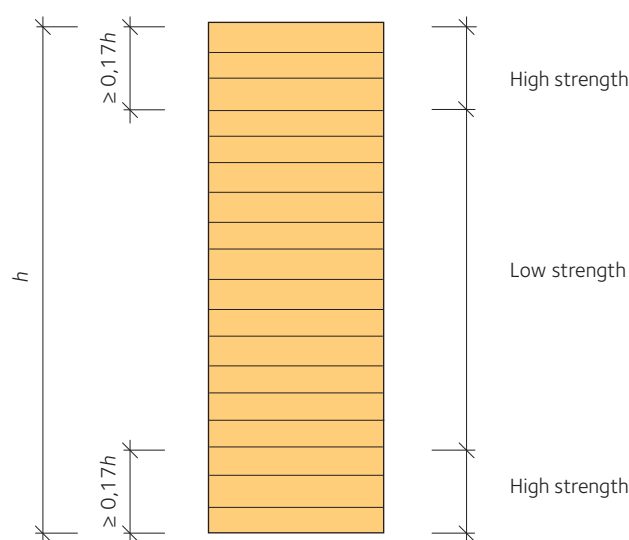


Figure 2.29: The lay-up for a combined glulam beam.

Straight glulam is produced in a highly mechanised process. The normal process is to start with the sawing and drying of the laminations. For the gluing process the laminations are dried to a moisture content between 6 and 15 %. All laminations, have to have less than 5 % difference in moisture content, are thereafter strength graded. The laminations are finger jointed together to create the correct length of the laminations. The laminations are planed and the adhesive is thereafter spread on the flat surfaces. In Sweden, the adhesive is commonly a Melamine-Urea-Formaldehyde (MUF) adhesive, in some special applications Phenol-Resorcinol-Formaldehyde (PRF) adhesive are still used. The boards are then laid up in a press where the adhesive cures under a certain pressure and in a specified temperature. The beams are finally planed and possibly a layer of wax to protect it from moisture is applied. This mechanised production process also includes quality controls in terms of regular checks of the strength of the finger-joints and delamination tests of the glue line.

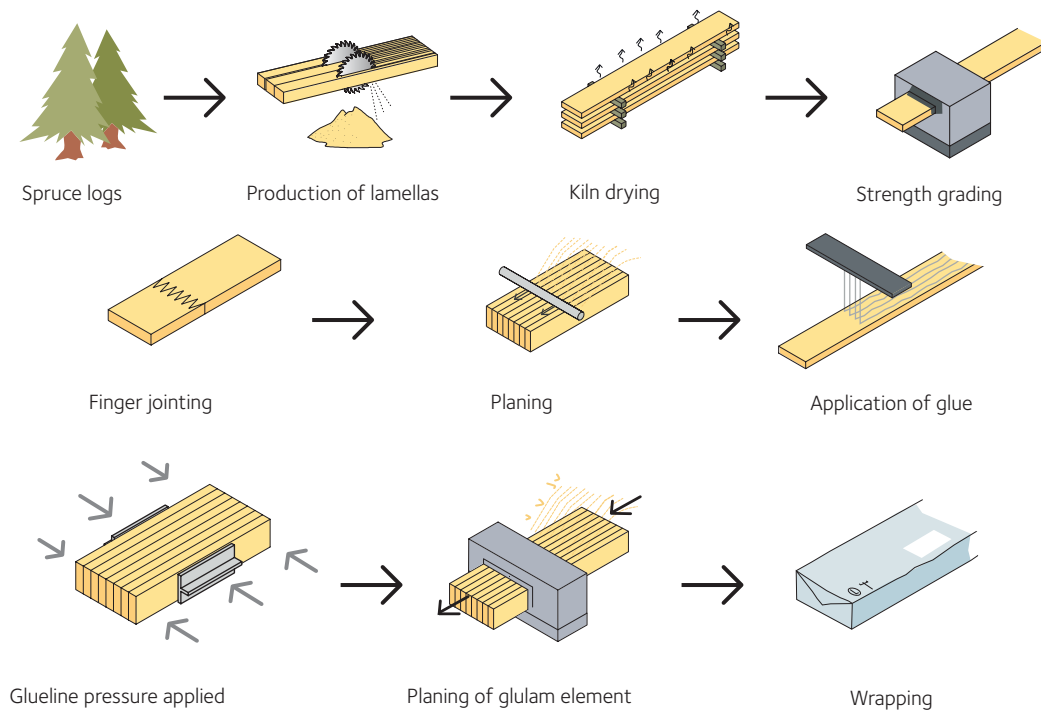


Figure 2.30: The manufacturing process for straight glulam beams.

The beams can also be manufactured with a curvature, either a small pre-camber to minimize deflection or as a curved beam. The curved beams are normally made with thinner laminations and forced into the desired curved shape before curing of the adhesive.

Tests have shown that in average glued laminated timber is not significantly stronger than solid beams of the same size but the variability in strength is lower, see Figure 2.31. The low variability has often been explained by smearing-out effects, i.e. cutting the solid wood into smaller pieces and glue them together randomly causes the low-strength defects to be more uniformly distributed and each defect has a less significant effect.

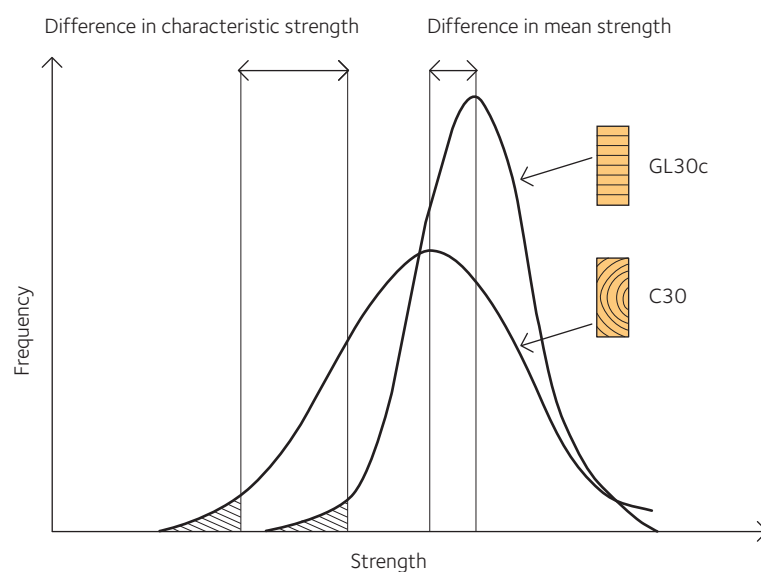


Figure 2.31: Distribution function for the strength of glulam beams and structural timber.

The most common failure for a glulam beam subjected to bending is a tensile failure parallel to the grain of the outer lamination, often originating in a knot or a finger-joint. Another type of failure that can occur is shear failure. Cracks can occur along the grain due to drying effects and some researchers state that this will reduce the shear capacity of the beams, but other researchers do not agree with this conclusion. High tensile stresses perpendicular to the grain need to be taken into account, especially in curved beams and beams with holes and notches.

2.7.1.2 Cross-laminated timber – X-Lam

Cross-laminated timber (X-Lam) is a product that has won significant market shares in the timber industry in the 2000's. Cross-laminated timber is made of sawn timber glued together in a layered structure with the boards in each layer placed perpendicular to the layer above and below; see Figure 2.32. The exact lay-up of the cross-laminated boards is dependent on the manufacturer. Cross-laminated timber can be manufactured as a 3, 5, 7 or even more layer product with varying thickness of the different layers. The use of an uneven number of layers implies that the outer layers always are oriented in the same direction, and better strength is obtained.

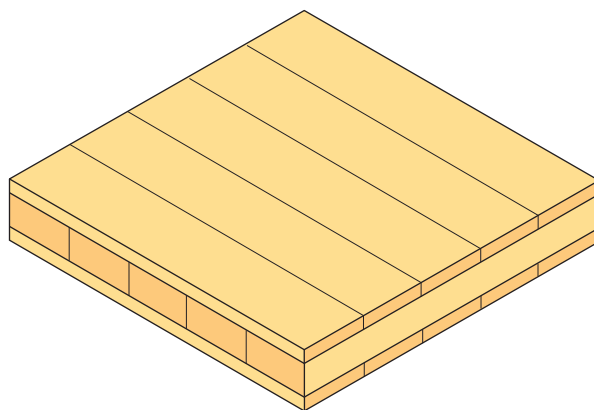


Figure 2.32: Cross-laminated timber panel with three layers.

The size of X-Lam panels depends on the possibilities of the manufacturer but generally they are at least 60 mm thick and can be as large as 500 mm thick, 3000 mm wide and with a length of up to 24 m. The strength parameters for X-Lam are so far mainly set for each manufacturer after testing of the material by an independent third party. X-Lam panels can be used as load-bearing vertical walls and as horizontal floor diaphragms. X-Lam panels often arrive to the building site ready for assembly with for example holes for doors and windows as well as grooves for electrical fittings already prepared. The customization of the elements is often done already at the factory with the help of advanced CNC-machines.

There are also cross-laminated timber boards manufactured using nails or screws instead of adhesive. The properties of this type of panels vary somewhat from glued panels.

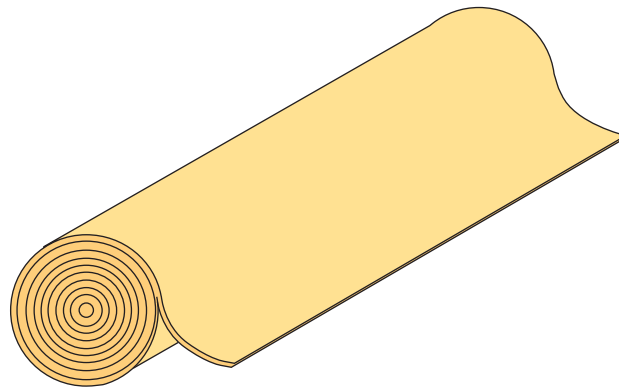


Figure 2.33: Veneer production from logs.

2.7.2 Engineered wood products based on veneers

Another way of utilizing the raw material is to produce veneers from the logs. Veneers are thin (2 – 4 mm) layers of wood. The logs are debarked and steamed in hot water before being rotary peeled; see Figure 2.33. These thin veneers are then dried to the target moisture content 12 – 15 % and often also strength graded before being used in EWPs.

The veneers are then glued together using adhesives into different formats of structural elements. The two most commonly used products are laminated veneer lumber (LVL) and plywood.

2.7.2.1 Laminated veneer lumber – LVL

Laminated veneer lumber is made by gluing the wood veneer sheets together to form thick (20 – 90 mm) structural panels with maximum size 3 000 × 24 000 mm. For normal LVL all the layers are oriented with the fibre direction in the same direction, normally in the long direction of the finished product, see Figure 2.34. After gluing and curing the thick panels are sawn into board dimensions. The process of gluing the sheets together creates a structural element of higher reliability and lower variability through defect elimination and distribution of defects, in the same way as for glulam.

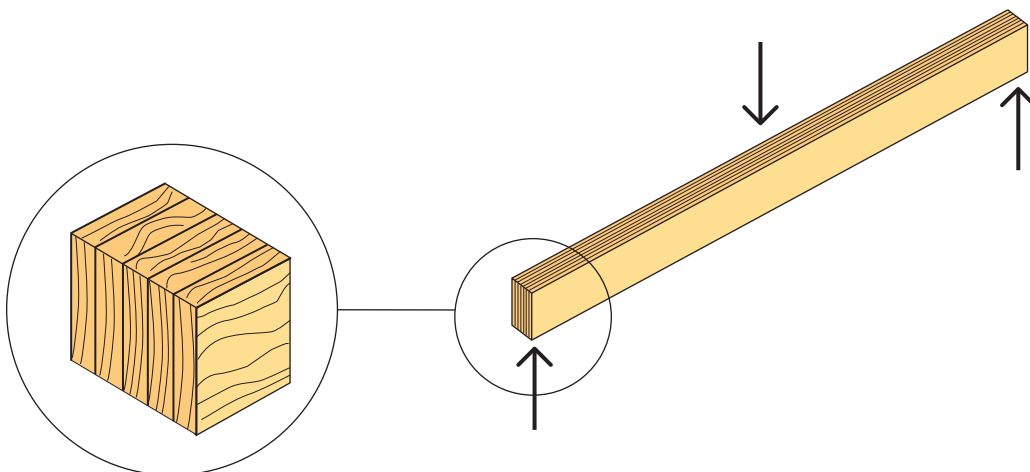


Figure 2.34: Laminated-veneer lumber – LVL, all veneers with the same fibre orientation.

In general LVL has high bending, tension and compression strength, as well as high shear strength and a relatively high modulus of elasticity. There is a considerable size effect in LVL that has to be taken into account when calculating design values in bending for LVL. Some grades of LVL are produced with some of the layers with the grain direction perpendicular to the fibre direction of the majority of the layers. This production method allows for a production of structural panels with a higher stiffness across the panel which can be beneficial in some structural components, such as floor structures.

2.7.2.2 Plywood

Plywood was one of the first EWPs to be produced. It is produced in much the same manner as LVL, but with the veneers laid up perpendicular to each other. The number of veneers (plies) are always odd, meaning that the fibre direction of the outer layers are always in the same direction; see Figure 2.35. Structural plywood is most commonly produced in sheets of the size 1 200 × 2 400 mm or 1 220 × 2 440 mm with a thickness of 12 – 24 mm. The outer veneers normally have the fibre direction oriented in the long direction of the panel.

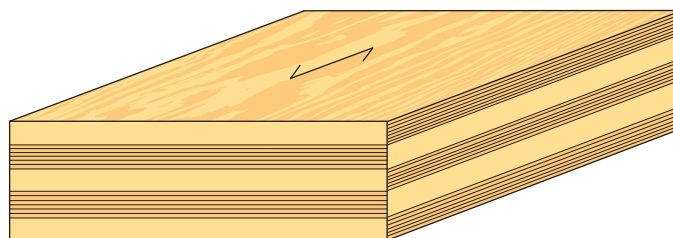


Figure 2.35: Five-layer plywood cross-section.

The structural properties of plywood are dependent on the number and thickness of the veneers and load direction. It is important to keep in mind the direction of the forces, bending perpendicular to the plane of the panel and in-plane bending, see Figure 2.36. The layers with grain direction in the same direction as the normal stresses have the highest stiffness and take the major part of the load. The contribution to the load bearing of the layers with grain direction perpendicular to the direction of the normal stresses is very small and can in practice be disregarded.

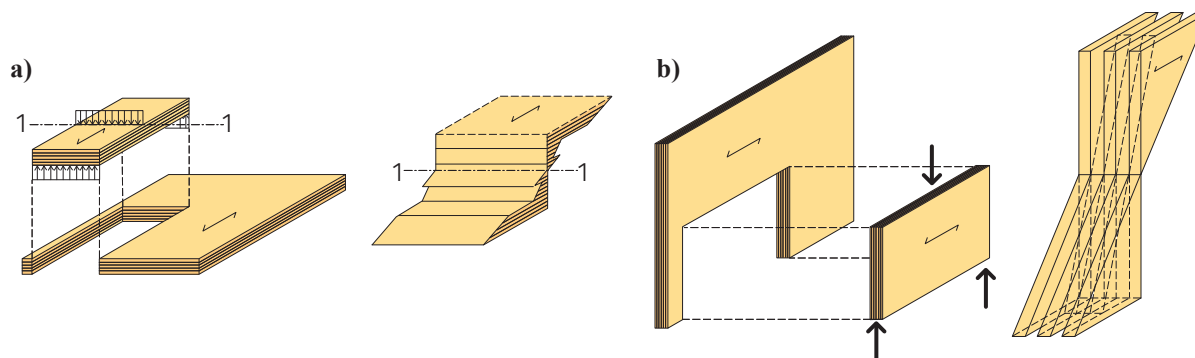


Figure 2.36: Bending a) perpendicular to the plane and b) in-plane bending including distribution of normal stresses.

2.7.3 Engineered wood products based on strands, chips or fibres

EWPs can also be manufactured based on smaller wood elements such as strands, chips or fibres. These small elements are glued together in industrialised processes to form panels of different formats. Due to the processing technique, these types of products have in general a high density in the surface layers and a slightly lower density in the middle. The strength of the product is also to a large extent governed by the production technology, but also to a large extent by the type and amount of adhesive used in the product.

2.7.3.1 Oriented strand board – OSB

The most commonly used panel for structural use is oriented strand board – OSB. OSB is manufactured from thin wood strands (size $0,8 \times 13 \times 100$ mm) sliced from mostly small-diameter logs. These strands are bonded together using adhesive in a ratio approximately 95 percent wood and 5 percent adhesive under heat and pressure. The strands are in the process oriented to form the best possible product, this means that in the outer layers the strands are oriented parallel to the long direction of the panel and in the inner layer with the strands randomly oriented.

The most commonly used OSB product is in the form of panels with a size of $1\,200 \times 2\,400$ mm with a thickness between 6 and 25 mm used as sheathing material in walls or in floor structures. OSB can also be manufactured in larger panels with width up to 3 m, length up to 25 m and thickness up to 75 mm. These panels can be used as structural elements in the same manner as X-Lam panels.

2.7.3.2 Chip, particle or fibre board

Chip, particle and fibre boards are all produced of wood chips of different sizes mixed with adhesives and hot pressed to panel form. Chipboards are made of wood chips slightly smaller than the strands in OSB, and are normally produced with 85 percent wood chips and 15 percent adhesive. Chipboards are uncommon in Sweden. Particle boards are made from smaller wood particles; saw-dust combined with adhesives. Particle boards are today mainly used in floor structures but are often replaced with plywood or OSB.

Fibre boards can be produced using wood fibres and adhesive in a process similar to the production of particle board or in a wet-process using the lignin within the wood material as a binder. The later process was developed by William H. Mason. The most commonly used name for this type of product is masonite. Masonite is used as a structural element while other types of fibre boards are more commonly used in joinery and furniture.

2.7.4 Built up structures – I-beams

Another type of EWP is I-beams, see also Chapter 5. These beams are made up with flanges of sawn timber or LVL and with webs of some kind of board material, plywood, OSB or masonite, see Figure 2.37. These beams are produced in automated production processes under control of third party organisations. The webs are glued to the flanges either in grooves in the flange or between two pieces of the flange.

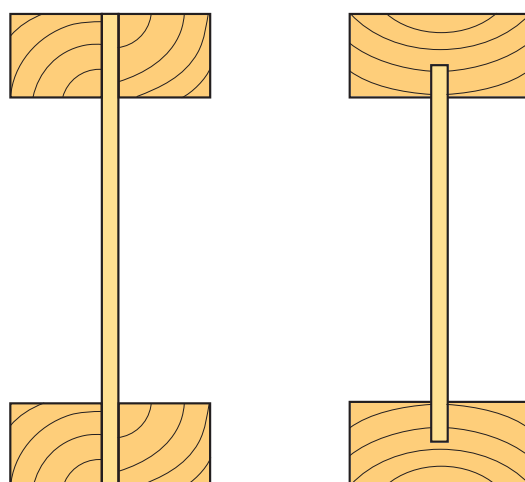


Figure 2.37: I-beams with flanges of solid timber and a web of a board material.

I-beams can be seen as built-up structures and designed as such. The quality control for the manufacturing process is, however, very difficult to fulfil without an industrialised production process. This has led to that most I-beams are produced in factories and that characteristic strength values are given by the manufacturer.

2.8 End-user requirement on sawn timber

Different end use of the timber product means different requirements on the sawn timber. A building consists of many parts to build a whole structure; these parts are subjected to different requirements in terms of geometrical tolerances and different types of loading for example. This section presents some different types of commonly used structures and their requirements in terms of geometry, visual appearance and mechanical properties.

A floor structure is a load-bearing horizontal structure with the main purpose to divide to different storeys and to carry loads from furniture and people transversally. A floor structure is often produced with wooden beams (or EWP beams) with a spacing of 300 mm to 600 mm with a flooring panel above. Floor structures have to fulfil three main design criteria; strength in terms of bending and shear, deflection and vibrations. The design of a floor structure according to the most commonly used codes, for example Eurocode 5, shows that the criteria for deflection and vibration are the hardest to meet. These are both dependent on the stiffness (modulus of elasticity) of the material. For sawn timber used in a floor structure a high modulus of elasticity is the most important material property. To meet the requirements for a plane floor the distortions of the sawn timber can not be too large. The most critical distortion mode for a floor structure is spring as this will directly affect the shape of the floor. Twist is also important as it is difficult to reach a good connection between the flooring panel and the wooden beams when the beam is heavily twisted.

Another commonly used structural element is the timber framed wall with sawn timber studs. In this structure the sawn timber is used as vertical studs covered with a sheet material on both sides. The main load on a wall is normal forces from above and a distributed load from the side caused by wind loads. The main design criteria for a wall stud are compression parallel to

grain and bending. There is, however, also a criterion from building physics that requires insulation. The requirement for insulation in the wall makes it necessary to use larger beam dimensions than what is normally necessary for the load bearing purpose. The requirement for strength of a wall stud is therefore normally rather low. The requirement for shape stability is still rather hard. The requirements for a straight wall mean that there are strict requirements for twist of wall studs.

Other products like for example timber for roof trusses have high requirements on tensile strength. There is also a requirement to limit the occurrence of knots in the areas used for the connections in a roof truss. Tensile strength is also important for material to be used as laminations in a glulam beam. There are also geometrical requirements to be able to use the board in the glue lamination process. For sawn timber delivered to a glulam factory there is also a strict criterion for the moisture content.

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Chapter 3

Design of structural timber elements in ultimate limit state

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3 Design of structural timber elements in ULS

3.1 Prismatic beam elements

3.1.1 Bending and shear

Flexural members are those elements in a structure that are subjected to bending, and several types and forms of such members are used in timber construction. Typical examples are solid section rectangular beams, floor joists, and purlins, see also Figure 3.1.

This section deals in detail with the general requirements that are necessary for the design of flexural members made from straight solid timber or wood-based structural products of uniform cross-section in which the grain runs essentially parallel to the member axis.

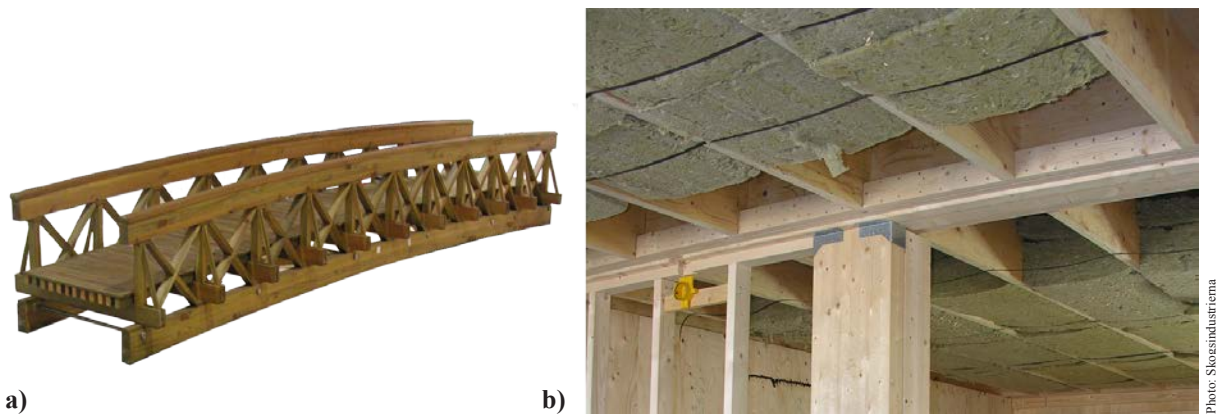


Figure 3.1: Examples of flexural members. a) External timber structure, b) floor joists.

For beams design checks have to be done with respect to moment capacity, shear capacity, deflections and vibrations. In general, bending is the most critical criterion for medium-span beams, deflection and vibration for long-span beams, and shear for heavily loaded short-span beams.

The elastic theory of bending states that when, for example, a solid rectangular member as shown in Figure 3.2 is subjected to a bending moment M about the y - y axis (the strong axis), the design stress at any distance z from this axis will be:

$$\sigma = \frac{M \cdot z}{I_y} \quad \text{Eq. 3.1}$$

where I_y is the second moment of area of the cross-section about the y - y -axis. The term $I_y/0,5h$ is referred to as the section modulus about the strong axis and is denoted by the symbol W_y . This means that the moment capacity of a timber beam can be determined from:

$$M_{Rd} = f_{m,y,d} \cdot W_y \quad \text{Eq. 3.2}$$

where $f_{m,y,d}$ is the design value of bending strength of the material. The index R refers to resistance.

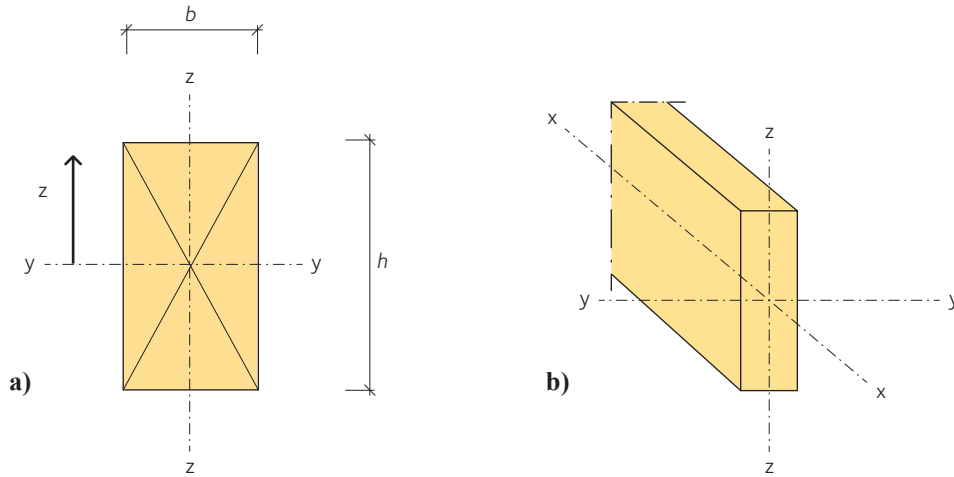


Figure 3.2: a) Cross-section of a rectangular beam. b) Orientation of coordinate system in these compendiums.

Similar expressions may be derived for the design bending capacity about the z - z axis (the weak axis).

Lateral instability may affect a member subjected to bending when the compression zone of the member is not fully restrained against lateral movement and the relative slenderness for bending is high. The design procedure for taking the effect of lateral instability into account is given in Section 3.2.2.

If a member is subjected to bending about both axes and the relative slenderness ratio for bending about the y - y axis is such that lateral instability can be neglected, the design conditions to be met are given by:

$$\frac{M_{y,Ed}}{M_{y,Rd}} + k_m \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1$$

$$k_m \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1$$

Eq. 3.3

where k_m is a modification factor that allows for redistribution of stress in the element. The index E refers to forces and moments determined from existing loads. Values for the factor k_m are given by Eurocode 5: For structural timber, LVL and glued-laminated timber, $k_m = 0,7$ for rectangular sections, and $k_m = 1,0$ for other cross-sections. For other wood-based structural products, $k_m = 1,0$ for any cross-section.

When a beam is subjected to bending, shear stresses will also arise. In accordance with elastic bending theory, shear stresses will be generated parallel to the longitudinal axis of the beam. The value of the shear stress at any level in the cross-section of a beam, as derived from elastic theory, is:

$$\tau = \frac{V \cdot S}{I \cdot b}$$

Eq. 3.4

where τ is the shear stress at the required level, V is the shear force at the position being considered, S is the first moment of the area above the shear stress level about the neutral axis,

I is the second moment of area of the cross-section about the neutral axis, and b is the width of the cross-section at the shear stress level.

At any position along the beam the shear stress at the top and bottom faces of the cross section will be zero and the maximum shear stress will arise at the neutral axis position. For a rectangular section of width b and depth h the maximum shear stress will occur at mid-depth and will be:

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

This means that the shear capacity is given by:

$$V_{Rd} = \frac{2f_{v,d}A}{3} \quad \text{Eq. 3.5}$$

where A is the cross section area and $f_{v,d}$ is the design shear strength.

The shear capacity may be reduced by cracks in the wooden member caused by moisture induced stresses. This is considered in Eurocode 5, which recommends that a reduced effective width b_{ef} shall be used in Equation 3.4 instead of the actual width b .

3.1.2 Axial forces

This section covers the design of straight structural timber or glulam in which the grain runs essentially parallel to the member lengths and the members are subjected to the effects of axial compression or tension.

3.1.2.1 Tension

Although the tensile strength, $f_{t,0,k}$, of clear wood samples is greater than the compression strength, $f_{c,0,k}$, because tension failure occurs in a brittle rather than in a ductile mode and also because of its sensitivity to the effects of grain slope, knots and other defects, the tensile strength of structural timber is generally less than the compression strength. This is particularly the case for the lower strength classes.

Design with respect to tension is normally very simple and the main consideration is the strength value of the wooden member. In some cases it can be relevant to make considerations of the volume subjected to the tensile stresses since it has been shown that the strength is size dependent, see Section 2.4.6.

In some cases the wooden members will be subjected to both bending and axial loading. In the case of combined bending and axial tension the design criteria are quite straightforward, that is failure will not occur when the following relation is valid:

$$\frac{M_{Ed}}{M_{Rd}} + \frac{N_{t,Ed}}{N_{t,Rd}} \leq 1 \quad \text{Eq. 3.6}$$

where $N_{t,Ed}$ is the tensile force due to external loads.

3.1.2.2 Compression

Examples of members subjected to compression forces are given in Figure 3.3 (columns, posts, stud members in walls or struts in trusses). These are members that are subjected to a compressive action acting parallel to the grain and along the centroidal $x-x$ axis of the member.



Figure 3.3: Examples of members subjected to compression forces. a) Columns in a timber frame, b) studs in a multi-storey timber framed building, c) external columns.

When subjected to an axial load, as the slenderness of the member increases there is a tendency for it to displace laterally and to eventually fail by buckling. Since the main focus of members subjected to compression parallel to grain is on buckling stability, compression will be discussed in total in Section 3.2. The same applies for members subjected to compression and bending, it will also be discussed in Section 3.2.

3.1.3 Bearing capacity

In many structural systems based on timber or other wooden products, wood will be subjected to compression perpendicular to grain, see Figure 3.4. Since wood has low stiffness in this direction and exhibits large moisture induced movements, compression will lead to large deformations. When this occurs in a connection between floor and wall structure it may lead to undesired settlements in the buildings. Although the main problem with compressive forces perpendicular to grain usually is the deformation and not actual failure, most codes treat this problem as an ultimate load situation, as a design with regard to failure.

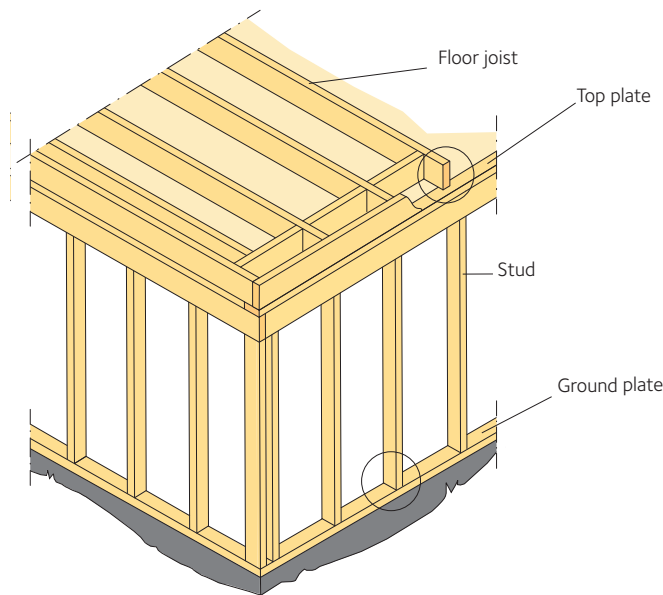


Figure 3.4: Timber frame structure. Positions where significant deformations due to compressive forces may occur are indicated by a circle.

When timber is compressed perpendicular to the grain the wood fibres withstand increased loading as they are squeezed together and as they start to collapse the rate of load increase is reduced, see also Section 2.4.1.4. This behaviour continues until the fibres are fully squashed and if the wood is strained beyond this stage the sustained load will continue to rise. It is hard to define a clear failure in this mode. The deformations may be very large but the wood will still withstand the load. Therefore, the value of the compression strength perpendicular to grain is often given as a limit value defined by a certain value of the strain. However, depending on the position of the bearing area, the bearing length and the member depth, this limit can be exceeded.

Examples of bearing effects (failures) are illustrated in Figure 3.5. The design check in the ultimate limit state can then be given by:

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,d} \quad \text{Eq. 3.7}$$

Here $\sigma_{c,90,d}$ is the design compressive stress perpendicular to the grain and $f_{c,90,d}$ is the design compressive strength perpendicular to the grain, while $k_{c,90}$ is a factor that takes into account how the loading is being applied. The value normally used for $k_{c,90}$ is 1,0 but higher values can be adopted if certain conditions apply, for instance when a stud is situated on a rail, so that only a small part of the rail is subjected to compression forces. Consider for example the case shown in Figure 3.5 d). If the rail rests on the whole bottom surface, the stress will be less at that boundary than at the top boundary, since the loaded area is larger there. It should also be noted that the stressed area on the top also can be considered to be larger than the cross section area of the stud, since the fibres in the rail in the vicinity of the stud increase the resistance against the force. The latter factor can be taken into account either in the factor $k_{c,90}$ or by dividing the force with a larger area than the cross section area of the stud. This would mean that $\sigma_{c,90,d}$ can be determined either by N/bh or by N/A_{ef} where N is the design bearing force, b is the width of the stud and h is the depth of the stud and A_{ef} is the effective contact area, which then is larger than bh .

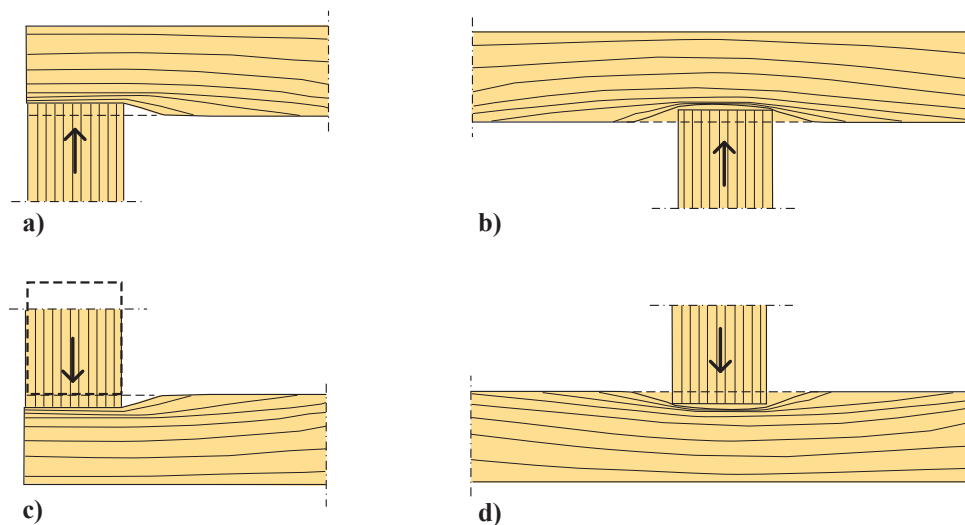


Figure 3.5: Bearing perpendicular to grain.

It is in many cases important to estimate the deformation in the zones subjected to compression since in most cases it is the deformations and not the actual failure that will be of largest interest. This is of larger importance in multi-storey buildings where the deformations will add up from each floor.

In EKS 10 from Boverket concerning compression perpendicular to grain, following advice is to be found:

If verification of compression perpendicular to grain concerns design situations where the consequence is solely increased deformations that have no significant impact on the system's stability and load-bearing capacity, $\gamma_M = 1,0$ and $k_{mod} = 1,0$ can be used when design strength $f_{c,90,d}$ is calculated. Examples of such cases include the indentation of bolts/joints into a ground or top plate of low-rise buildings, as well as support reactions on beams. For cases where indentation of the wood may be deemed to affect the load-bearing capacity (for example local compression in a truss) or where deformations have a significant effect on the function (for example in high-rise buildings), the recommended partial safety factors in Table 2.3 of SS-EN 1995-1-1 should be used. (*EKS 10 (BFS 2015:6)*)

In Limträhandbok Del 2 Section 4.1.4 it is recommended that "this procedure should still be limited to such timber structures where the ratio between self weight and variable loads is relatively small, say $g_k/q_k \leq 0,4$ For timber structures with more self weight ($g_k/q_k > 0,4$) yet the design value for compression perpendicular to grain should be used."

3.1.4 Beams with holes and notches

In structural design of timber structures it is important to avoid tensile stresses perpendicular to grain since the strength in that direction is very low, Gustafsson (1995, 2003). Tensile stresses can be induced in several situations, see also Figure 3.6:

- The geometrical shaping of the structural member can induce tension perpendicular to the grain when the structure is loaded. This can be the case for curved members as well as for beams with holes or notches, see Figure 3.6 a), b) and c).
- Joints of different types can also lead to stresses perpendicular to the grain, see Figure 3.6 d), e) and f).
- Variations in moisture content can lead to undesired stresses within an element, especially if abrupt changes in cross section area occur.

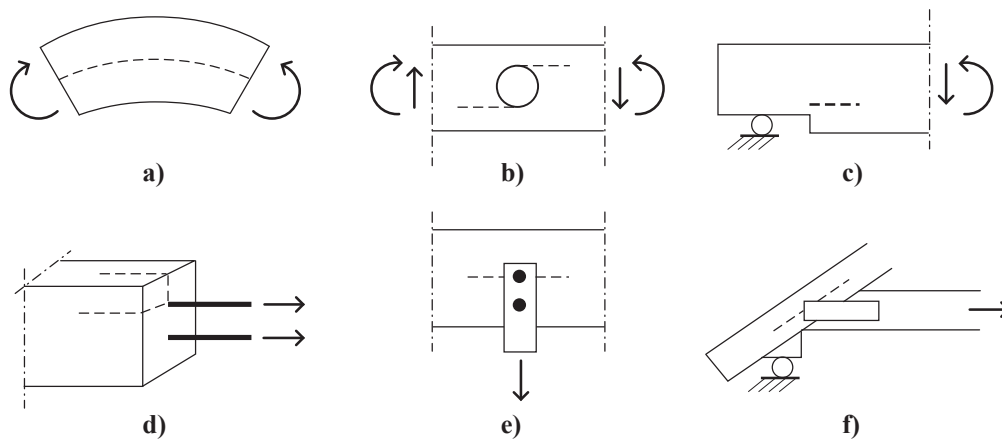


Figure 3.6: Various types and causes of fracture perpendicular to grain.

Here, the problem with holes and notches will be discussed. Both holes and notches may significantly reduce the load bearing capacity of a beam and should preferably be avoided in design. Since tensile forces are induced around a hole or a notch the failure will be brittle and thereby sudden which means that there will be nearly no warning before failure as there can be with other type of failures. In Figure 3.7 the distribution of tensile stresses near the tip of a notch is presented and it is obvious that the stresses can be larger than the tensile strength. The theoretically calculated elastic stress will actually go towards infinity. Shear stresses may also lead to failures in the vicinity of a sudden change in cross section area.

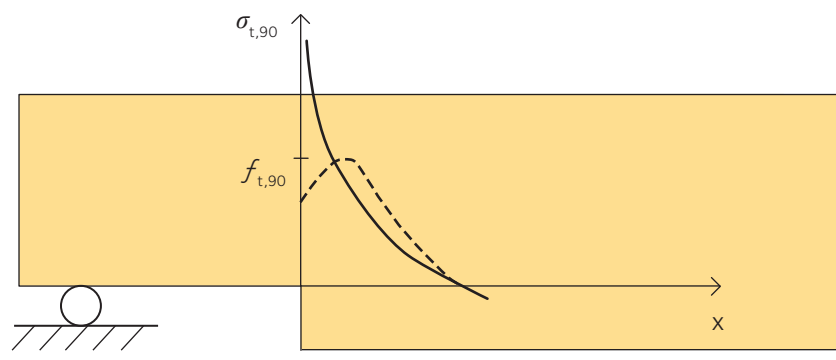


Figure 3.7: Stress at the tip of a notch according to linear theory and as estimated in practice, full line shows theoretical stress and dotted line estimated stress.

As the very high stress is often concentrated in a very small region it is difficult to try to determine the load bearing capacity of a beam with a notch or a hole with a conventional stress criterion. Instead it is necessary to rely on tests or on concepts based on fracture mechanics instead of conventional stress criteria.

Fracture mechanics is a part of the science of the strength of materials. A solid body responds to extreme loading by undergoing large deformation or fracture. Often the main interest for the engineer is the magnitude of the load that causes fracture. In cases when there is no or only minor stress concentrations, for example in the case of a structural member in homogenous tension or bending, the calculation of the fracture load can be carried out by a conventional stress criterion. On the other hand in the case of a very high stress concentration some other approach is needed. Then different analyses can be carried out applying theories of fracture

mechanics. There are a number of different alternative theories but none of them will be described thoroughly here, only the basic ideas for the theory that constitutes the equations given in Eurocode, namely linear fracture mechanics especially analysis of the energy release rate. In this theory the energy release in the fracture process region is the critical value, that is when the energy release has reached a critical value G_c fracture will occur. The critical value can be seen as a material parameter that can be determined by tests. By using G_c the value of the fracture load can be determined, Gustafsson (1995). Other parameters that are necessary to know are geometrical characteristics for the notched beam as well as the modulus of shear stiffness and the modulus of elasticity parallel to grain since those affect the behaviour. In the equations of Eurocode it has been assumed that $\sqrt{(EG_c)}$ is proportional to the shear strength, which has been shown to be the case from tests.

In the design process the shear stress is controlled against the shear strength in the following form:

$$\tau_d = \frac{3V_d}{2b\alpha h} \leq k_v \cdot f_{v,d} \quad \text{Eq. 3.8}$$

where k_v is a reduction factor that depends on the geometrical conditions and material, $f_{v,d}$ is the design value of the shear strength, V_d is the design shear force, b is the beam depth, h is the beam height and α is a non-dimensional parameter that is used to describe the geometry of the notch.

For beams with a hole there is no strength equation given in Eurocode but such relations can be found in other codes. The bases for the codes differ and they don't give the same results, Danielsson (2008). One reasonable approach to deal with this is to avoid holes in timber structures and when they are used to be observant of where they are situated and use design equations with care. There are in general some basic rules concerning for example hole placement, maximum allowed hole size and minimum corner radius for rectangular holes that need to be fulfilled. In Table 3.1 and Figure 3.8 examples of these are presented. It is generally recommended to place holes in the neutral axis of the beam.

Table 3.1 Examples on restrictions for geometry and placement of unreinforced holes, Danielsson (2008).

	Limträhandbok	DIN 1052:2004
l_A	$\geq 0,5h$	$\geq 0,5h$
l_v	$\geq h$	$\geq h$
l_z	$\geq 1,5h$ and ≥ 300 mm	$\geq h$ and ≥ 300 mm
h_{ro}	$\geq 0,35h$	$\geq 0,25h$
h_{ru}	$\geq 0,35h$	$\geq 0,25h$
a	$\leq 0,4h$	$\leq h$
h_d	$\leq 0,15h$	$\leq 0,4h$
r	≥ 25 mm	≥ 15 mm

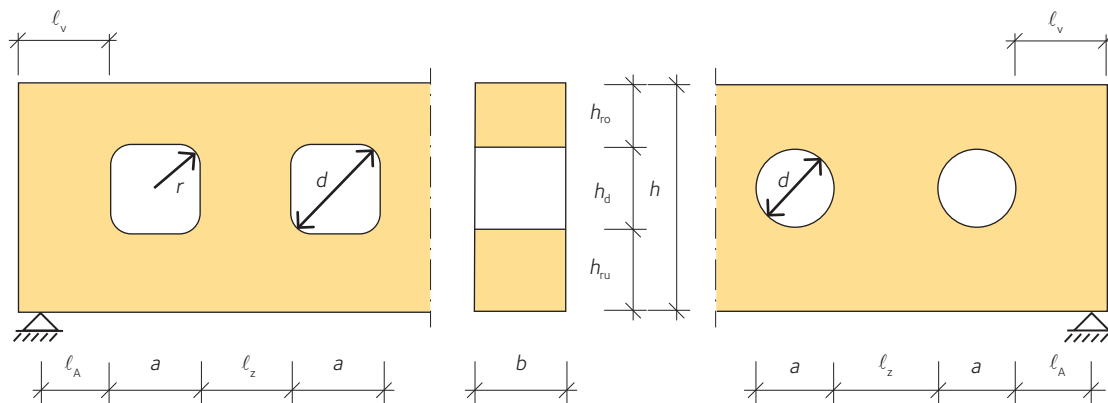


Figure 3.8: Notations for regulations concerning hole geometry and placement.

Since there are large uncertainties in the few existing design rules, the best thing at the moment is to assume that in areas with holes practically no forces can be transferred by the timber but instead reinforcement has to be used around the holes. This can for example be done by reinforcing with rods or with plywood. In order to do so there are some design advices given in literature that can be adopted but also these should be used with great care.

3.2 Design of slender members

Timber members are often designed fairly slender. Therefore, it is crucial to pay due attention during the design process, in order to avoid problems related to stability.

3.2.1 Column buckling

As said previously there is a tendency for axial loaded members subjected to compression to displace laterally which eventually can lead to failure by buckling as shown in Figure 3.9. The more slender the member is the larger the risk for buckling. The slenderness ratio is defined as the effective buckling length of the member, ℓ_e , divided by the radius of gyration, i , of the cross section:

$$\lambda = \frac{\ell_e}{i}$$

where the radius of gyration about an axis $i = \sqrt{I/A}$, I is the second moment of area, and A is the cross-sectional area of the member.

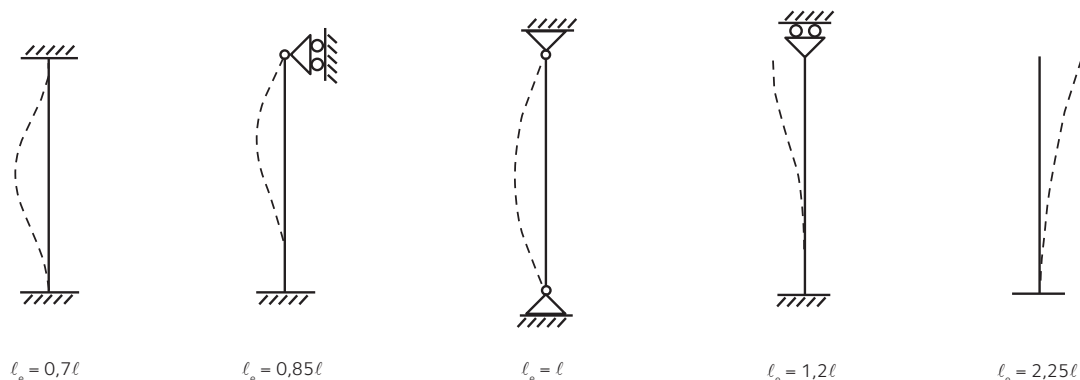


Figure 3.9: Effective buckling length ℓ_e for different end conditions. ℓ is actual column length.

Buckling will occur about the axis with the highest slenderness ratio. The effective length ℓ_e (or buckling length) of a compression member is the distance along its length between adjacent points of contra-flexure, where the bending moment in the member is zero. Examples of values for the buckling lengths that can be used in practical design are given in Figure 3.9. The values in Figure 3.9 are slightly higher than the theoretical values given by Euler theory because consideration has to be taken to uncertainties in boundary conditions. For an idealized perfectly straight column of length ℓ having uniform properties and pin jointed at both ends, the theoretical axial load at which buckling will occur about the y - y or the z - z axes will be the Euler buckling load for the respective axis. Expressing the Euler buckling load in terms of the effective length, it can be written as:

$$N_{cr} = \frac{\pi^2 EI}{\ell_e^2} \quad \text{Eq. 3.9}$$

The expression given above is a theoretical value of the load capacity since no real column is perfectly straight and also other factors will influence the behaviour of a column subjected to axial compression such as:

- strength/stiffness – compressive strength and modulus of elasticity of the material
- geometry of the member – cross-sectional sizes and length
- support conditions, which are accounted for by effective buckling length
- geometric imperfections – deviations from nominal sizes, initial curvature and inclination
- material variations and imperfections – density, effect of knots, effect of compression, wood and moisture content.

This is usually described by introducing a reduction factor χ that depends on the slenderness ratio λ and on the above mentioned factors so that the compression capacity is given by:

$$N_{c,R} = \chi \cdot f_{c,d} \cdot A \quad \text{Eq. 3.10}$$

where $f_{c,d}$ is the design value of the compression strength and A is the cross section area.

For wooden members the influence of the above mentioned factors have to be considered in the calculations. The reduction factor χ has been evaluated from simulations for a large number of columns, Blass (1995). The columns were modelled by assigning them material properties and geometric imperfections based on observations of real columns. Material properties, geometry, initial curvature and deviations from target values were chosen randomly for each simulated column. Correlation between different properties was taken into account. The calculation of the ultimate load for the simulated columns was based on second order plastic analysis, using the plastic deformation potential of timber subjected to compression. Based on the simulations, buckling curves for different circumstances were developed. In order to do so a measure called the relative slenderness ratio λ_{rel} was defined:

$$\lambda_{rel} = \frac{\lambda}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad \text{Eq. 3.11}$$

where $f_{c,0,k}$ is the characteristic compressive strength of the timber parallel to the grain and $E_{0,05}$ is the fifth percentile of the elastic modulus.

From the simulation results, curves have been derived in terms of $\lambda_{\text{rel},y}$ (or $\lambda_{\text{rel},z}$) from which buckling strength reduction factors, $k_{c,y}$ (or $k_{c,z}$), called instability factors, are obtained for structural timber and glulam members subjected to axial compression, see also Figure 3.10. This gives the following form for compressive strength:

$$N_{c,Rd} = k_c \cdot f_{c,0,d} \cdot A \quad \text{Eq. 3.12}$$

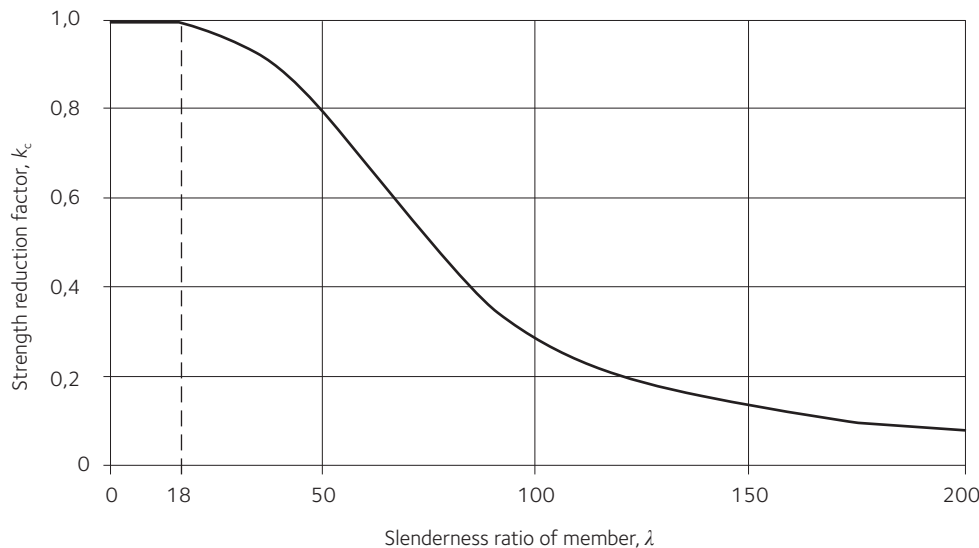


Figure 3.10: Typical graph showing the strength reduction in a structural timber compression member as a function of the slenderness ratio.

3.2.2 Combined bending and axial compression

For members subjected to compression and bending three cases can be distinguished: the case when no instability effects are valid, a case where buckling may occur and the case when both buckling and lateral instability may occur. The last case will be discussed in Section 3.2. In the following the other two cases will be discussed.

For a member with a low slenderness ratio subjected to compression and bending, no buckling will occur and failure will be based on the compressive strength of the member. Figure 3.11 shows interaction diagrams for a member subjected to combined bending moment and axial compression based on the application of elastic and plastic theory. Under elastic theory, the failure condition will occur when the combined compression stress in the member reaches the compressive strength of the material. When plastic theory applies, the material yields when it reaches the compressive strength allowing the stress in the section to redistribute to some extent, which will enhance the strength. With the elastic theory approach, the sum of the combined axial and bending stress to strength ratios cannot exceed unity. With the plastic theory solution however, the axial stress to axial strength ratio becomes a squared function enabling the member to take a higher value of bending stress to bending strength ratio for the same value of axial stress to strength ratio.

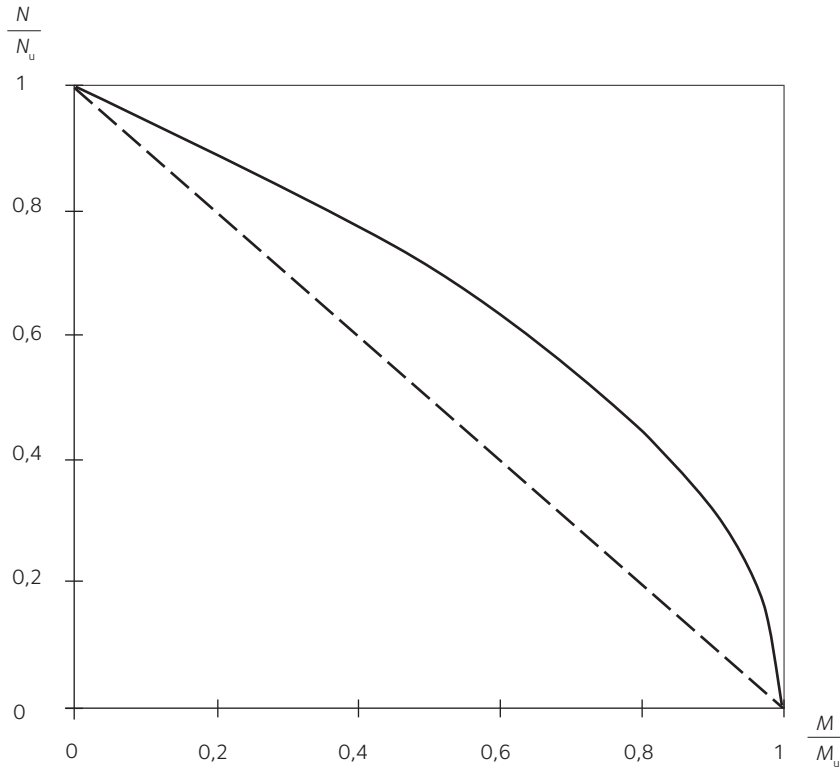


Figure 3.11: Interaction diagrams for combined bending and axial compression of a member about an axis. N and M is the applied load and moment and N_u and M_u is the ultimate load and moment, valid for either elastic (dashed line) or plastic (solid line) theory.

In the case where no strength reduction is occurring, that is when the slenderness is low in the member, the strength benefits associated with the plastic behaviour of timber when subjected to compression stresses can be adopted. This is described in the following equation in the Eurocode 5 for the case with bending about both axes:

$$\frac{M_{y,Ed}}{M_{y,Rd}} + k_m \frac{M_{z,Ed}}{M_{z,Rd}} + \left(\frac{N_{c,Ed}}{N_{c,0,Rd}} \right)^2 \leq 1 \quad \text{Eq. 3.13}$$

where k_m is a reduction factor that depends on the form of the cross section, see Section 3.1.2.1.

In the case when the slenderness value is high, that is when 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. This is in line with the elastic theory interaction approach shown in Figure 3.11. The equation to be used in design of members subjected to both compression and bending is then given by Eurocode in the following form for the case with bending around both axes:

$$\frac{M_{y,Ed}}{M_{y,Rd}} + k_m \frac{M_{z,Ed}}{M_{z,Rd}} + \frac{N_{c,Ed}}{k_{c,y} \cdot N_{c,0,Rd}} \leq 1 \quad \text{Eq. 3.14}$$

where k_c is the reduction factor for buckling. The equation is valid for buckling around the y - y axis.

3.2.3 Lateral buckling

3.2.3.1 Introduction

For an idealized, perfectly straight elastic beam, there will be no out-of-plane deformations as long as the bending moment generated by the applied external loading P is lower than the elastic buckling moment M_{crit} . When the bending moment generated by P exceeds M_{crit} , the beam buckles by deflecting laterally and twisting, as shown in Figure 3.12. If the compression zone of the beam is not braced to prevent lateral movement, the member may buckle at a bending stress that is less than the design value defined in Section 3.1.1.

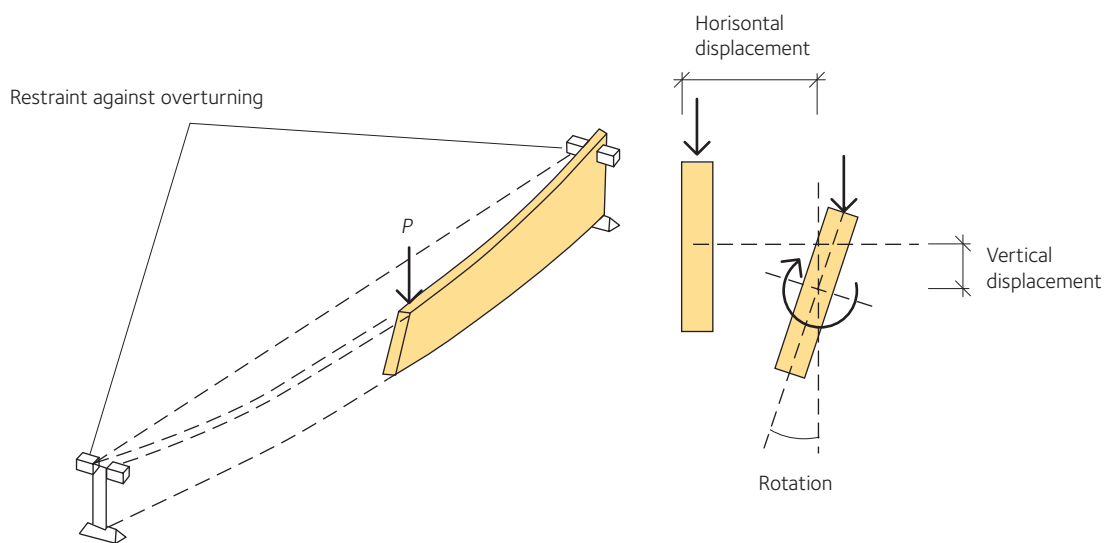


Figure 3.12: Lateral-torsional buckling of simply supported beam.

In practice, however, as in axially loaded columns, beams are never perfectly straight, nor perfectly homogeneous, and are often not loaded in the plane that is assumed for design and analysis.

Rarely, moreover, does a timber beam exist with its compression zone entirely free of all restraint. Most beams, in fact, are laterally supported by such things as attached plywood or decking, nogging pieces, purlins, or diagonal bracing.

It should be pointed out that it is necessary to examine not only the individual beam for adequate bracing, but also the entire system.

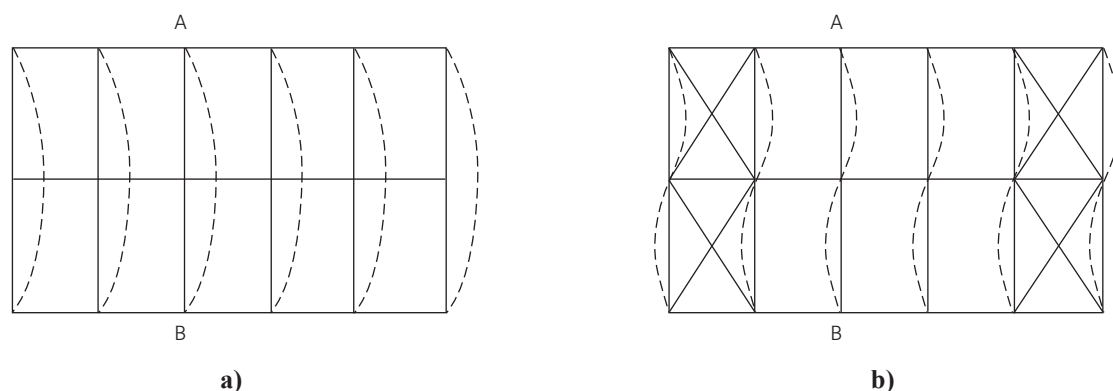


Figure 3.13: Lateral buckling of a roof or floor system. a) Unbraced, b) braced.

Figure 3.13 a) shows a beam AB with a cross beam framing at midlength; in such a case, buckling of the entire system is still possible unless the system is braced, as shown in Figure 3.13 b). All too frequently in design, the engineer encounters situations where the adequateness 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 purlins or light-gauge decking floor systems that rest on but are not solidly attached to the beams.

However, it is better to assume no lateral support in doubtful situations. Alternatively, it may be possible in some cases to evaluate it as an elastic restraint. Lateral support must not be ignored: many failures in timber structures are the result of inadequate bracing against lateral instability. The engineer should also carefully consider the construction stage when all the restraints which may eventually act are not yet in place.

3.2.3.2 Theory of elastic stability of beams

A beam that is bent in the plane of greatest flexural stiffness may buckle laterally at a certain critical value of the load. As long as the load on such a beam is below the critical value, the beam will be stable. As the load is increased, however, a condition is reached at which a slightly deflected and twisted form of equilibrium becomes possible. The lowest load at which this critical condition occurs represents the critical load for the beam.

To determine the critical load for the special case of uniform bending moment along the length of the beam, consider the beam of Figure 3.14 with applied end moments M , about an axle parallel to the y -axis. The beam ends are free to rotate about the y -axis, but are fixed against twisting (that is the beam ends are fixed against rotation about z -axis).

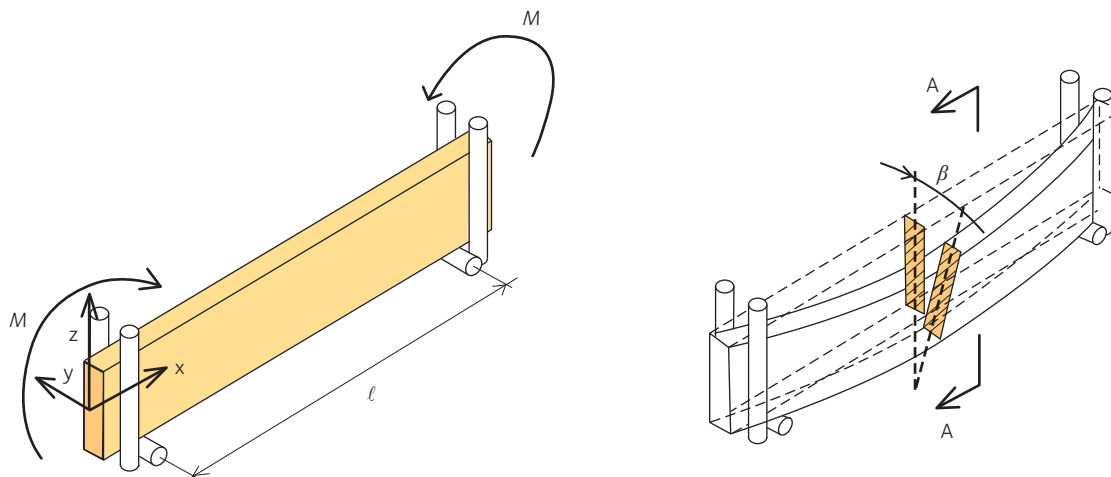


Figure 3.14: Lateral torsional buckling of laterally unsupported beam subjected to pure bending moment about the strong axis.

Due to the external bending moment, the beam deflects both in the vertical direction, that is parallel to z -axis, and in the transversal direction, that is parallel to y -axis. These two deformations are interdependent; when the beam deflects laterally, the applied moment has a component which exerts a torque about the deflected longitudinal axis which causes the beam to twist.

The displacement in the z -direction is denoted w and $dw/dz = \beta$ is the angle of twist. From equilibrium of the buckled beam, the bending and twisting moment at any cross section are according to Figure 3.15:

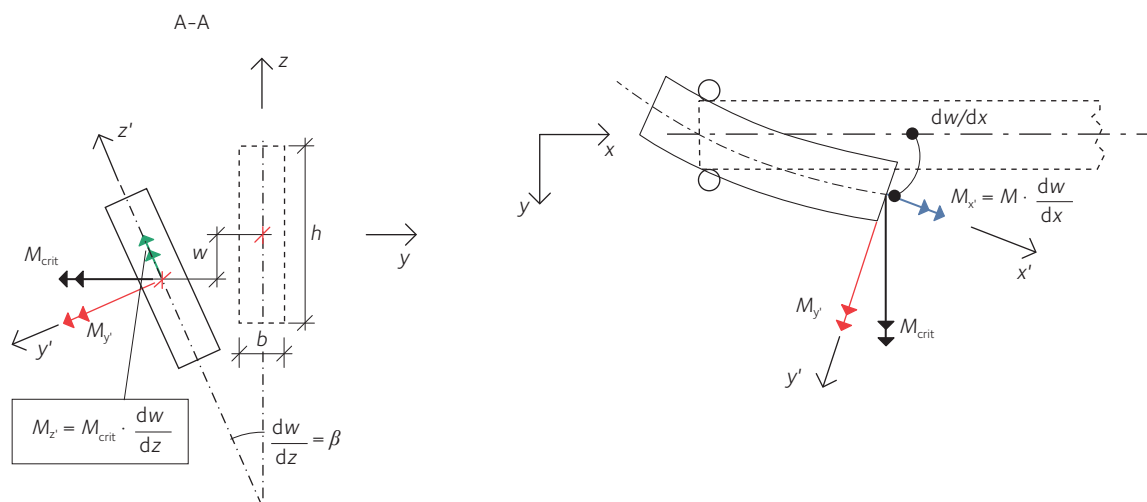


Figure 3.15: Lateral torsional buckling of laterally unsupported beam.

$$M_{x'} \approx M_{\text{crit}} \cdot \frac{dw}{dx} \quad \text{Eq. 3.15}$$

$$M_{y'} \approx M_{\text{crit}} \quad \text{Eq. 3.16}$$

$$M_{z'} \approx M_{\text{crit}} \cdot \frac{dw}{dz} = M_{\text{crit}} \cdot \beta \quad \text{Eq. 3.17}$$

But by the theory of elasticity:

$$M_z = -E \cdot I_z \cdot \frac{d^2 w}{dx^2} \quad \text{Eq. 3.18}$$

and:

$$M_x = G \cdot K_v \cdot \frac{d\beta}{dx} \quad \text{Eq. 3.19}$$

where:

E	modulus of elasticity of the beam
I_z	moment of inertia about the weak axis
K_v	torsional stiffness factor
G	shear modulus

Inserting Equation 3.15 into Equation 3.19 and then differentiating with respect to x gives:

$$G \cdot K_v \cdot \frac{d^2 \beta}{dx^2} = M_{\text{crit}} \cdot \frac{d^2 w}{dx^2} \quad \text{Eq. 3.20}$$

Inserting Equation 3.18 into Equation 3.17 and then substituting in Equation 3.20:

$$G \cdot K_v \cdot \frac{d^2 \beta}{dx^2} = M_{\text{crit}} \cdot \left(-\frac{M_{\text{crit}} \cdot \beta}{E \cdot I_z} \right) \quad \text{Eq. 3.21}$$

or:

$$\frac{d^2 \beta}{dx^2} + \frac{M_{\text{crit}}^2 \cdot \beta}{E \cdot I_z \cdot G \cdot K_v} = 0 \Rightarrow \frac{d^2 \beta}{dx^2} + \phi^2 \cdot \beta = 0 \quad \text{Eq. 3.22}$$

where:

$$\phi^2 = \frac{M_{\text{crit}}^2}{E \cdot I_z \cdot G \cdot K_v} \quad \text{Eq. 3.23}$$

The solution of differential Equation 3.22 is:

$$\beta(x) = A \cdot \cos(\phi \cdot x) + B \cdot \sin(\phi \cdot x) \quad \text{Eq. 3.24}$$

where A and B are arbitrary constants.

The boundary conditions are that the twisting angle is zero at the supports, that is:

$$\begin{aligned} \beta(x=0) = 0 &\Rightarrow A = 0 \\ \beta(x=\ell) = 0 &\Rightarrow B \cdot \sin(\phi \cdot \ell) = 0 \end{aligned} \quad \text{Eq. 3.25}$$

The nontrivial solution is therefore:

$$\sin(\phi \cdot \ell) = 0 \Rightarrow \phi \cdot \ell = n \cdot \pi \quad \text{Eq. 3.26}$$

where n is any integer. For the fundamental buckling mode, $n = 1$.

Substituting Equation 3.23 into Equation 3.26 gives:

$$M_{\text{crit}} = \frac{\pi}{\ell} \cdot \sqrt{E \cdot I_z \cdot G \cdot K_v} \quad \text{Eq. 3.27}$$

The corresponding bending stress is:

$$\sigma_{\text{m,crit}} = \frac{M_{\text{crit}}}{W_y} = \frac{6 \cdot \pi}{b \cdot h^2 \cdot \ell} \cdot \sqrt{E \cdot I_z \cdot G \cdot K_v} \quad \text{Eq. 3.28}$$

3.2.3.3 The approach according to Eurocode 5

As common for buckling phenomena, Eurocode 5 proposes the use of a reduction factor to be applied to the strength of the beam. In the case of an unsupported beam, such a factor, k_{crit} is used to keep the bending stresses in the compression zone of the beam lower than the bending stresses which would induce lateral torsional buckling:

$$\sigma_{\text{m,d}} \leq k_{\text{crit}} \cdot f_{\text{m,d}} \quad \text{Eq. 3.29}$$

where:

$\sigma_{\text{m,d}}$	is the design bending stress
$f_{\text{m,d}}$	is the design bending strength
k_{crit}	is a factor which takes into account the reduced bending strength due to lateral torsional buckling.

In Eurocode 5, k_{crit} is given as a function of $\lambda_{\text{rel,m}}$, which is defined as the relative slenderness for bending:

$$\lambda_{\text{rel,m}} = \sqrt{\frac{f_{\text{m,k}}}{\sigma_{\text{m,crit}}}} \quad \text{Eq. 3.30}$$

The critical bending stress for a simply supported straight beam of constant section which is restrained against buckling at the supports and loaded with a constant moment (see Figure 3.14) is:

$$\sigma_{\text{m,crit}} = \frac{M_{\text{y,crit}}}{W_y} = \frac{6 \cdot \pi}{b \cdot h^2 \cdot \ell} \cdot \sqrt{E_{0,05} \cdot I_z \cdot G_{0,05} \cdot I_{\text{tor}}} \quad \text{Eq. 3.31}$$

where the index 0,05 indicates 5th-percentile values of modulus of elasticity and shear modulus (note that characteristic values are normally adopted when stability checks are performed).

For a rectangular section of width b , depth h , the moment of inertia about the weak axis is $I_z = (h \cdot b^3/12)$, and the torsional stiffness factor for relatively slender beams can be taken to approximately equal $I_{tor} \approx (b^3 \cdot h/3)$. Inserting these relationships in Equation 3.31 the elastic critical bending stress can be written as:

$$\sigma_{m,crit} = \frac{\pi \cdot b^2}{h \cdot \ell} \cdot \sqrt{E_{0,05} \cdot G_{0,05}} \quad \text{Eq. 3.32}$$

For softwood rectangular sections, the ratio of $E_{0,05}/G_{0,05}$ is approximately 16, and by applying this to Equation 3.32, the critical bending stress of a beam bent about its strong axis can be written as:

$$\sigma_{m,crit} \approx \frac{0,78 \cdot b^2}{h \cdot \ell} \cdot E_{0,05} \quad \text{Eq. 3.33}$$

where $E_{0,05}$ is the 5th-percentile value of the modulus of elasticity parallel to the grain, b is the width of the beam, h is the depth of the beam, ℓ is the design span of the simply supported beam between lateral supports at the ends of the beam. Equation 3.33 is only valid for a uniform moment condition applied to a simply supported beam where the beam ends are prevented from moving laterally, are free to rotate in plan, and are fully restrained against torsional rotation.

For situations with different end fixing conditions and where moment is induced by other types of loading, as well as the cases where load is applied at the compression (or tension) face rather than the centroidal axis of the beam, the elastic critical moment can be obtained by using the same expression but replacing the design span ℓ by what is termed the “effective length”, ℓ_{ef} , of the beam:

$$\sigma_{m,crit} \approx \frac{0,78 \cdot b^2}{h \cdot \ell_{ef}} \cdot E_{0,05} \quad \text{Eq. 3.34}$$

The effective length is obtained by adjusting the design span to take account of the effect of the moment distribution and end fixing conditions. Values for commonly used cases in timber design are given in Table 3.2.

Table 3.2 Effective length as a ratio of the span.

Beam type	Loading type	ℓ_{ef}/ℓ
Simply supported	Constant moment	1,0
	Uniformly distributed load	0,9
	Concentrated force at midspan	0,8
Cantilever	Uniformly distributed load	0,5
	Concentrated force at the free end	0,8

Source: Table according to SS-EN 1995-1-1:2004, 6.3.3.

The ratio between the effective length ℓ_{ef} and the span ℓ is valid for a beam with torsionally restrained supports and loaded at the centroidal axis. If the load is applied at the compression edge of the beam, ℓ_{ef} should be increased by $2 \cdot h$ and should be decreased by $0,5 \cdot h$ for a load at the tension edge of the beam, where h is the depth of the beam.

Once the slenderness ratio for bending $\lambda_{rel,m} = (f_{m,k}/\sigma_{m,crit})^{0.5}$ is known, the reduction factor k_{crit} can be determined according to Table 3.3.

Table 3.3 Values of k_{crit} as a function of $\lambda_{rel,m}$.

Value of k_{crit}	Relative slenderness for bending, $\lambda_{rel,m}$
1	$\lambda_{rel,m} \leq 0,75$
$1,56 - 0,75 \lambda_{rel,m}$	$0,75 < \lambda_{rel,m} \leq 1,4$
$1/(\lambda_{rel,m})^2$	$\lambda_{rel,m} > 1,4$

A graphical representation of k_{crit} plotted against the relative slenderness ratio for bending, $\lambda_{rel,m}$, is shown in Figure 3.16.

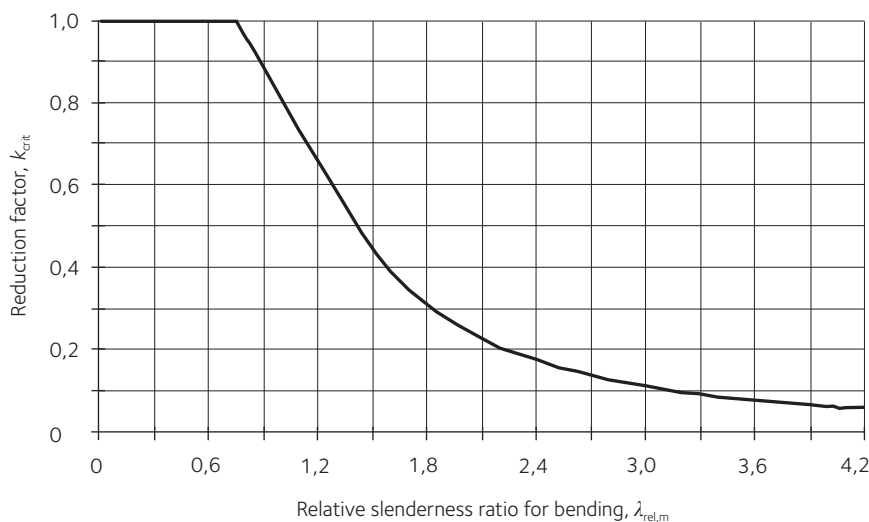


Figure 3.16: The value of k_{crit} for different values of the relative slenderness ratio $\lambda_{rel,m}$.

As can be seen from Figure 3.16 and Table 3.3, the value $\lambda_{rel,m} = 0,75$ has been adopted as the limit below which the beam will be stiff enough to prevent lateral torsional buckling. In other words, full bending strength can be achieved without risk for instability problems if $\lambda_{rel,m} < 0,75$. This has been shown also experimentally, see Piazza et al. (2005).

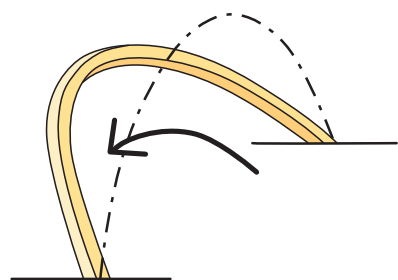
When $\lambda_{rel,m} > 1,4$, that is when $f_{m,k} > 2 \cdot \sigma_{m,crit}$ the beam is considered to fail solely by elastic buckling and the elastic critical bending strength will become the design condition. Finally, for relative slenderness values between 0,75 and 1,4 the beam will fail in bending, but after rather pronounced lateral (and vertical) deformations. Since there will be an interaction of bending in two directions (vertical and lateral) and also torsion, the beam will not be able to achieve the full design bending strength. Eurocode 5 adopts the approximation of a linear relationship between these limits.

3.2.4 Buckling of frames and arches

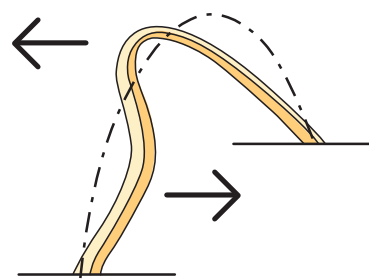
In frames and arches, as in columns, the buckling resistance about both the major axis and the minor axis must be verified by stability checks.

3.2.4.1 Lateral behaviour with special emphasis on out-of-plane buckling

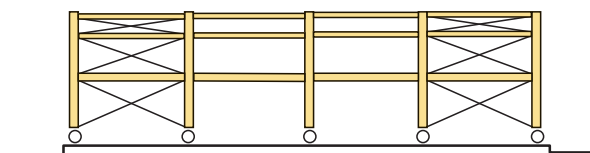
It is obvious that a frame or an arch, which lies in one vertical plane, must be prevented from toppling over sideways, see Figure 3.17 a). Two methods could be adopted to prevent this. One is fixed-base connections, which is rather cumbersome to achieve and which also requires – especially in case of large structures – a massive foundation to prevent overturning. Another, more commonly used method for achieving lateral stability is by bracing with members that are transversal to the frame or to the arch, as shown in Figure 3.17 c). The second major problem with respect to the behaviour of frames and arches in the lateral direction is that of lateral buckling (or out-of-plane buckling). Since timber elements can be fairly slender, out-of-plane buckling of the type illustrated in Figure 3.17 b) may occur.



a) Arch turning over sideways.



b) Lateral buckling due to high normal force.



c) The lateral stability can be achieved by lateral bracing of arches with other elements, such as purlins from the roof structure. Cross bracing or some other mechanism is needed to assure lateral stability.

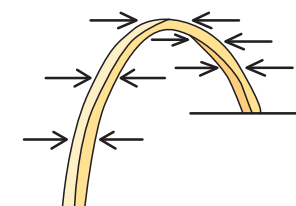


Figure 3.17: Considerations concerning the lateral behaviour of arches.

One solution to the out-of-plane buckling is to increase the stiffness of the frame or the arch in the lateral direction by increased lateral dimension. When braced according to Figure 3.17 c), the frame or the arch can become fairly slender. It should be noted that the same system used to stabilize arches or frames from overturning laterally also provides lateral bracing and prevents lateral buckling from occurring.

In normal portal frames and arches, **out-of-plane buckling** is checked in the same way as any other beam-column between lateral restraints and torsional restraints provided by bracings. These bracings make the effective lengths of each element easily identifiable. In the case of a portal frame, for example, the out-of-plane buckling should be checked in the unbraced zones as for a column with buckling length according to Figure 3.18.

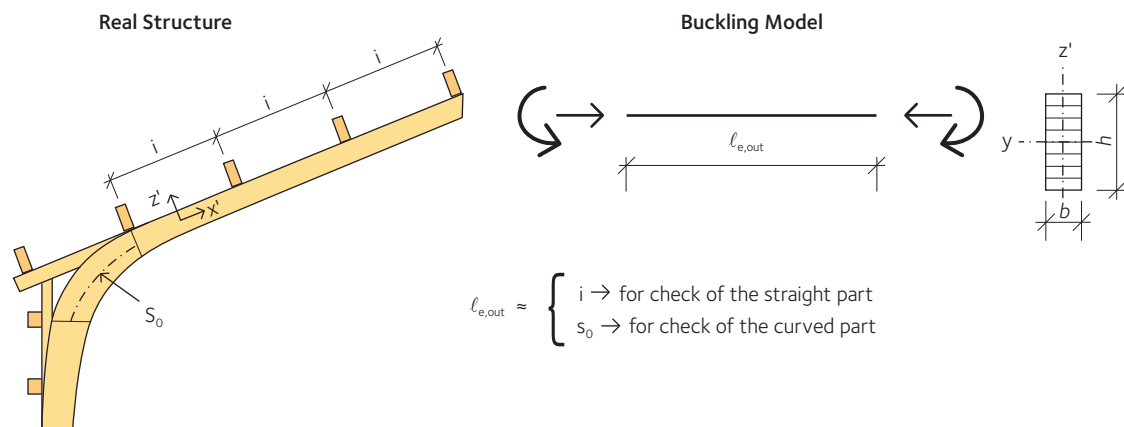


Figure 3.18: Buckling lengths for out-of-plane buckling of a portal frame.

3.2.4.2 In-plane buckling

Buckling **in the plane of the frame** or of the arch is in general more complicated than in normal beam-column elements. The common method of analysis for frames and arches are:

- **Simplified analysis**
- **Second order analysis**

If **simplified analysis** is adopted, frames and arches can be verified in the same manner as beam-columns, that is elements 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 geometrical and structural in-plane and lateral imperfections and induced in-plane deflections are taken into account by multiplying the compression and bending strength values by buckling reduction factors k_c and k_{crit} . The first buckling mode of a portal frame (asymmetric buckling) has the shape shown in Figure 3.19.

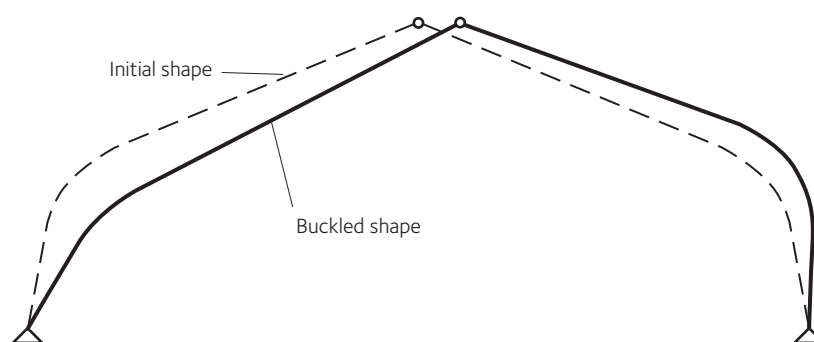


Figure 3.19: In-plane buckling of a three-hinged portal frame.

For the determination of the buckling load according to the simplified analysis, the buckling length ℓ_e is required. In order to find a reasonable value for such a length, let us first study a three-hinged arch. For common rise to span ratios – that is h/ℓ within the range 0,14 – 0,3 – the lowest buckling mode for arches is unsymmetrical and has the shape shown in Figure 3.20.

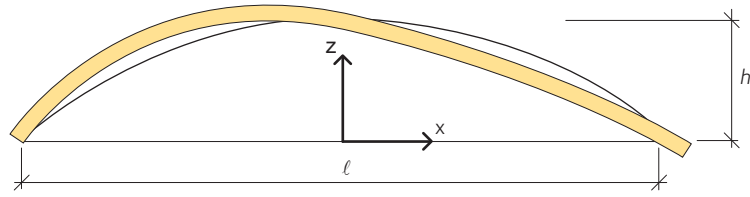


Figure 3.20: Unsymmetrical buckling shape of an arch.

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 one, two or three hinges, can be expressed by the following equation:

$$q_{crit} = \gamma_4 \cdot \frac{E \cdot I}{\ell^3} \tag{Eq. 3.35}$$

The numerical factor γ_4 is expressed graphically as a function of h/ℓ in Figure 3.21, where the portion of the curves indicated by dotted lines correspond to symmetrical forms of buckling. In these cases unsymmetrical buckling can still occur and in obtaining values of γ_4 , the curves for arches without central hinge (two-hinged arch) must be used. For example, in the case of a three-hinged arch with $h/\ell > 0,3$ the γ_4 value is taken from the curve for a two-hinged arch.

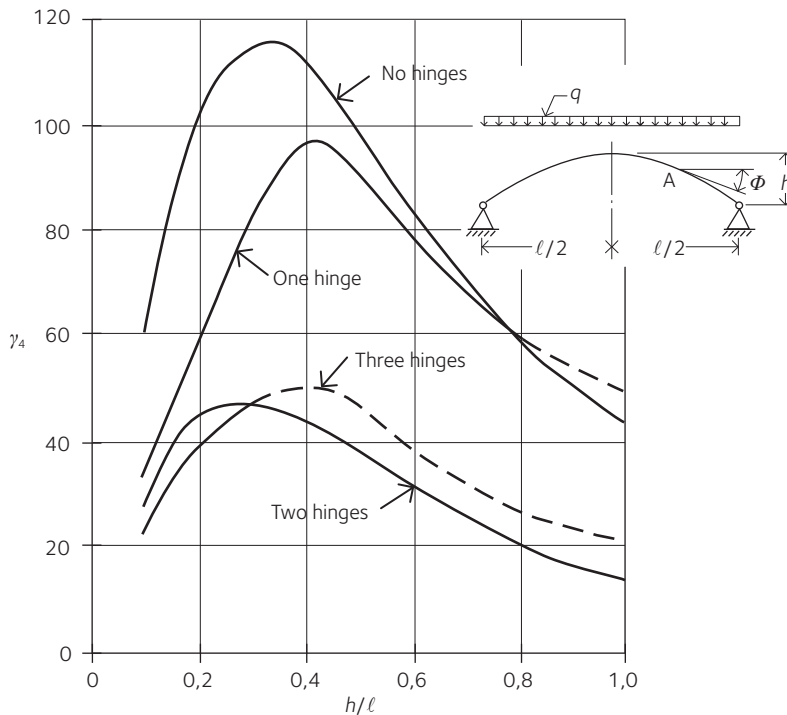


Figure 3.21: γ_4 values as a function of h/ℓ .

Let us now derive the buckling length for a parabolic three-hinged arch with constant cross section and uniformly distributed load. We can assume a ratio $f/\ell \approx 0,14 - 0,15$ (which is a very common ratio for timber arches). The arch length ℓ_{arch} of a symmetrical parabolic arch with span ℓ and rise f can be calculated using the following approximate formula:

$$\ell_{arch} \approx 1,06 \cdot \ell \tag{Eq. 3.36}$$

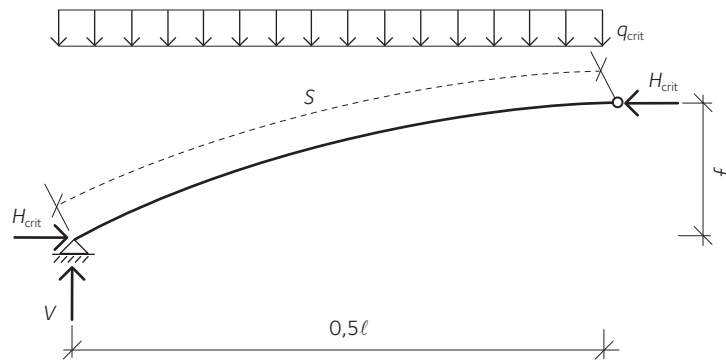


Figure 3.22: Forces acting on a half three-hinged arch.

The critical value of normal force at the crown of the arch:

$$H_{\text{crit}} = \frac{q_{\text{crit}} \cdot \ell^2}{8 \cdot f} = \gamma_4 \cdot \frac{E \cdot I}{8 \cdot f \cdot \ell} \quad \text{Eq. 3.37}$$

can be compared with the critical load for an axially loaded column with length $s = \ell_{\text{arch}}/2 (= 0,53 \cdot \ell)$:

$$N_{\text{crit}} = \pi^2 \cdot \frac{E \cdot I}{(\beta \cdot s)^2} = \pi^2 \cdot \frac{E \cdot I}{\beta^2 \cdot 0,53^2 \cdot \ell^2} \quad \text{Eq. 3.38}$$

where β is buckling length factor. Imposing $H_{\text{crit}} = N_{\text{crit}}$, $h/\ell = 0,15$ and $\gamma_4 \approx 30$ (see Figure 3.22):

$$\beta = 1,17 \quad \text{Eq. 3.39}$$

For practical design, the effective length for buckling is often assumed to be:

$$\ell_e = 1,25 \cdot s \quad \text{Eq. 3.40}$$

Analogously, in the case of a portal frame the buckling length of the “equivalent column” can be assumed to be:

$$\ell_e = 1,25 \cdot a \quad \text{Eq. 3.41}$$

with symbols shown in Figure 3.23.

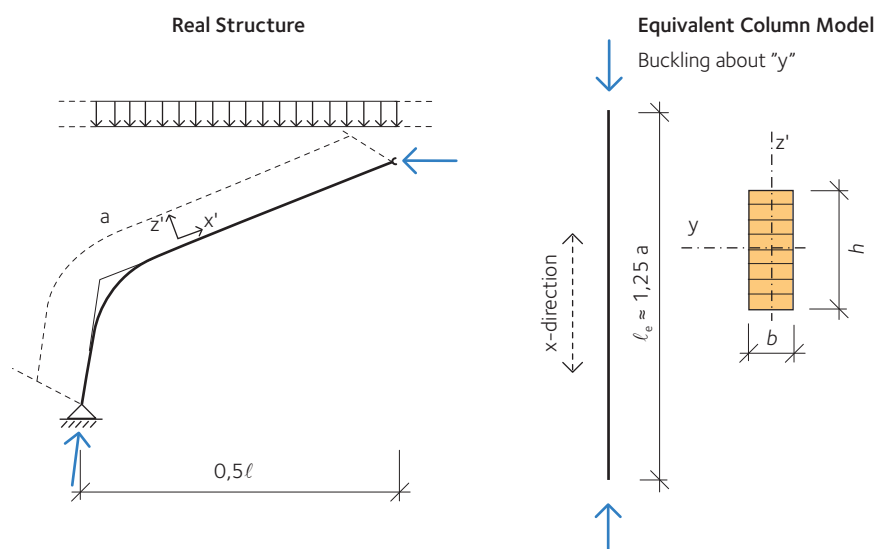


Figure 3.23: Buckling length for in-plane buckling of a portal frame.

If **second order analysis** is adopted, the calculation of stresses is based on geometric nonlinear theory considering equilibrium of the deformed static system. When any frame or arch is loaded, it deflects so that the shape under load is different from the undeformed shape, which is the shape in the analysis if this is ordinary small deflection (first-order) analysis. The deflection causes the axial loads in the members to act along different lines from those assumed in the analysis, as shown in Figure 3.24. The effects, commonly called second-order effects or P-delta effects, can reduce the resistance of the frame.



Figure 3.24: Left: asymmetric (or sway mode) deflection. Right: symmetric deflection.

In second order analysis, design loads are gradually applied in increments. Before an incremental load is applied on the structure, displacements due to previously applied loads are calculated. Incremental loads are applied at locations worked out after adding displacements due to previously applied loads. Thus effect of all displacements, including horizontal sway, due to previously applied loads on member forces is considered in second order analysis. This type of analysis is commonly performed by means of adequate finite element based computer programmes.

If geometrical imperfections are also modelled, member forces given by second order analysis can be directly used for member design, without reduction of the design strength due to buckling. Initial deformation should be assumed to be affine with the deformation figure under load and can be assumed to correspond to an initial deformation which is found by applying an angle ϕ of inclination to the structure or relevant parts, together with an initial sinusoidal curvature between the nodes of the structure. Some examples are shown in Figure 3.25.

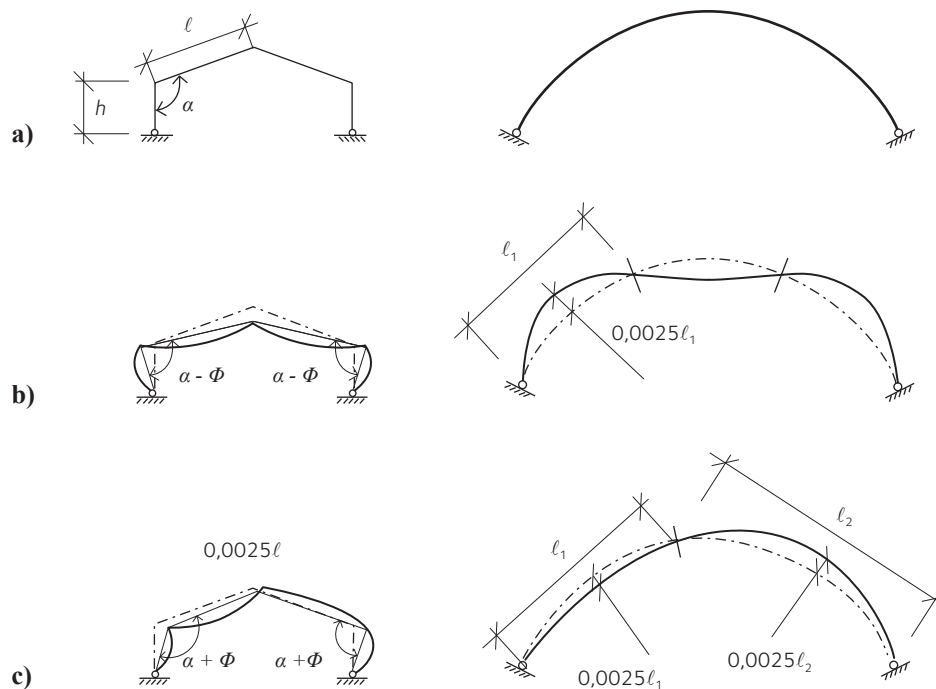


Figure 3.25: Examples of assumed initial deviations in the geometry for a) frames and arches, when b) symmetrical load and c) non-symmetrical load.

3.3 Special timber elements

By special timber elements is here meant timber components with a varying cross-sectional depth or curved members. Typical examples of members with varying cross-sectional depth are pitched beams, continuous beams with a deeper section over the intermediate supports, or frames. Typical examples of curved elements are pitched cambered beams, curved beams and arches. Examples are shown in Figure 3.26. Special timber elements are normally made of glued laminated timber, see Section 2.7.1.1.

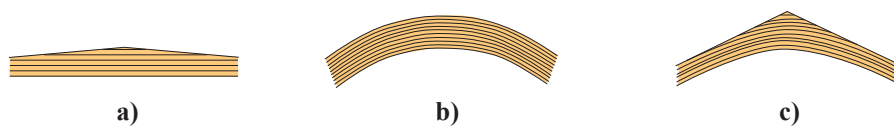


Figure 3.26: a) Double tapered beam, b) curved beam, c) pitched cambered beam.

A peculiarity of special timber elements is that the distribution of bending stresses is non-linear. In order to understand this non-linear behaviour, let us consider a curved beam element subjected to positive bending moment. The total displacements of the fibres in the curved beam are proportional to the distances of the fibres from the neutral surface. However, the strains of the fibres are not proportional to these distances because the fibres are not of equal length. Within the elastic limit the stress in any fibre of the beam is proportional to the strain of the fibre and hence the elastic stresses in the fibres of a curved beam are not proportional to the distances of the fibres from the neutral surface. Hence, the neutral axis in a curved beam does not pass through the centroid of the section. The distribution of stress over the section and the relative position of the neutral axis are shown in Figure 3.27.

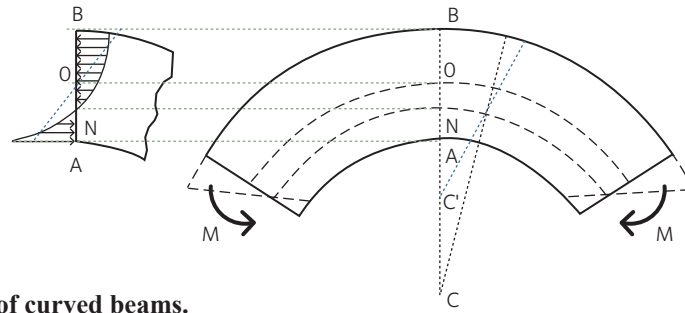


Figure 3.27: Bending of curved beams.

If the beam were straight, the stress would be zero at the centroidal axis and would vary proportionally with the distance from the centroidal axis as indicated by the dotted line in the figure.

The formulas which will be shown in the next paragraphs were derived assuming the material to be isotropic. In reality, timber is an anisotropic material. However, it has been shown experimentally that the formulas derived for isotropic materials are also valid for anisotropic materials such as timber, see Stalnaker et al. (1989).

3.3.1 Tapered beams

Structural timber elements, especially glulam components, are often designed with a varying cross-sectional depth, for example symmetrical double tapered beams, continuous beams with a deeper section over the intermediate supports, or frames. As a rule, variations in the cross-section are achieved by tapering the laminations along one edge.

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 3.28. 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. Note that the shear and bending stresses shown in Figure 3.28 are in the direction parallel to laminations, rather than parallel to the tapered side.

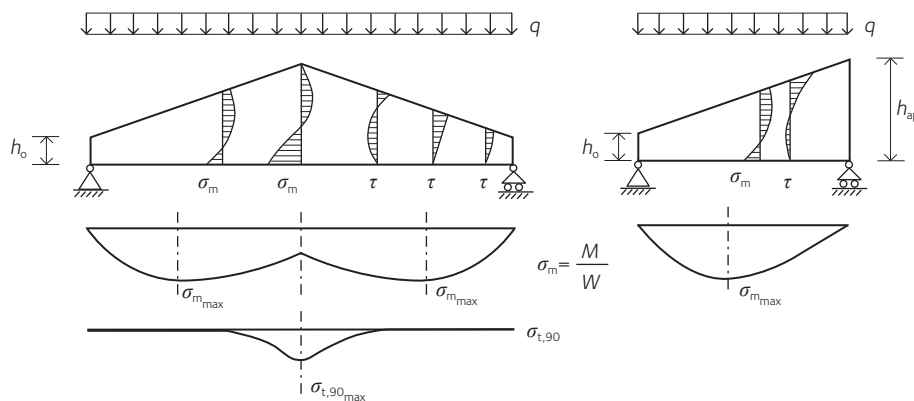


Figure 3.28: Bending stresses (σ_m), shear stresses (τ) and perpendicular to the grain tensile stresses ($\sigma_{t,90}$) in double tapered beam (left) and single tapered beam (right).

For small slopes of the tapered edge, say $\alpha \leq 7^\circ$, which covers the major part of practical cases, the effect of tapering on the bending stress is small. For design purposes, therefore, the maximum bending stress can be calculated as for a beam with constant cross sectional depth (that is $\sigma_m = M/W$), both at the tapered edge and at the straight edge. However, the bending strength f_{md} is reduced by a reduction factor at the tapered edge to take into account that shear stress and stress perpendicular to the grain are acting simultaneously with bending stress at the tapered edge, see Figure 3.29.

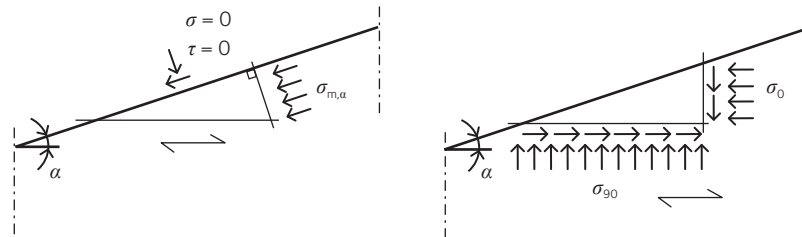


Figure 3.29: Stresses at the tapered edge of a beam. Left: bending stress parallel to the tapered edge (principal stress). Right: bending stress σ_0 parallel to the grain.

Knowing the stress component $\sigma_{m,\alpha}$ parallel to the tapered edge (Figure 3.29, left), the stress acting on the planes parallel to the grain and perpendicular to the grain respectively (Figure 3.29, right) can be calculated from the equation of statics:

$$\sigma_0 = \sigma_{m,\alpha} \cdot \cos^2 \alpha \quad \text{Eq. 3.42}$$

$$\sigma_{90} = \sigma_{m,\alpha} \cdot \sin^2 \alpha \quad \text{Eq. 3.43}$$

$$\tau = \sigma_{m,\alpha} \cdot \sin \alpha \cdot \cos \alpha \quad \text{Eq. 3.44}$$

Equations 3.42 – 3.44 show that – close to the tapered edge – there are both stresses perpendicular to the grain (σ_{90}) and shear stresses parallel to the grain (τ). The magnitude of such stresses increases with increasing slope of the tapered edge. If the tapered edge is located at the compression side of the beam, compression perpendicular to grain occurs. On the other hand, if the tapered edge is located at the tension side of the beam, tension perpendicular to grain occurs. For this reason, tapered edges with an angle exceeding 10° on the compression side or $2 - 3^\circ$ on the tension side should be avoided (it should be remembered that tension perpendicular to grain can cause brittle failure at very low level of stress and should therefore be kept as low as possible).

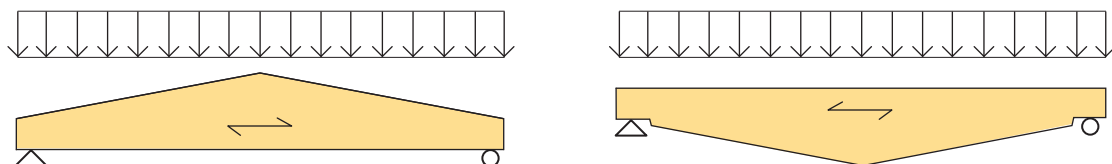


Figure 3.30: Left: compression perpendicular to the grain occurs at the tapered edge. Right: tension perpendicular to the grain occurs at the tapered edge. Nowadays the last mentioned beam shape is not advised. It is from several aspects better and more common to use so called fish beam, with curved laminations at the bottom surface and horizontal sawn at the top.

Since both the moment and the depth vary along the axis of the beam, maximum bending stress as a rule occurs not where the moment is greatest but at a section nearer the supports, see Figure 3.31. The position of this section can be determined analytically from the condition:

$$\frac{d\sigma(x)}{dx} = \frac{d}{dx} \left(\frac{M(x)}{W(x)} \right) = 0 \quad \text{Eq. 3.45}$$

For simply supported single tapered beams or symmetrical double tapered beams with uniformly distributed loads, the critical section is at the distance:

$$x = \frac{h_0}{2 \cdot h_{\text{ap}}} \cdot \ell \quad \text{Eq. 3.46}$$

from the support, where h_0 is the depth of the beam at the support and h_{ap} is the maximum beam depth.

3.3.2 Curved beams and pitched cambered beams

Amongst the most prominent advantages of glulam is the possibility of designing curved structural members. During manufacture, the individual laminations are bent to the desired form before the glue has hardened. To avoid damaging of the laminations as they are bent, the curvature must be limited. 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. As an example, let us try to estimate the bending stress in a lamination of a curved glulam beam during manufacturing, when it is bent to a given radius of curvature. Realistic values could be:

- lamination thickness: $t = 33$ mm
- radius of curvature $r_{\text{in}} = 8$ m
- modulus of elasticity $E = 13\,000$ MPa.

The theoretical bending stress which occurs in the lamination due to bending is:

$$\sigma = \frac{E \cdot t}{2 \cdot r_{\text{in}}} = \frac{13000 \cdot 33}{2 \cdot 8000} = 26,8 \text{ MPa} \quad \text{Eq. 3.47}$$

which is close to the bending strength of the lamination. However, due to the creep characteristics of timber, the bending stresses (σ) are significantly relaxed during gluing, which is carried out with added heat and moisture. These initial stresses can therefore 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 should be reduced. According to Eurocode 5, such a reduction factor must be applied when $r_{\text{in}}/t < 240$.

When bending moment is applied to a beam that is initially curved in the plane of bending, radial stresses (as well as bending stresses) occur. These radial stresses may be either tensile or compressive, see Figure 3.31.

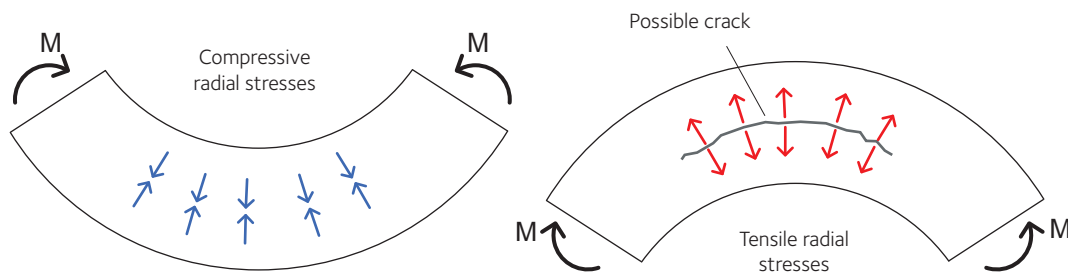


Figure 3.31: Left: bending moment tending to increase the curvature of the glulam member. Right: bending moment tending to straighten the glulam member.

When the applied moment tends to increase the curvature of the glulam member, the laminations are pressed more firmly together, see to the left in Figure 3.31. This means that compressive radial stresses occur between laminations. On the other hand, when the applied bending moment tends to straighten the glulam member, the laminations try to move apart, see to the right in Figure 3.31. This means that tensile radial stresses occur between laminations. Tensile radial stresses should be kept as low as possible since they may cause cracking of the element.

This effect can easily be demonstrated by the reader by holding together firmly several sheets of paper in an initially curved shape. Bending the bundle one way separates the pages while bending in the other way compresses them together.

Figure 3.32 shows the apex zone of a curved beam subjected to constant bending moment. Assuming, for simplification, a linear stress distribution, it can easily be shown that the resulting tensile and compressive forces, T and C respectively, give rise to a force T_{90} in the radial direction:

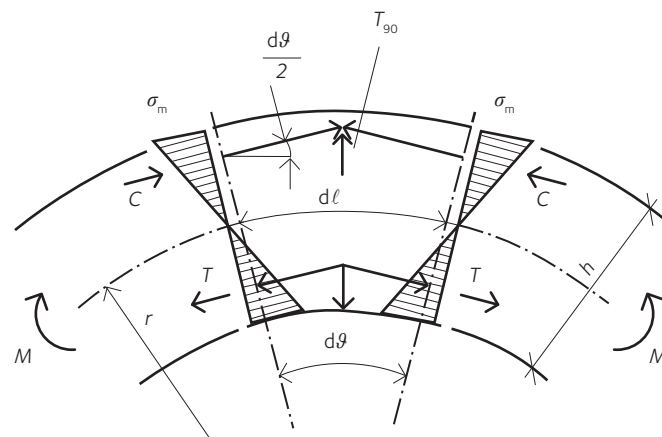


Figure 3.32: Simplified model for determining stresses perpendicular to grain at the apex of a curved beam subjected to pure bending moment.

$$T_{90} \approx 2 \cdot C \cdot \frac{d\vartheta}{2} = C \cdot d\vartheta \quad \text{Eq. 3.48}$$

Resultant forces T and C are equal to each other. For a beam cross section with breadth b and depth h , the resultants can be easily derived:

$$C = -T = \frac{1}{2} \cdot \left(\sigma_m \cdot b \cdot \frac{h}{2} \right) \quad \text{Eq. 3.49}$$

By geometric considerations:

$$d\ell = r \cdot d\vartheta \quad \text{Eq. 3.50}$$

Substituting Eq. 3.49 and Equation 3.50 into Equation 3.48:

$$T_{90} = \frac{1}{2} \cdot \left(\sigma_m \cdot b \cdot \frac{h}{2} \right) \cdot \frac{d\ell}{r} \quad \text{Eq. 3.51}$$

Knowing the tension force perpendicular to the grain, the corresponding tensile stress is:

$$\sigma_{t,90} = \frac{T_{90}}{b \cdot dl} = \frac{h}{4 \cdot r} \cdot \sigma_m = k_p \cdot \sigma_m \quad \text{Eq. 3.52}$$

Eq. 3.52 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 $k_p = h/(4 \cdot r)$. It is important to observe that increasing the beam depth h and/or decreasing the radius of curvature r will increase the magnitude of the stress perpendicular to the grain.

Various studies have shown that tension strength perpendicular to the grain $f_{t,90}$ is highly dependent on the stressed volume of the timber. The basic design value of tensile strength perpendicular to the grain must therefore be modified, for example by multiplying it by a modification factor k_{vol} and k_{dis} :

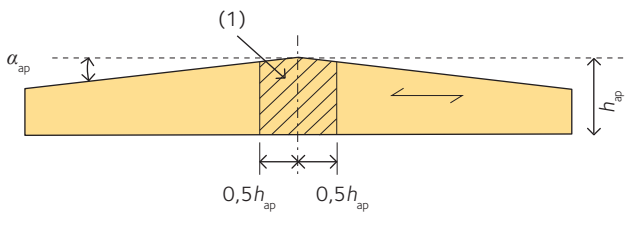
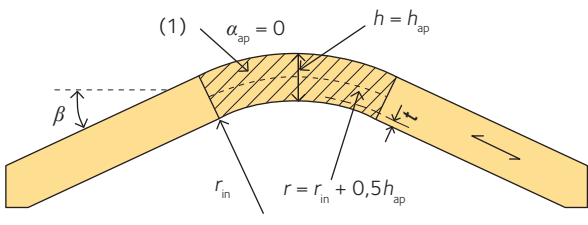
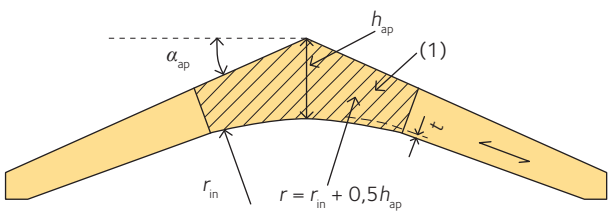
$$k_{vol} = \left(\frac{V_0}{V} \right)^{0,2} \quad \text{Eq. 3.53}$$

where :

V_0	reference volume. In EC 5, $V_0 = 0,01 \text{ m}^3$
V	stressed/curved volume determined with regard to the geometry of the member
k_{dis}	modification factor with regard to stress distribution in the beam.

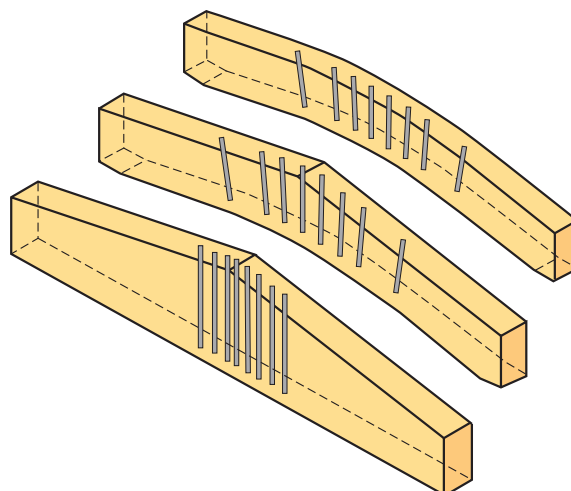
Values of k_{dis} and V for beams loaded by uniformly distributed load can be taken from Table 3.4. V needs, however, not be taken as more than $2/3V_b$, where V_b is the total volume of the beam.

Table 3.4 Values of k_{dis} and V according to Eurocode 5 for typical beam types.

Beam type	k_{dis}	V
Double tapered beam 	1,4	Volume of the stressed part (1) $\sim b \cdot (h_{ap})^2$
Curved beam 	1,4	Volume of the curved part (1) $\frac{\beta\pi}{180} b (h_{ap}^2 + 2h_{ap}r_{in})$
Pitched cambered beam 	1,7	Volume of the curved part (1) $b \left(\sin(\alpha_{ap}) \cos(\alpha_{ap}) (r_{in} + h_{ap})^2 - r_{in}^2 \frac{\alpha_{ap}\pi}{180} \right)$

Angles α and β in degrees, b relates to the width of the structural member.

In situations where the design tensile strength for stresses perpendicular to the grain is exceeded, mechanical fastenings such as glued-in rods or full-threaded screws may be used as reinforcement, see Figure 3.33.


Figure 3.33: Reinforcing of the apex zones of tapered, pitched cambered and curved beams by means of mechanical fasteners.

3.3.3 Design procedures

3.3.3.1 Tapered beams

For slopes $\alpha \leq 10^\circ$, the design bending stresses $\sigma_{m,\alpha,d}$ and $\sigma_{m,0,d}$ (see Figure 3.34) may be taken as:

$$\sigma_{m,\alpha,d} = \sigma_{m,0,d} = \frac{M_d}{W_{x_0}} = \frac{6 M_d}{b h_0^2} \quad \text{Eq. 3.54}$$

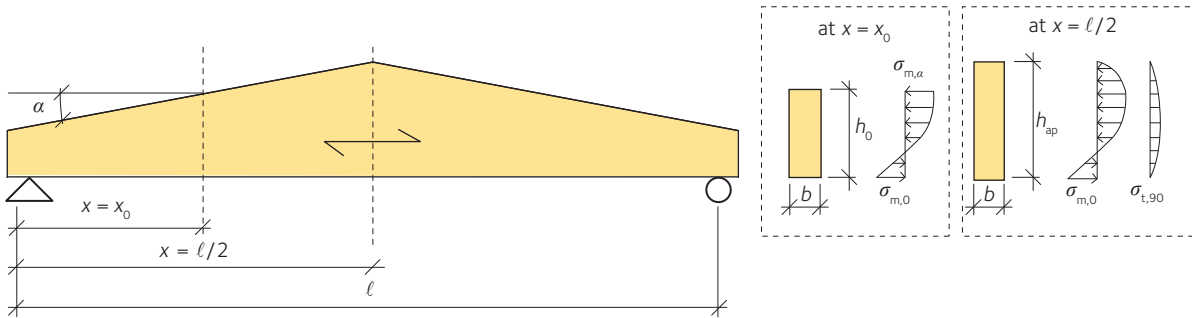


Figure 3.34: Single tapered beam showing critical cross sections for bending stresses.

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

$$\sigma_{m,\alpha,d} = \sigma_{m,0,d} \leq k_{m,\alpha} \cdot f_{m,d} \quad \text{Eq. 3.55}$$

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
- M_d is the design bending moment in the section $x = x_0$
- $x = x_0$ is the position of maximum bending stress ($x_0 = (h_0 \cdot \ell)/(2 \cdot h_{ap})$ for simply supported beams with uniformly distributed load)
- W_{x_0} is the section modulus at the cross section $x = x_0$
- $f_{m,d}$ 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.

The values of $k_{m,\alpha}$ for different slopes of the tapered edge are shown in Figure 3.35. The values are derived for the glulam class GL30c.

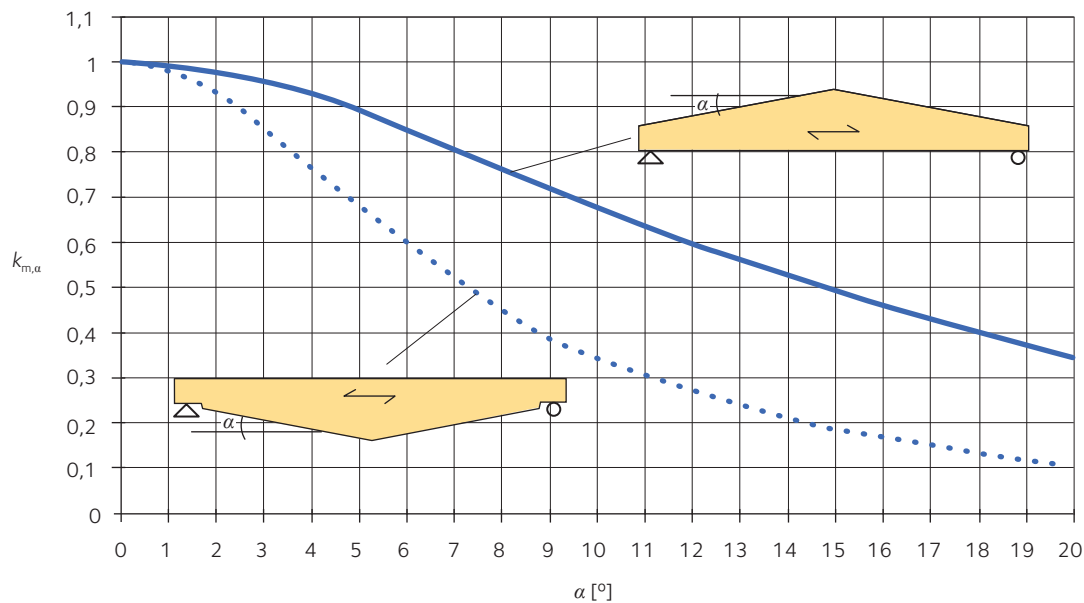


Figure 3.35: Values of $k_{m,\alpha}$ according to Eurocode 5 for different slopes of the tapered edge, glulam class GL30c.

The bending stress at the apex must also be checked, even though it seldom governs the design:

$$\sigma_{m,d} = k_\ell \cdot \frac{M_{ap,d}}{W_{ap}} = k_\ell \frac{6M_{ap,d}}{bh_{ap}^2} \quad \text{Eq. 3.56}$$

with:

- k_ℓ is a factor determined by finite element analysis that takes into account the tapering of laminations. Values of k_ℓ for glulam GL30c are given in Figure 3.37
- $M_{ap,d}$ is the design moment at the apex
- W_{ap} is the section modulus of the beam at the apex.

The design tensile stress perpendicular to the grain due to bending moment can be calculated as follows:

$$\sigma_{t,90,d} = k_p \cdot \frac{M_{ap,d}}{W_{ap}} = k_p \cdot \frac{6M_{ap,d}}{bh_{ap}^2} \quad \text{Eq. 3.57}$$

where:

- k_p is a factor determined by finite element analysis, defined as the ratio between perpendicular to grain stress and bending stress at the apex. Values of k_p for glulam GL30c are given in Figure 3.38.

The design tensile strength perpendicular to the grain must then be reduced to take into account the volume effect. According to Eurocode 5, the following inequality must be satisfied:

$$\sigma_{t,90,d} \leq k_{\text{dis}} \cdot \left(\frac{0,01}{V} \right)^{0,2} \cdot f_{t,90,d} \quad \text{Eq. 3.58}$$

with:

- k_{dis} , see Table 3.4
- V , see Table 3.4
- $f_{t,90,d}$ is the tensile strength perpendicular to the grain.

3.3.3.2 Curved beams and pitched cambered beams

The most critical section of curved beams and pitched cambered beams is normally the one at the apex, see Figure 3.36.

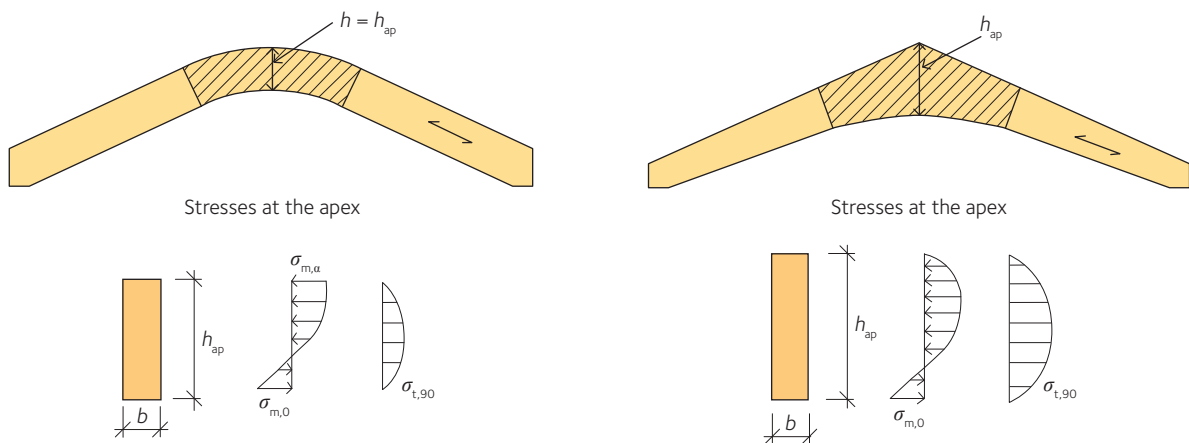


Figure 3.36: Bending stresses and tension stresses perpendicular to the grain for: curved beam (left) and pitched cambered beam (right).

The bending stress at the apex can be calculated as follows:

$$\sigma_{m,d} = k_{\ell} \cdot \frac{M_{\text{ap},d}}{W_{\text{ap}}} = k_{\ell} \frac{6M_{\text{ap},d}}{bh_{\text{ap}}^2} \quad \text{Eq. 3.59}$$

where:

- k_{ℓ} is a factor determined by finite element analysis that takes into account the geometry of the beam. Values of k_{ℓ} for glulam GL30c are given in Figure 3.37.

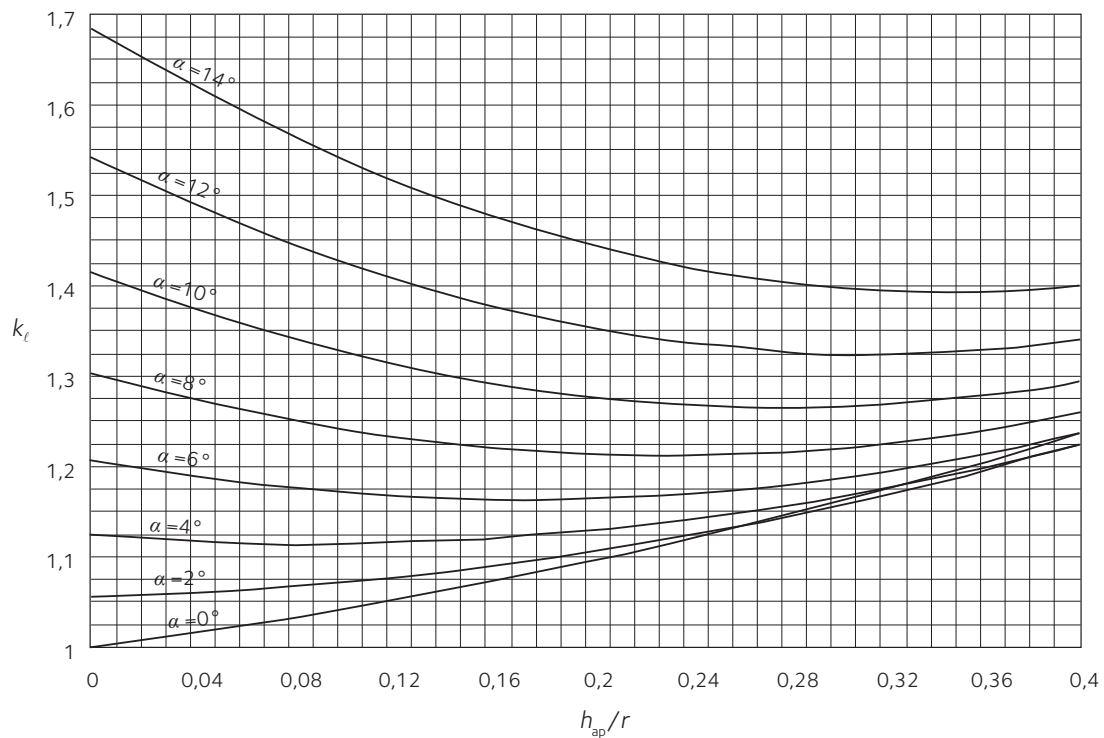


Figure 3.37: Factor k_ℓ according to Eurocode 5 for different radii of curvature, glulam class GL30c.

The bending strength of curved laminations should be reduced for taking into account the eigenstresses that occur when the laminations are bent during the manufacturing of the structural element. This can be done by multiplying the basic value of bending strength $f_{m,d}$ by a reduction factor k_r :

$$\sigma_{m,d} = k_r \cdot f_{m,d} \quad \text{Eq. 3.60}$$

The value of k_r decreases with decreasing ratio r_{in}/t (see table below)

Table 3.5 Reduction factor for bending strength k_r according to Eurocode 5 as a function of the ratio r_{in}/t , where r_{in} = inner radius of curvature and t = thickness of the lamination.

r_{in}/t	k_r
≥ 240	1
< 240	$0,76 + 0,001 \cdot r_{in}/t$

The design tensile stress perpendicular to the grain due to bending moment can be calculated as follows:

$$\sigma_{t,90,d} = k_p \cdot \frac{M_{ap,d}}{W_{ap}} = k_p \frac{6M_{ap,d}}{bh_{ap}^2} \quad \text{Eq. 3.61}$$

where:

- k_p is a factor determined by finite element analysis, defined as the ratio between perpendicular to grain stress and bending stress at the apex. Values of k_p for glulam GL30c are given in Figure 3.38.

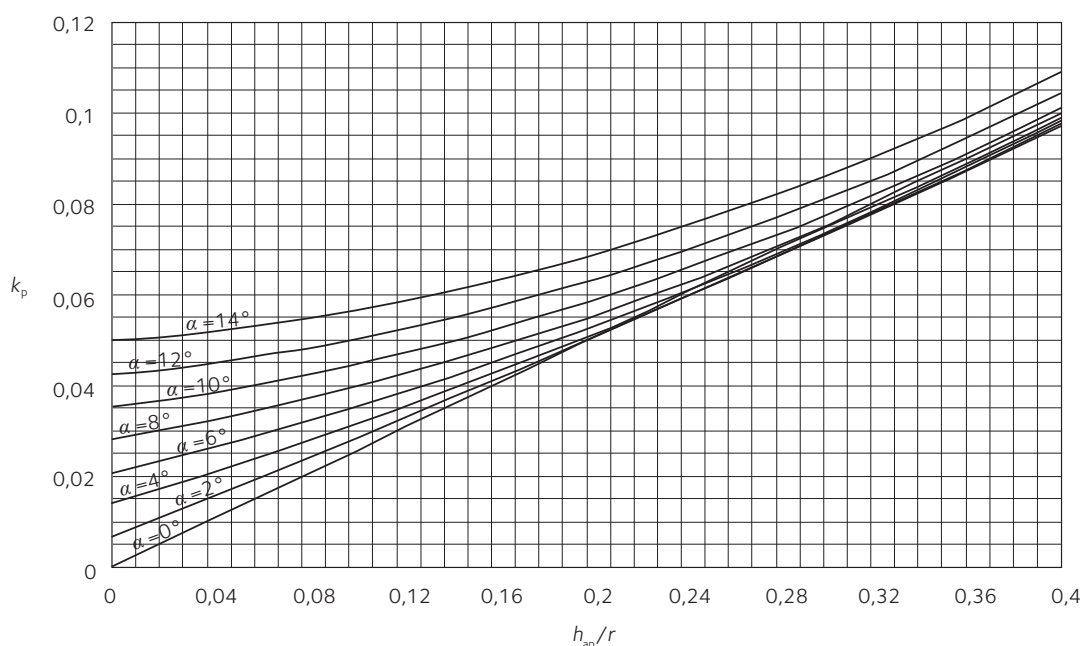


Figure 3.38: Factor k_p according to Eurocode 5 for different radii of curvature, glulam class GL30c.

The design tensile strength perpendicular to the grain shall be reduced then in the same manner as for tapered beams, see Equation 3.58.

3.4 Portal frames

Frame structures of timber are normally executed in glulam. The haunch can be made curved with continuous laminates, finger jointed, jointed with steel dowels and slotted-in plates, or built-up, see Figure 3.39.

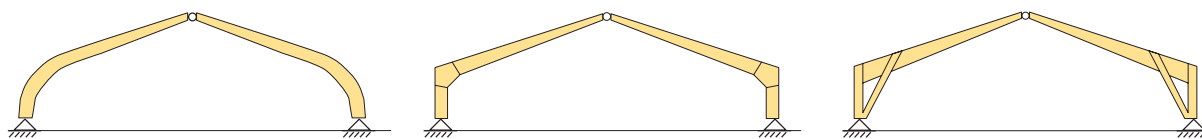


Figure 3.39: Examples of three-pin portal frames. a) Frame with curved haunches, b) frame with finger jointed haunches, c) built-up frame (knee braced frame).

The form of the frame should follow the funicular of the main load, as far as functional and aesthetic considerations permit. The slope of the roof should not be less than $14 - 15^\circ$ due to, among other reasons, the wish to reduce the deflection of the ridge.

Three-hinged portal frames are suitable for spans up to 30 – 40 m. If spans are greater, the two halves of the frame will be too large to transport in one piece. The two-hinged portal frame provides a stiffer structure but generally means that the frame must be manufactured and transported in three or more parts which are jointed with rigid joints on the site. Joints can suitably be placed at positions in the structure with small moments. Rigid joints typically demand more complicated workmanship than hinges and therefore command a higher price. They are in addition often highly visible in an undesirable way. The parts of the frame are on the other hand smaller than those in a corresponding three-hinged frame and therefore easier to transport.

Timber frames with one or no hinges are not usually employed for load-bearing structures. The three-pin portal frame is the commonest type. It is stable against horizontal forces in its own plane and statically determinate, which means that the moment distribution is not affected by uneven foundation settlement, moisture movements or by unforeseen deformations in joints and connections. Further, the three-hinged frame is hinged into the foundations, which simplifies their basic construction. In poor ground conditions the horizontal reactions at the supports can be taken up by tension members between the foundations (within or under the floor construction). The load on the substrate is then principally vertical. In normal cases, with roof slopes around 15° , the loading consists of self-weight and snow plus possible concentrated loads which are critical. In case of steep roof slopes, for example in churches or certain types of storage buildings, loading combinations together with wind can be critical. A preliminary, rough design can be carried out according to Figure 3.40.

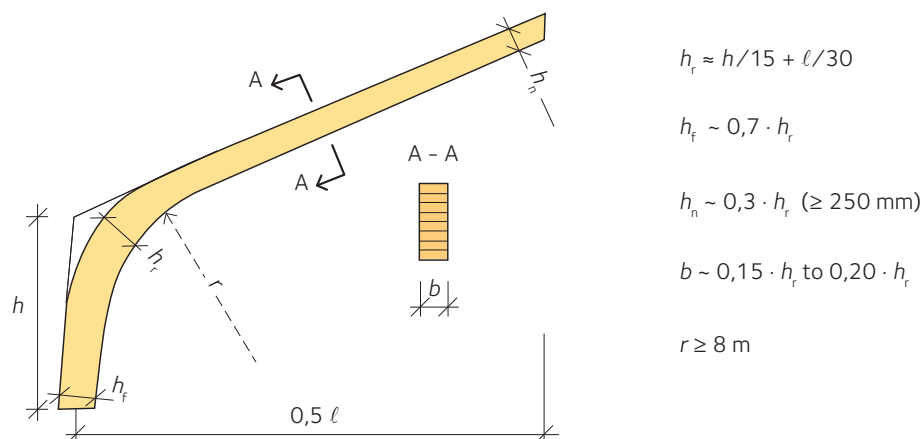


Figure 3.40: Preliminary sizing of a glulam portal frame.

The haunches of portal frames can be either curved or finger jointed.

Curved haunches are usually made with a constant cross-section. Radial tensile stresses (that is tensile stress perpendicular to the grain) must be checked if load combinations give positive moments (inner edge in tension) at the haunch – usually in combinations with wind load. Forces are calculated and checked in accordance with the instructions in Section 3.3.3.2. This means that a glued glulam haunch can be treated like a curved beam.

Finger jointed haunches are usually designed with a jointing piece, see Figure 3.41. The angle between the force and the grain at the joints should be limited, which is favourable for the loading capacity of the haunch. Tension stresses at the tapered side of members should be avoided. Therefore, for negative bending moments at the haunch– which is normally the most common case in practice – the tapered laminations should be placed at the underside of the frame, that is at the compression side (as shown in Figure 3.41). On the other hand, in situations with predominance of positive bending moments at the haunch, for example for portal frames with high roof slope and high wind loads, it could be advantageous to place the tapered side at the upper side of the frame.

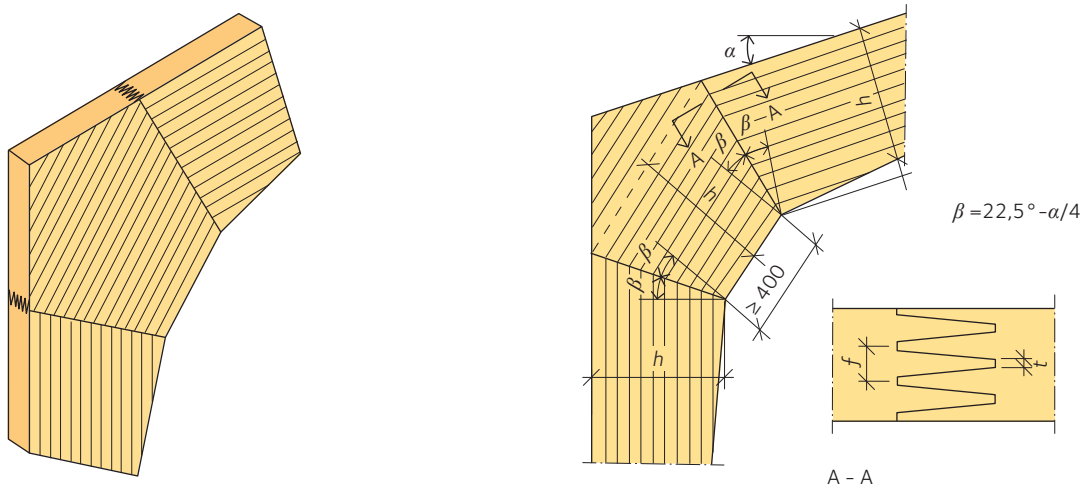


Figure 3.41: Finger jointed haunch with jointing piece.

The joints can be checked according to the following empirical method:

- Bending moment and normal force perpendicular to the joints is calculated at the centre of each joint;
- Effective cross-sectional area and effective section modulus at the joints are calculated.

With symbols as in the Figure 3.41, the following expressions apply:

$$A_{\text{joint}} = \frac{b \cdot h}{\cos \beta} \cdot \left(1 - \frac{t}{f}\right) \quad \text{Eq. 3.62}$$

$$W_{\text{joint}} = \frac{b \cdot h^2}{6 \cdot \cos^2 \beta} \cdot \left(1 - \frac{t}{f}\right) \quad \text{Eq. 3.63}$$

where:

- b is the width of the frame cross section
- t is the width of the tips in the finger joints
- f is the spacing of fingers centre to centre at base.

For most geometries of finger joints, it can be assumed that $(1-t/f) \approx 0,8$.

The design value of the compressive strength $f_{c,\beta}$ is determined from the angle β between longitudinal forces and the grain.

3.5 Arches

Timber arches are generally made of glulam, mainly due to the fact that glulam can be produced in curved forms and with varying depth without a great increase in price. As a rule, solid sections of constant depth are used, but composite sections of I or box form also occur, especially for large spans.

The form of the arch should be chosen so that the moments are as small as possible. As a rule, this means that the arch follows the thrust line (equilibrium polygon) of the dominating loading combination. The influence of moments can however not be avoided completely, as several load combinations must be taken into account, each with its own thrust line. As a compromise a parabola is often chosen. For functional reasons, for example to increase the headroom near the supports, an elliptical or other arch form may sometimes be preferable. The dividing line between frames and arches is fluid here. The same result can be achieved by placing the arch on columns, see Figure 3.42, left. The horizontal support reactions caused by the arch must in this case be taken care of by a tie rod between the springing points of the arch. When the arch rests directly on the ground floor slab, for example as in Figure 3.43 right, the horizontal forces can be taken up by the foundations if ground conditions permit, or by tie rods under the floor or cast into it. To limit the size of the horizontal reactions the rise of the arch should be equal or greater than 0,14 of its span. For a parabola this corresponds to an angle at the base α of 30° .

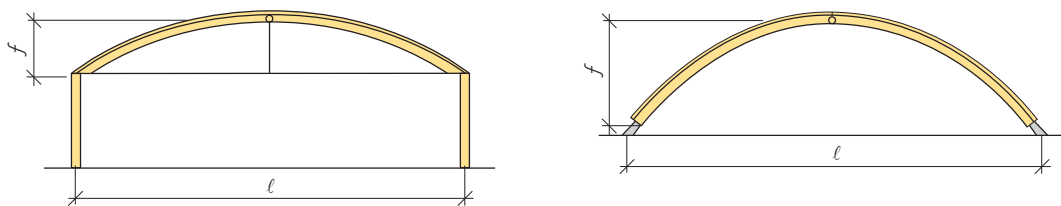


Figure 3.42: Left: Arch with tie rod, on columns. Right: Arch springing from foundations.

For more parts, which are joined rigidly on the site, hinges and rigid joints should be placed as in Figure 3.43. The choice between two- and three-hinged arches is made after similar considerations to those for frames. Three-hinged arches are thus preferable over spans of up to 60 – 70 m, while larger spans usually demand that the arch is manufactured and transported in three parts.



Figure 3.43: Suitable placing of joints in arch structures. Left: Hinge, Right: Rigid joints.

3.6 References

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Chapter 4

Design of timber joints

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4 Design of timber joints

Timber elements must be connected to each other to function as a system. Joints affect the structural behaviour of the elements for example by providing moment stiff connections or hinges. Joints between timber elements are also a strong contributor to the overall economy in a building system, mostly due to the time spent in producing joints. Joints in timber structures can be classified into one of the following groups:

- **Traditional timber joints:** were used in traditional timber construction when steel dowels or other steel details were unavailable. Dowels of stronger wood species such as oak could be used to stabilise the connection, which often was realised through cutting out parts of the timber members and fitting them together as a jig-saw puzzle. Traditional joints are produced by hand and are time-consuming to realise, therefore expensive. Furthermore, the load transfer is limited, especially tension forces, which narrows the scope to shortspan buildings and furniture. Restoration of older buildings is however an important area where knowledge of traditional joining techniques is valuable and there are numerous large historical timber buildings preserved.
- **Dowelled joints:** transfer forces through shear in mechanical fasteners mounted at an angle to the force direction. This is the most common fastener type for timber elements both in Sweden and in the world. Dowel-type connections can be designed to be ductile, which ensures a safe structure. Dowel-type connections involve:
 - Nails
 - Screws
 - Dowels
 - Nail plates (in combination with anchor nails/anchor screws) and punched metal plate fasteners
 - Bolts
- **Glued joints:** can be used to connect structural elements, but also to build up elements, for example glulam or LVL. Gluing requires a controlled environment in production, since many glues are affected by moisture content and temperature when setting. It is therefore not recommended to glue connections at a building site. Glued connections often show a brittle behaviour, even though the capacity might be high.

4.1 Dowel-types

Nails are smooth or rugged cylinders with a head, see Figure 4.1. The diameter is smaller than 8 mm. Nails can be driven directly into wood or the hole can be pre-drilled. In Eurocode 5 the terms smooth nails and other than smooth nails are being used. A smooth nail is a nail with a smooth shank without profiling, it can for instance be round, square or grooved nails. To other than smooth nails, all other nail types with a shank with some kind of profiling are counted, it can for instance be annular ring shanked, round jagged or twisted nails.



Figure 4.1: Examples of nails. a) Round, smooth nail, b) grooved, smooth nail (here a brad).

Screws are threaded cylinders with a head that usually do not require pre-drilling, see Figure 4.2. If the diameter of the screw is maximum 6 mm, it is theoretically treated as a nail.

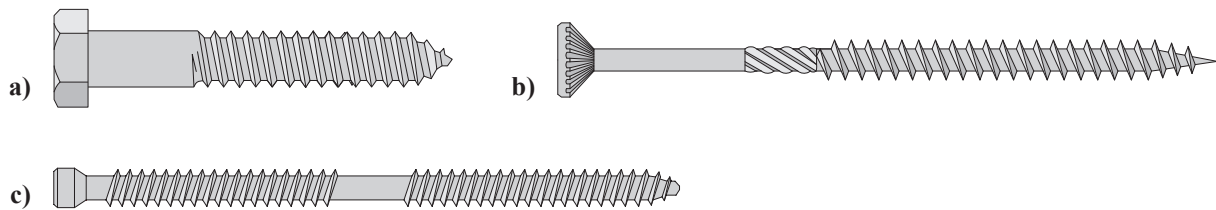


Figure 4.2: Examples of screws. a) Hexagon head wood screw requires pre-drilling and is however often replaced by for instance b) wood screw with countersunk head or c) double threaded wood screw.

Dowels are smooth (on rare occasions rugged) cylinders without a head, see Figure 4.3 respectively SS-EN 14592 for dowel-type connectors. According to Eurocode 5 dowels have a diameter 6 – 30 mm. All connectors to be used in timber structures must be CE marked by the supplier.

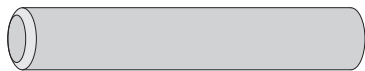


Figure 4.3: Example of dowel.

Nail plates are combined with anchor nails or anchor screws and have a pre-drilled pattern for the holes designed to automatically fulfil the code requirements on spacing, see Figure 4.4 a). Punched metal plate fasteners (SS-EN 14545) is a rationalisation of the nail plate with nails, where ‘nails’ are punched and folded out from a steel plate, Figure 4.4 b). The plate thickness is maximum 2 mm to enable folding of the nails. They are pressed into the wood under controlled conditions, for example when manufacturing timber trusses.

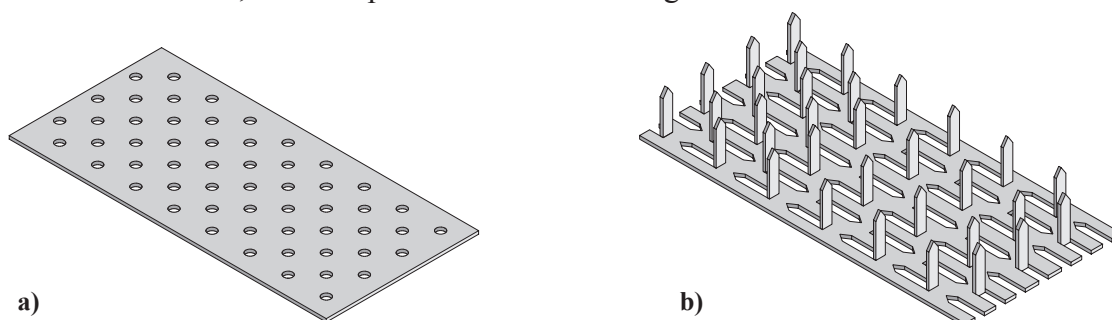


Figure 4.4: a) Nail plate, b) punched metal plate fastener.

Bolts are screws that usually have a hexagon head and that require pre-drilling, see Figure 4.5. The diameter is for construction purposes commonly at least 6 mm. Bolts often have a smooth shank and a threaded part nearby the nut.

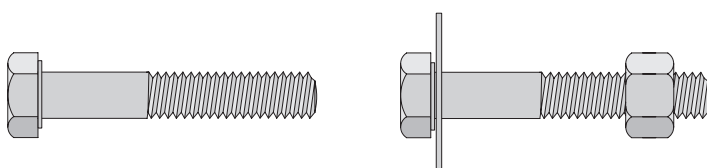


Figure 4.5: Bolt, often to be completed with washer and nut.

The material in dowel-type fasteners is most often steel, which can be treated to become rustproof or hardened. Steel is characterised by its ultimate strength, f_u and yield strength, f_y . A common system for classifying bolts in Sweden is to mark the fastener with a combination of numbers for example 8.8 is translated to $f_u = 800$ MPa and $f_y = 800 \times 0,8 = 640$ MPa.

Hereinafter a **dowel** referring to a dowel-type connector, such as nail, screw, steel dowel and bolt.

4.2 Shear capacity of single dowels

4.2.1 Dowel action

The fastener is mounted in an angle to the force direction, most often perpendicular to the same. When loaded, the dowel will press against the surrounding timber members, creating embedding pressure against the dowel, see Figure 4.6.

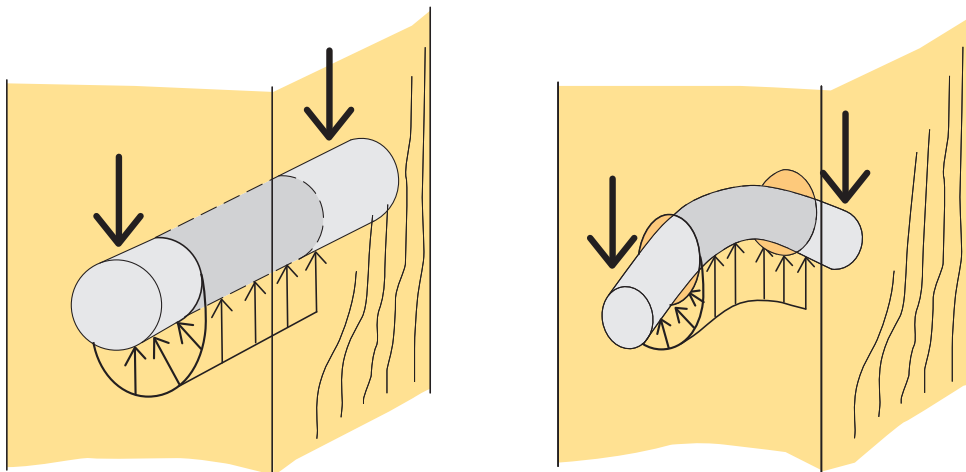


Figure 4.6: Stocky and slender dowel.

The dowel will act as a beam with a distributed load from the embedding pressure. If the dowel is stocky it will not bend, but if it is more slender it will deform in bending ultimately creating one or more plastic hinges in the dowel. Once deformed, the shear action can be complemented with a tensile action in the dowel. The tensile action can be increased by using bolts with a head and/or a nut to achieve anchorage, by mounting screws in an angle to the force direction to carry loads on the threads or by using screws or other dowels with a rough surface to increase withdrawal strength.

4.2.2 Material parameters

The load carrying capacity of a dowel-type connection in shear is determined by three parameters; the embedding strength of the timber f_h , the dowel strength represented by the yield moment M_y and the anchorage capacity enabling tensile action in the dowel F_{ax} .

4.2.2.1 Embedding strength

The embedding strength is the pressure the wood around the dowel can sustain. It is determined through a test made according to SS-EN 383, see Figure 4.7 a), where a stocky dowel is pressed against a wooden hole. The embedding strength is determined as the maximum force applied divided by the projected area of the dowel, $A = d \times t$ and is measured in MPa. A typical test result is shown in Figure 4.7 b).

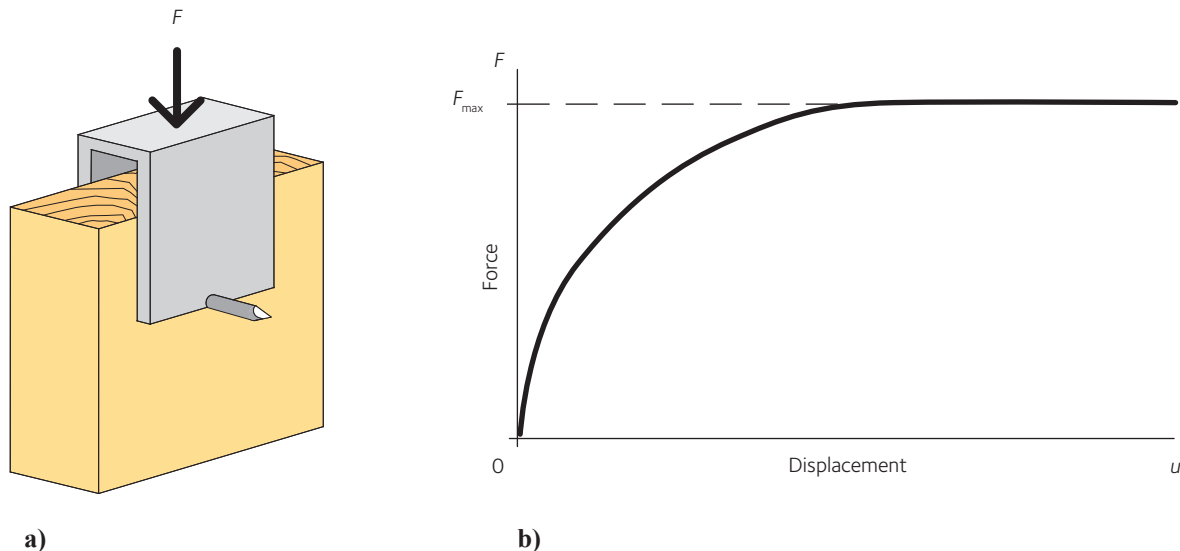


Figure 4.7: Embedding strength. a) Test set-up, b) typical test result.

The embedding strength itself is affected by several parameters:

- **The density of timber:** a higher density gives a higher embedding strength
- **The fastener diameter d :** small diameters yields higher embedding strength than larger diameters. The hole can be allowed to exceed the fastener diameter by 2 mm without any effect on the embedding strength, Blass (2003).
- **The angle α between grain and load direction:** the highest embedding strength is obtained in compression parallel to the grain and the lowest perpendicular to the grain.
- **The friction between dowel and timber:** dowels with a rough surface obtain higher embedding strength than those with a smooth surface.
- **The moisture content in wood:** as for all other strength properties, the moisture content affects the embedding strength negatively if it is high and positively if it is low
- **Any reinforcement of timber in tension perpendicular to the grain:** the failure in embedding is initiated by a crack along the grain caused by the dowel expanding the hole when pressing against the wood. It is thus the strength in tension perpendicular to the grain that limits the embedding strength. Any reinforcement in tension perpendicular to the grain will therefore increase the embedding strength.
- **If the hole is pre-drilled or not:** if the hole is pre-drilled, most of the load parallel to the grain will be carried through compression parallel to the grain, while holes that are not pre-drilled will give a load situation where the embedding strength is a mixture between compression parallel and perpendicular to the grain, see Figure 4.8.

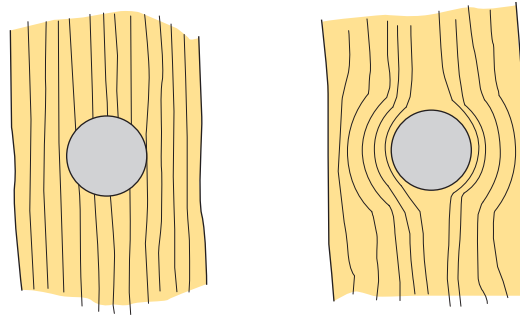


Figure 4.8: Load situation in a pre-drilled and not pre-drilled hole. The fibres are cut off upon predrilling while they are bent when the hole is not pre-drilled.

Empirical expressions for the determination of characteristic embedding strength for softwood capture characteristic density and the fastener diameter:

$$f_{h,0,k} = 0,082\rho_k d^{-0,3} \quad \text{without pre-drilling, that is } d < 8 \text{ mm} \quad \text{Eq. 4.1}$$

$$f_{h,0,k} = 0,082(1 - 0,01d)\rho_k \quad \text{with pre-drilling, all diameters} \quad \text{Eq. 4.2}$$

where $f_{h,0,k}$ is the characteristic embedding strength under loading parallel to the grain. Note that the density ρ_k should be inserted in kg/m^3 and the diameter d in mm in the Equations 4.1–4.2.

For nails the embedding strength does not vary to any larger extent with the angle between load and grain direction. For bolts the effect is larger and a reduction is made using Hankinson's formula if loading takes place at an angle α to the grain:

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \quad [\text{N/mm}^2] \quad \text{Eq. 4.3}$$

$$k_{90} = 1,35 + 0,015d$$

where $f_{h,\alpha,k}$ is the embedding strength under load directed in an angle α to the grain direction and d is the diameter of the fastener in mm.

k_{90} is here given for softwood. The embedding strength $f_{h,0,k}$ is calculated according to Equation 4.2. For other materials than softwood, such as particleboard, plywood and fibre-board, refer to Volume 2, Chapter 10, alternatively to Eurocode 5, Sections 8.3 and 8.5.

4.2.2.2 Yield moment

The yield moment is referred to as the plastic moment in steel design. It is the moment needed to produce a plastic hinge in a dowel and is determined through a test for nails according to SS-EN 408, see Figure 4.9, or by calculating the plastic moment for larger dowel diameters.

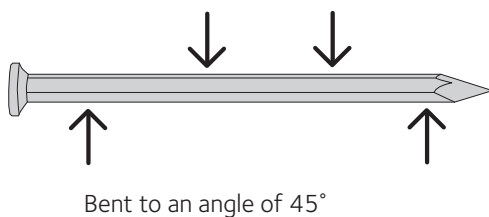


Figure 4.9: Determination of yield moment for nails.

The characteristic value of the yield moment $M_{y,Rk}$ of a dowel-type fastener is dependent on the fastener diameter d in mm and the material of the dowel represented by f_u the ultimate steel strength in N/mm². Empirical expressions for the determination of yield moment are given for nails and larger diameter dowels:

$$M_{y,Rk} = \frac{f_u}{600} 180d^{2,6} \text{ [Nmm]} \text{ for round nails} \quad \text{Eq. 4.4}$$

$$M_{y,Rk} = \frac{f_u}{600} 270d^{2,6} \text{ [Nmm]} \text{ for square and grooved nails} \quad \text{Eq. 4.5}$$

$$M_{y,Rk} = 0,3f_u d^{2,6} \text{ [Nmm]} \text{ for all fasteners with } d > 8 \text{ mm} \quad \text{Eq. 4.6}$$

For all other dowel-type fasteners testing must be performed according to SS-EN 14592, to determine $M_{y,Rk}$, alternatively acquired from the supplier.

4.3 Johansen theory – timber-timber joints and panel-timber joints

When a dowel joint is loaded in shear it can fail in a number of failure modes depending on the relation between the embedding strength, the yield moment of the dowel and the thickness of the timber members. The possible failure modes are displayed in Figure 4.10. The thicknesses are labelled 1 for the thinner member and 2 for the thicker member (2 for the inner and 1 for the outer member in joints with two shear planes). If the timber members are of different species, a relation between their embedding strengths is defined as:

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} [-] \quad \text{Eq. 4.7}$$

This situation can also occur if timber members of the same species are mounted at an angle to each other implying that the embedding strength may be different in the two members.

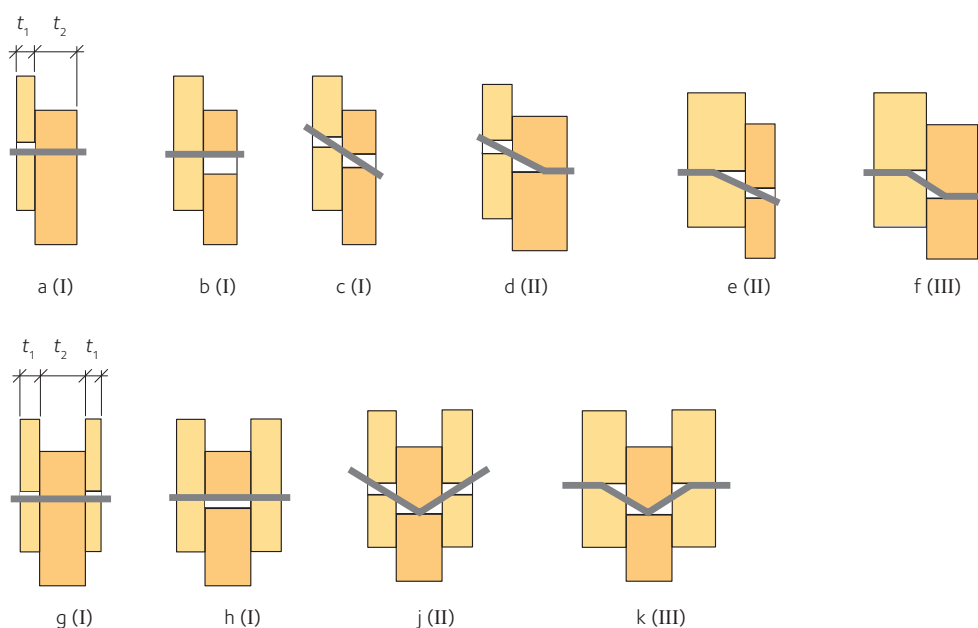


Figure 4.10: Possible failure modes in timber-to-timber joints, single shear in the top row, double shear in the bottom row.

The first theory to establish the load-carrying capacity for a dowelled joint was put forward by Johansen (1949). He identified three possible failure modes coupled to the number of plastic hinges forming in the dowel; failure mode I has no plastic hinges, failure mode II has one plastic hinge forming in the dowel and failure mode III has two plastic hinges, see Figure 4.10.

4.3.1 Failure mode I

Referring to Figure 4.10, failure modes marked with I are mode I failures. The resistance of failure modes a(I), b(I), g(I) and h(I) are calculated similarly to the embedding strength described in Section 4.2.2.1 and shown in Figure 4.7.

For failure mode Ia the following relation is valid, see Figure 4.11:

$$f_{v,Rk} = f_{h,1,k} t_1 d \tag{Eq. 4.8}$$

To determine $F_{v,Rk}$ for the second timber member, index 1 is replaced by index 2. The dowel remains straight and the timber fails due to the embedding pressure. This failure mode is common in bolted connections, but occurs very seldom in nailed joints.

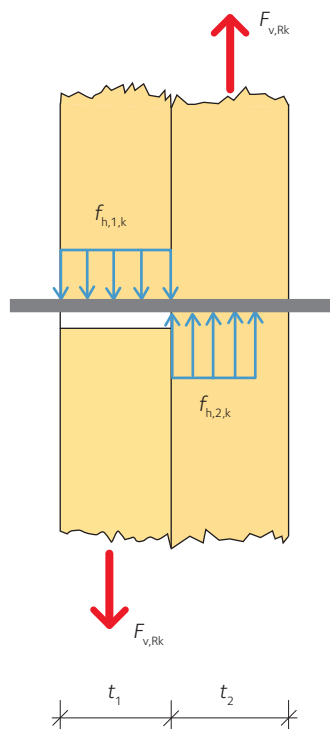


Figure 4.11: Embedding failure in mode I.

There is also a possibility that the dowel can remain straight but rotate. This occurs only in joints with a single shear plane (failure mode c(I)). To calculate the resistance of such a joint a moment equilibrium is set up.

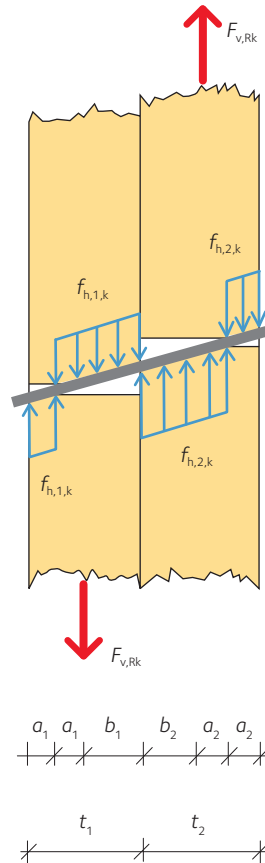


Figure 4.12: Mode I failure by rotation of the dowel.

Once again starting from the embedding failure:

$$f_{v,Rk} = f_{h,1,k}db_1 = f_{h,2,k}db_2 = \beta f_{h,1,k}db_2$$

$$b_1 = \beta b_2$$

Equilibrium at the interface gives:

$$M_{\text{left}} = f_{h,1,k}d \left(\frac{b_1^2}{2} + a_1 \left(b_1 + \frac{a_1}{2} \right) - a_1 \left(b_1 + \frac{3a_1}{2} \right) \right) = \dots = f_{h,1,k}d \left(\frac{b_1^2}{2} - a_1^2 \right)$$

$$M_{\text{right}} = f_{h,2,k}d \left(a_2^2 - \frac{b_2^2}{2} \right) = \beta f_{h,1,k}d \left(a_2^2 - \frac{b_2^2}{2} \right)$$

Equalling and replacing $b_2 = b_1/\beta$ leads to:

$$\frac{b_1^2}{2} \frac{1+\beta}{\beta} = \beta a_2^2 + a_1^2$$

Expressing $a_1 = (t_1 - b_1)/2$ and $a_2 = (t_2 - b_2)/2$ and substituting a second order equation remains:

$$b_1^2 \left(\frac{1+\beta}{\beta} \right) + 2b_1(t_1 + t_2) - (t_1^2 + \beta t_2^2) = 0$$

The solution for b_1 is entered in the expression for $F_{v,Rk}$:

$$F_{v,Rk} = f_{h,1,k} db_1 = \frac{f_{h,1,k} dt_1}{1 + \beta} \left(\sqrt{\beta + 2\beta^2 \left(1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1} \right)^2 \right)} + \beta^3 \left(\frac{t_2}{t_1} \right)^2 - \beta \left(1 + \frac{t_2}{t_1} \right) \right) \quad \text{Eq. 4.9}$$

In a situation where $\beta = 1$ and the timber members have the same thicknesses, the equation is simplified to:

$$F_{v,Rk} = f_{h,k} dt (\sqrt{2} - 1) \quad \text{Eq. 4.10}$$

In conclusion, rotation of the dowel is the worst case for failure mode I when the timber members have the same thickness.

4.3.2 Failure mode II

If the thickness of one of the timber members is large enough, the dowel will bend inside the timber, forming a plastic hinge, see Figure 4.13. Failure mode II refers to modes d, e, and j in Figure 4.10.

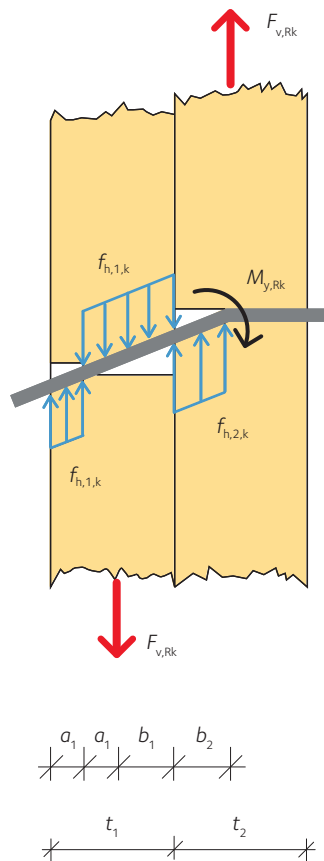


Figure 4.13: Failure mode II.

Recognizing that the moment has its maximum in the plastic hinge, the shear force must equal zero at the plastic hinge:

$$F_{v,Rk} = f_{h,1,k} db_1 = f_{h,2,k} db_2 = \beta f_{h,1,k} db_2$$

$$b_1 = \beta b_2$$

Moment equilibrium at the hinge:

$$M_{y,Rk} = -f_{h,2,k} d \frac{b_2^2}{2} + f_{h,1,k} d (b_1 + a_1) \left(b_2 + \frac{b_1 + a_1}{2} \right) - f_{h,1,k} d a_1 \left(b_1 + b_2 + \frac{3a_1}{2} \right)$$

Substituting $f_{h,2,k} = \beta f_{h,1,k}$, $b_2 = b_1/\beta$ and $a_1 = (t_1 - b_1)/2$ leads to a second order equation for b_1 . Inserting the solution into $F_{v,Rk}$ in the same way as for failure mode I gives:

$$F_{v,Rk} = \frac{f_{h,1,k} dt_1}{2 + \beta} \left(\sqrt{2\beta(1 + \beta) + \frac{4\beta(2 + \beta)M_{y,Rk}}{f_{h,1,k} dt_1^2}} - \beta \right) \quad \text{Eq. 4.11}$$

If $\beta = 1$ ($f_{h,1,k} = f_{h,2,k} = f_{h,k}$), the expression is simplified to:

$$F_{v,Rk} = \frac{f_{h,k} dt_1}{3} \left(2 \sqrt{1 + \frac{3M_{y,Rk}}{f_{h,k} dt_1^2}} - 1 \right)$$

If instead the timber member on the left-hand side is thicker, the expression for the resistance will end up in:

$$F_{v,Rk} = \frac{f_{h,1,k} dt_2}{1 + 2\beta} \left(\sqrt{2\beta^2(1 + \beta) + \frac{4\beta(1 + 2\beta)M_{y,Rk}}{f_{h,1,k} dt_2^2}} - \beta \right) \quad \text{Eq. 4.12}$$

$$\beta = 1 \Rightarrow$$

$$F_{v,Rk} = \frac{f_{h,k} dt_2}{3} \left(2 \sqrt{1 + \frac{3M_{y,Rk}}{f_{h,k} dt_2^2}} - 1 \right)$$

When $\beta = 1$, it is seen that Equations 4.11 and 4.12 describe the same resistance.

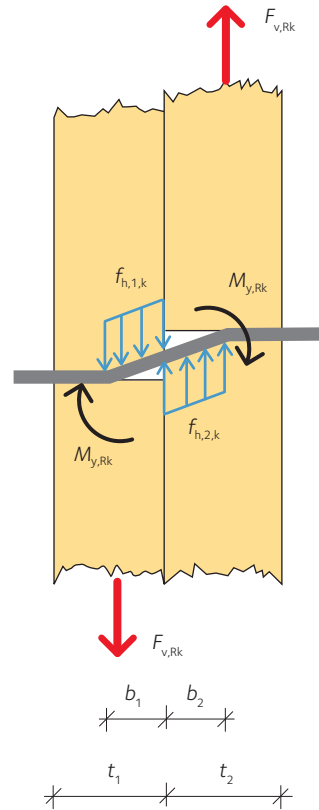


Figure 4.14: Failure mode III.

4.3.3 Failure mode III

The most favourable failure mode in terms of ductility is failure mode III, where the dowel is bent in both timber members (mode f and k). To derive the equation for the resistance a moment equilibrium is set up for the inclined part of the dowel, given that the shear force in the dowel is zero in both of the plastic hinges:

$$M_{y,Rk} + M_{y,Rk} = f_{h,1,k} db_1 \left(b_2 + \frac{b_1}{2} \right) - \beta f_{h,1,k} d \frac{b_2^2}{2}$$

$$b_2 = \frac{b_1}{\beta} \Rightarrow b_1 = \sqrt{\frac{2M_{y,Rk}}{f_{h,1,k} d}} \sqrt{\frac{2\beta}{1+\beta}}$$

$$F_{v,Rk} = f_{h,1,k} \cdot d \cdot b_1 = \sqrt{\frac{2\beta}{1+\beta}} \sqrt{2M_{y,Rk} f_{h,1,k} d} \quad \text{Eq. 4.13}$$

Using $\beta = 1$, the equation is reduced to:

$$F_{v,Rk} = \sqrt{2M_{y,Rk} f_{h,1,k} d}$$

4.3.4 Design situation

All resistances for all failure modes must be established, whereafter the lowest value for the particular geometry is taken as the resistance. As an example, the resistances for a nailed joint with grooved nail with a diameter of 4 mm and a timber member thickness on the heads side of $7d = 28$ mm is presented in Figure 4.15, as a function of the thickness of the second member.

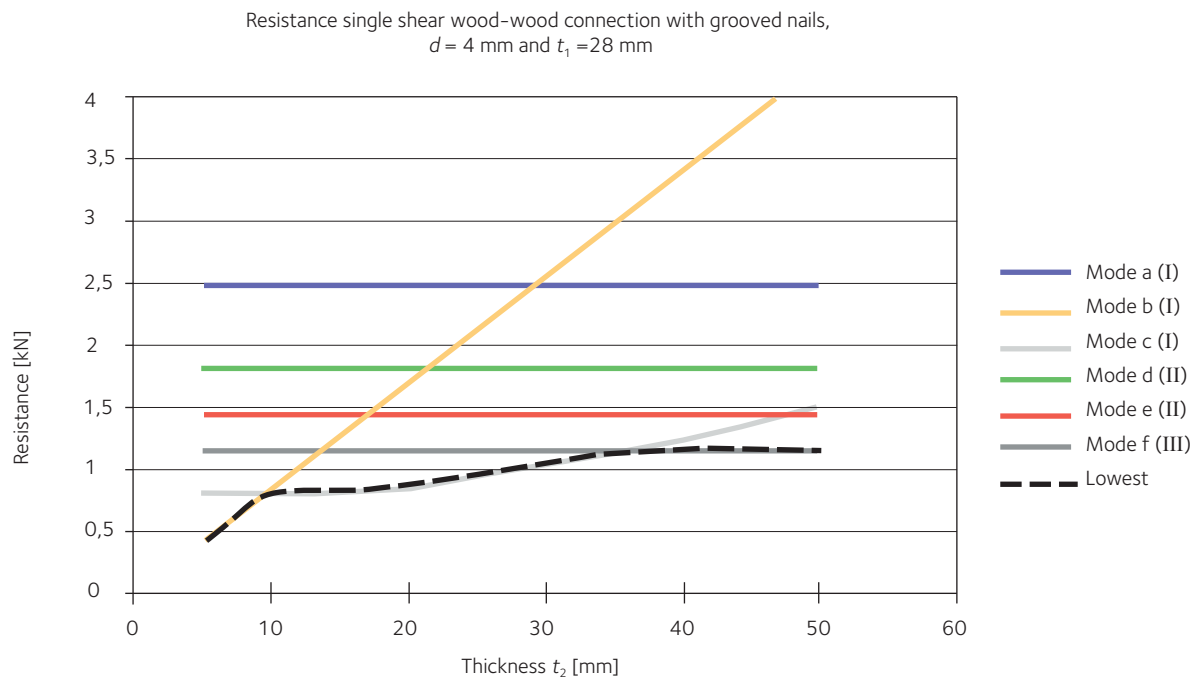


Figure 4.15: Resistances in different failure modes for timber-to-timber single shear joint.

As the thickness of the timber member on the point-side increases, the failure mode changes from mode b (I) to c (I) and finally mode e (II). To reach mode f (III), the timber member on the headsides additionally needs to be made thicker. A full presentation of resistances for failure modes in single shear is assembled in Table 4.1. This set-up is fully consistent with Eurocode 5. Unlike the theoretical derivation described above and which is based on Blass (2003), the formulas in Table 4.1 in some cases have been adjusted for correspondence with test results.

4.3.5 Double shear timber-to-timber joints

In double shear joints (modes g–k in Figure 4.10) rotation of the dowel is restrained due to symmetry in the loading situation. Otherwise, the same approach to determine the resistance can be used as for single shear joints and failure modes I, II and III can all occur. The formulas for the resistances are presented in Table 4.2. This set-up is fully consistent with Eurocode 5. Unlike the theoretical derivation described above and which is based on Blass (2003), the formulas in Table 4.2 in some cases have been adjusted for correspondence with test results.

4.4 Steel-to-timber joints

Using a steel plate as one of the members in a timber joint is very common. Theoretically, this has the consequence that the forming of the plastic hinge is always located at the timber-steel interface. This increases the capacity of the joint compared to timber-to-timber connections. There is a condition on the thickness of the steel plate t_{steel} to enable the forming of a plastic hinge:

$$t_{\text{steel}} \geq d \rightarrow \text{fixed support}$$

$$t_{\text{steel}} \leq 0,5d \rightarrow \text{pinned support}$$

If the thickness of the steel plate, t_{steel} is larger than or equal to the diameter of the dowel, it is stiff enough to provide a fixed support for the forming of a plastic hinge. If the thickness is less than or equal to half the diameter of the dowel, the support from the steel plate can be regarded as pinned and the dowel will merely rotate in the hole without the formation of a plastic hinge. The resistance of the joint is higher if a plastic hinge forms. Interpolation between resistance formulas is permitted if needed. A full presentation of formulas for the resistance is found in Tables 4.3 – 4.4. An example of failure modes and resistances is given in Figure 4.16.

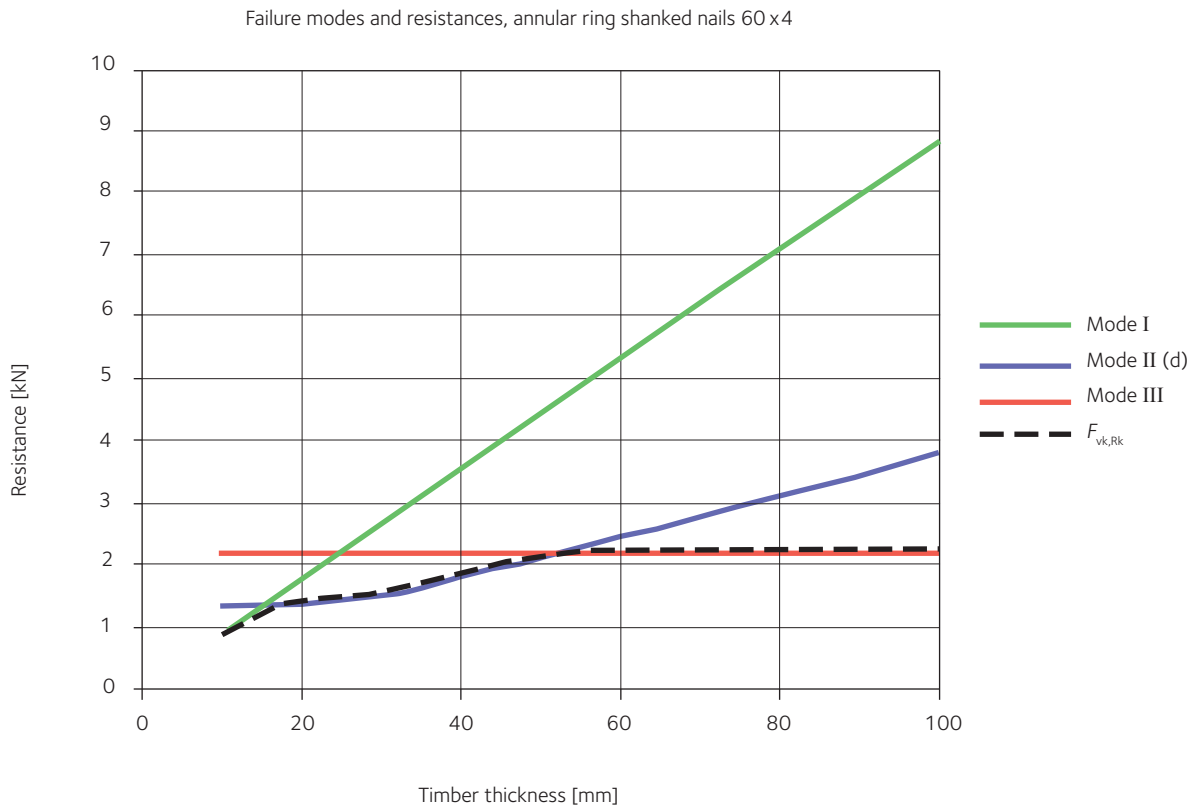


Figure 4.16: Resistances and failure modes for single shear steel-to-timber joint with fixed support of the nail from the steel plate.

4.4.1 Slotted-in steel plates

Steel plates are very efficient in carrying loads in joints, but they are not always a good solution when there is a risk for fire since the steel plates early on in the fire becomes very hot and lose their strength. Steel plates in structures subject to fire must therefore be protected e.g. by fire proof paint or coverage with wood. A solution is therefore to hide the steel plates inside the timber member from the beginning, which is often also preferred from the point of view of appearance. This is done by making slots into the timber member and inserting the steel plates. Holes for dowels are pre-drilled both in wood and steel and the dowels are inserted in the holes to complete the connection, see Figure 4.17.

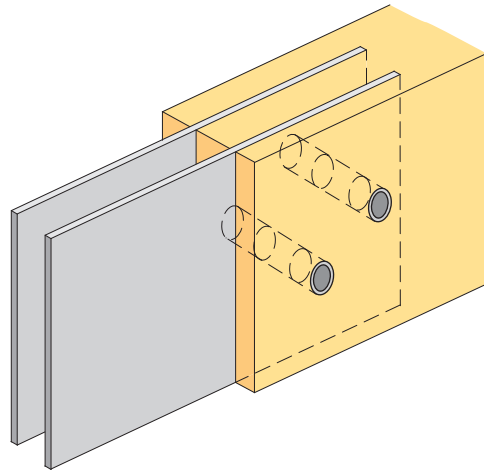


Figure 4.17: Joint with slotted-in steel plates.

The resistance of a joint with slotted-in steel plates is derived in the same manner as described for timber-to-timber joints. The only difference is the automatic location of the plastic hinges, which will be placed at the steel-timber interface. Furthermore, the condition for the thickness of the steel plate need not be fulfilled. A plastic hinge will form regardless of the thickness of the steel plate due to symmetry in the loading situation. However, the steel plate must be made thick enough to withstand the embedding pressure from the dowel. The resistances for joints with slotted-in steel plates are given in Table 4.5.

4.4.2 Double shear steel-to-timber joints

In double shear connections it is common to replace both the outer members with steel plates, see Figure 4.18. This can be done in several ways. One possibility is to have nails that end within the timber member, which makes the joint a single shear steel-to-timber joint. It is also possible to have the dowel protrude all the way through the timber member and both steel plates, see Figure 4.18. This is then a double shear steel-to-timber joint, for which the resistances are presented in Tables 4.6 – 4.7.

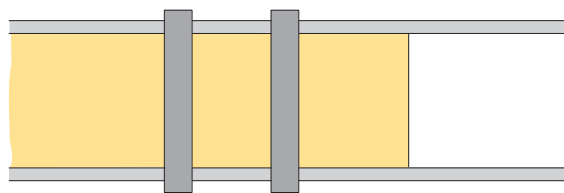


Figure 4.18: Double shear steel-to-timber joint.

The production of a double shear steel-to-timber joint is most easily accomplished if both steel and timber can be pre-drilled in the same operation. Special fasteners have been developed to accomplish this, both for slotted-in steel plates and multiple shear steel-to-timber joints, see Figure 4.19.



Figure 4.19: Special fastener designed for steel-to-timber joints enabling drilling the hole and mounting the fastener in one working moment.

4.5 Expressions for the resistance of a single dowel

Tables 4.1 – 4.7 express the resistance per fastener per shear plane. If for example two shear planes are present, the calculated value must be multiplied by 2. $\beta = f_{h,2,k}/f_{h,1,k}$. The presentation can also be found in Eurocode 5, Chapter 8.

Table 4.1 Single shear timber-to-timber joint.

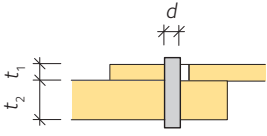
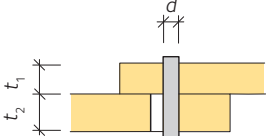
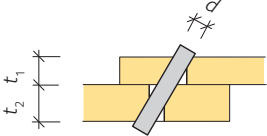
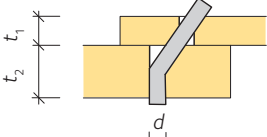
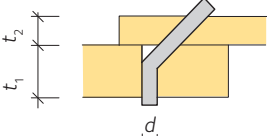
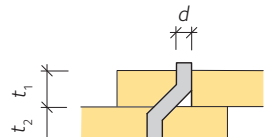
$F_{v,Rk} = f_{h,1,k} t_1 d$	
$F_{v,Rk} = f_{h,2,k} t_2 d$	
$F_{v,Rk} = \frac{f_{h,1,k} t_1 d}{1 + \beta} \left(\sqrt{\beta + 2\beta^2 \left(1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1} \right)^2 \right) + \beta^3 \left(\frac{t_2}{t_1} \right)^2} - \beta \left(1 + \frac{t_2}{t_1} \right) \right) + \frac{F_{ax,Rk}}{4}$	
$* F_{v,Rk} = 1,05 \frac{f_{h,1,k} t_1 d}{2 + \beta} \left(\sqrt{2\beta(1 + \beta) + \frac{4\beta(2 + \beta) M_{y,Rk}}{f_{h,1,k} d t_1^2}} - \beta \right) + \frac{F_{ax,Rk}}{4}$	
$* F_{v,Rk} = 1,05 \frac{f_{h,1,k} t_2 d}{1 + 2\beta} \left(\sqrt{2\beta^2(1 + \beta) + \frac{4\beta(1 + 2\beta) M_{y,Rk}}{f_{h,1,k} d t_2^2}} - \beta \right) + \frac{F_{ax,Rk}}{4}$	
$* F_{v,Rk} = 1,15 \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2 M_{y,Rk} f_{h,1,k} d} + \frac{F_{ax,Rk}}{4}$	

Table 4.2 Double shear timber-to-timber joints.

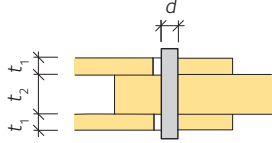
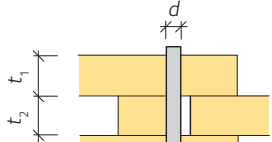
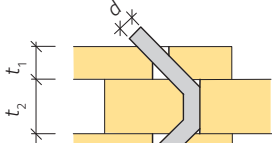
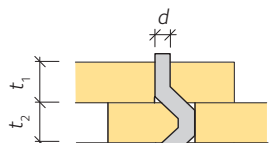
$F_{v,Rk} = f_{h,1,k} t_1 d$	
$F_{v,Rk} = 0,5 f_{h,2,k} t_2 d$	
$* F_{v,Rk} = 1,05 \frac{f_{h,1,k} t_1 d}{2 + \beta} \left(\sqrt{2\beta(1 + \beta) + \frac{4\beta(2 + \beta) M_{y,Rk}}{f_{h,1,k} d t_1^2}} - \beta \right) + \frac{F_{ax,Rk}}{4}$	
$* F_{v,Rk} = 1,15 \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2 M_{y,Rk} f_{h,1,k} d} + \frac{F_{ax,Rk}}{4}$	

Table 4.3 Single shear steel-to-timber joints, where $t_{steel} \geq d$ (thick steel plate).

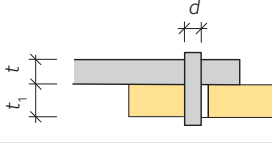
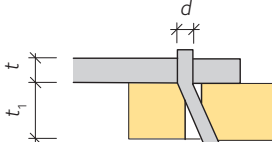
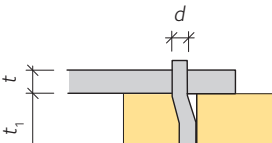
$F_{v,Rk} = f_{h,k} t_1 d$	
$* F_{v,Rk} = f_{h,k} t_1 d \left(\sqrt{2 + \frac{4 M_{y,Rk}}{f_{h,k} d t_1^2}} - 1 \right) + \frac{F_{ax,Rk}}{4}$	
$* F_{v,Rk} = 2,3 \sqrt{M_{y,Rk} f_{h,k} d} + \frac{F_{ax,Rk}}{4}$	

Table 4.4 Single shear steel-to-timber joints, where $t_{\text{steel}} \leq 0,5d$ (thin steel plate).

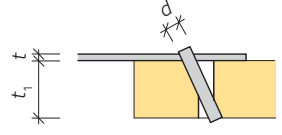
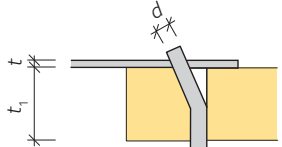
$F_{v,Rk} = 0,4 f_{h,k} t_1 d$	
$* F_{v,Rk} = 1,15 \sqrt{2 M_{y,Rk} f_{h,k} d} + \frac{F_{ax,Rk}}{4}$	

Table 4.5 Slotted-in steel plates.

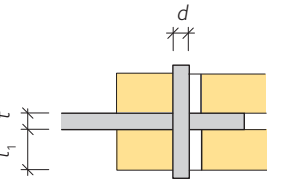
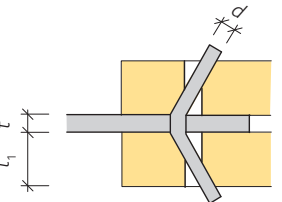
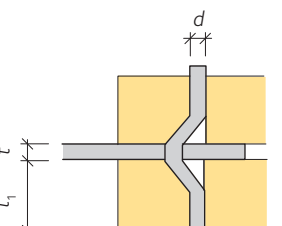
$F_{v,Rk} = f_{h,1,k} t_1 d$	
$* F_{v,Rk} = f_{h,1,k} t_1 d \left(\sqrt{2 + \frac{4 M_{y,Rk}}{f_{h,1,k} d t_1^2}} - 1 \right) + \frac{F_{ax,Rk}}{4}$	
$* F_{v,Rk} = 2,3 \sqrt{M_{y,Rk} f_{h,1,k} d} + \frac{F_{ax,Rk}}{4}$	

Table 4.6 Double shear steel-to-timber joints where $t_{\text{steel}} \geq d$ (thick steel plates).

$F_{v,Rk} = 0,5 f_{h,2,k} t_2 d$	
$* F_{v,Rk} = 2,3 \sqrt{M_{y,Rk} f_{h,2,k} d} + \frac{F_{ax,Rk}}{4}$	

Table 4.7 Double shear steel-to-timber joints where $t_{\text{steel}} \leq 0,5d$ (thin steel plates).

$F_{v,Rk} = 0,5 f_{h,2,k} t_2 d$	
$* F_{v,Rk} = 1,15 \sqrt{2 M_{y,Rk} f_{h,2,k} d} + \frac{F_{ax,Rk}}{4}$	

4.6 Tensile capacity of single dowels – rope effect

In all failure modes in shear where the dowel is bent (marked with * in Tables 4.1 – 4.7), some part of the load uptake also occurs in tension. Depending on the surface and end anchorage of the dowel, the part carried in tension can be larger or smaller. The surface of the dowel can have a higher anchorage resistance F_{ax} due to:

- twisted dowels
- annular rings
- threading (the dowel is then a screw or a bolt).

The anchorage of the dowel can be enhanced by:

- washers and nuts on the headside
- washers and nuts on the pointside.

The contribution of tension to the shear capacity of a single dowel can be substantial. Kuipers and Van Der Put (1982) showed that threading can increase the resistance of a joint by as much as 2,6 times the shear capacity calculated through Tables 4.1 – 4.7, omitting the second term in the right part of the formulas. The effect of tensile action can be determined either by empirical formulas or by testing. Since the empirical expressions are derived for a multitude of cases, testing is suggested if a particular joint is to be used repeatedly.

4.6.1 Eurocode 5 application

In Eurocode 5, the rope effect is taken into account by adding the term $F_{ax,Rk}/4$ to the expression for the shear capacity of a single dowel according to Section 4.4.

The contribution from the rope effect is limited to given percentages of the shear capacity (Tables 4.1 – 4.7) as presented in Table 4.8.

Table 4.8 Maximum contribution from rope effect in relation to the shear capacity of a single dowel-type fastener.

Fastener type	Percentage
Round nails	15 %
Square and grooved nails	25 %
Other nails	50 %
Screws	100 %
Bolts	25 %
Dowels	0 %

4.6.2 Determination of tensile capacity of single fasteners

4.6.2.1 Nails

For nails, the capacity in tension is dependent on the surface roughness along the nail and the anchorage capacity of the nail head. These can be characterised through f_{ax} and f_{head} respectively, which can be determined through tests according to SS-EN 1382, SS-EN 1383 and/or SS-EN 14358. If these strengths are not known, empirical expressions exist to establish them for smooth nails (in other cases the nail is tested according to SS-EN 14592):

$$f_{ax,k} = 20 \cdot 10^{-6} \rho_k^2 \quad \text{Eq. 4.14}$$

$$f_{head,k} = 70 \cdot 10^{-6} \rho_k^2 \quad \text{Eq. 4.15}$$

Expressions for calculating the withdrawal capacity $F_{ax,Rk}$ for the nail, simply based on the surface area of the nail or the head, are then given as:

$$F_{ax,Rk} = \min \begin{cases} f_{ax,k} dt_{pen} \\ f_{head,k} d_h^2 \end{cases} \quad (\text{other than smooth nails}) \quad \text{Eq. 4.16}$$

$$F_{ax,Rk} = \min \begin{cases} f_{ax,k} dt_{pen} \\ f_{ax,k} dt + f_{head,k} d_h^2 \end{cases} \quad (\text{smooth nails}) \quad \text{Eq. 4.17}$$

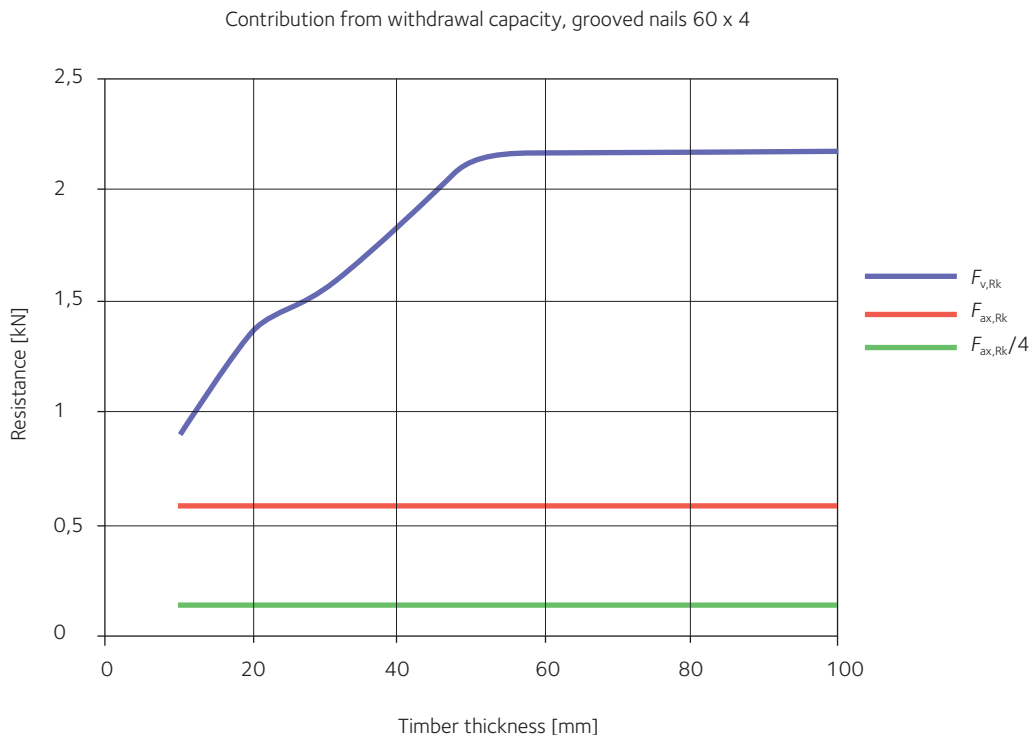


Figure 4.20: Withdrawal and shear capacity of grooved nails. $F_{v,Rk}$ is the capacity in shear and $F_{ax,Rk}$ the capacity in tension. The total capacity is the sum $F_{v,Rk} + F_{ax,Rk}/4$. The joint is of the same type as in Figure 4.16.

where t_{pen} is the penetration depth and d_h the diameter of the nail head. (For annular ring shanked nails t_{pen} is the penetration depth for the threaded part). Conditions apply to the thickness of the members, see Eurocode 5, 8.3.2(7). Permanent axial loads are not allowed for smooth nails. In Figure 4.20, the addition of the withdrawal capacity to the shear capacity is shown for grooved nails.

4.6.2.2 Bolts

For bolts, the withdrawal capacity is dependent on the anchorage capacity of washers and nuts and the tensile capacity of the bolt itself. When determining the anchorage capacity of the washer, the characteristic compression strength perpendicular to grain for wood can be taken as three times the characteristic base value due to concentrated loading:

$$F_{\text{ax,washer,Rk}} = 3f_{\text{c},90,\text{k}}A_{\text{washer}} \quad \text{Eq. 4.18}$$

If an entire steel plate with thickness t_{steel} is used instead of washers, A_{washer} should be replaced with a circular area having the diameter:

$$D = \min \begin{cases} 12t_{\text{steel}} \\ 4d \end{cases}$$

where d is the bolt diameter.

4.6.2.3 Screws

The withdrawal capacity of screws is dependent on the threading and can reach very high values. The withdrawal strength can be determined as:

$$f_{\text{ax,k}} = 0,52d^{-0,5}\ell_{\text{ef}}^{-0,1}\rho_{\text{k}}^{0,8} \quad d \text{ and } \ell_{\text{ef}} \text{ [mm]}, \rho_{\text{k}} \text{ [kg/m}^3\text{]} \quad \text{Eq. 4.19}$$

where ℓ_{ef} is the length of the threaded part embedded in wood. The characteristic withdrawal capacity of a connection with screws is established as:

$$F_{\text{ax},\alpha,\text{Rk}} = \frac{n^{0,9}f_{\text{ax,k}}d\ell_{\text{ef}}k_{\text{d}}}{1,2\cos^2\alpha + \sin^2\alpha} \quad \text{Eq. 4.20}$$

where α is the angle between the screw axis and the grain ($\alpha \geq 30^\circ$) and k_{d} is $\min(d/8;1)$ with d in mm, and n is the number of screws acting together in the connection. The outer thread diameter d must be $6 \leq d \leq 12$ mm and $0,6 \leq d_1/d \leq 0,75$ where d_1 is the inner thread diameter.

The contribution from the withdrawal capacity for bolts (with washer and nut) and screws is shown in Figure 4.21.

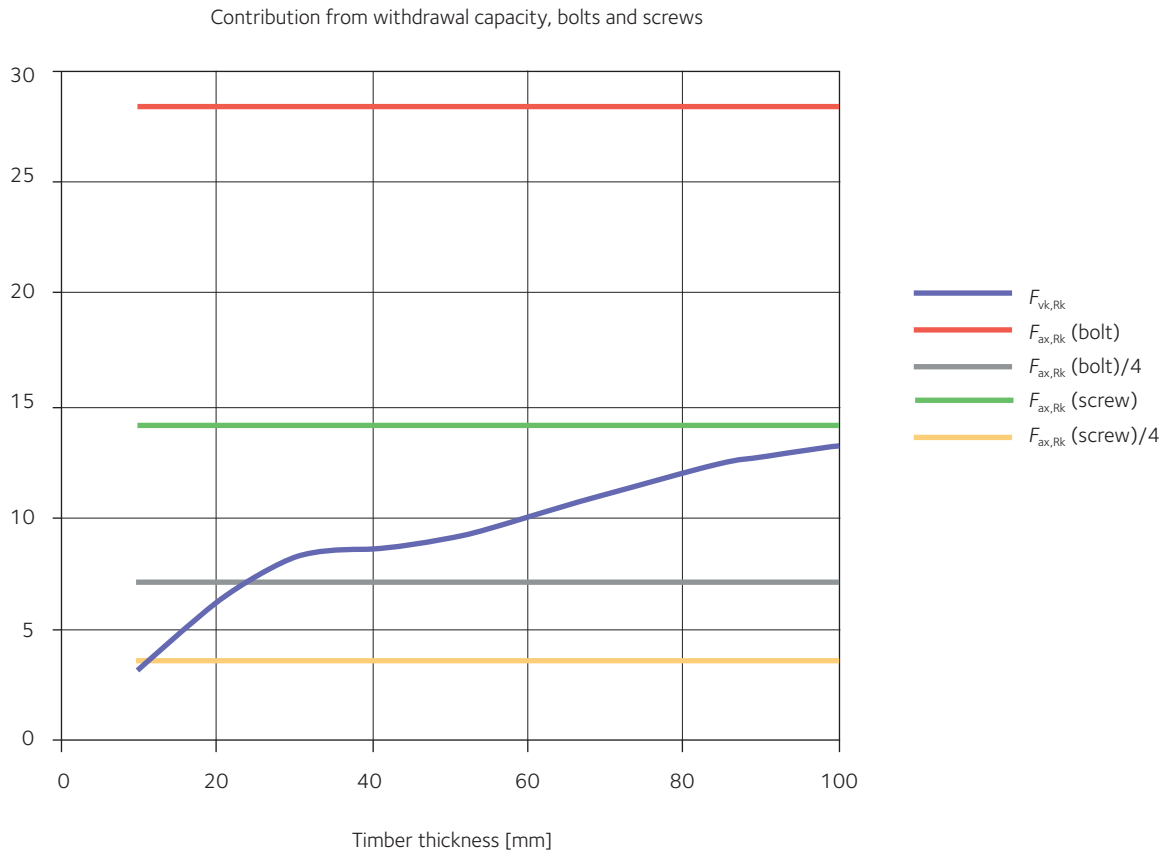


Figure 4.21: Shear and withdrawal capacity for bolts and screws. The absolute value of the withdrawal capacity is not equal for bolts and for screws. The total capacity of the bolt respectively the screw in shear is taken as $F_{v,Rk} + F_{ax,Rk}/4$.

4.7 Combined loading

If a joint is loaded with simultaneous shear load and axial load, the combination of these loads must be taken into account in design. The following interaction relations can then be used:

$$\frac{F_{ax,Ed}}{F_{ax,Rd}} + \frac{F_{v,Ed}}{F_{v,Rd}} \leq 1 \quad (\text{smooth nails}) \quad \text{Eq. 4.21}$$

$$\left(\frac{F_{ax,Ed}}{F_{ax,Rd}} \right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}} \right)^2 \leq 1 \quad (\text{other types of nails and screws}) \quad \text{Eq. 4.22}$$

4.8 Joints depending solely on tensile capacity

The tensile capacity of screws is so large that it can be beneficial to mount them in such a manner that they are loaded only in tension or compression. This can be accomplished by using the analogy with a truss, where shear forces are carried through a force couple in tension and compression. Examples of applications are shown in Figure 4.22.

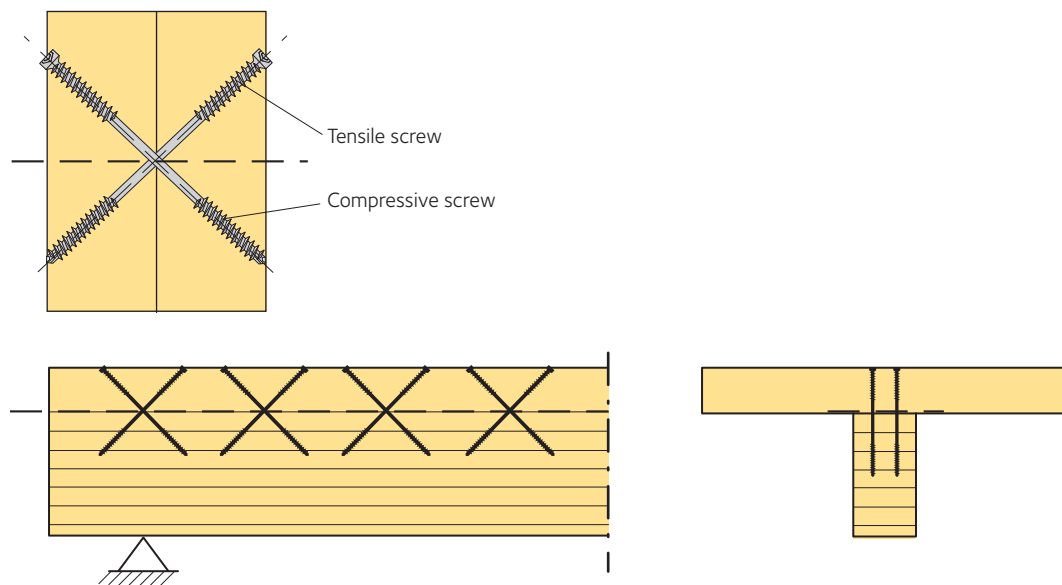


Figure 4.22: Screws loaded solely in tension or compression, Blass (2003).

The capacity of the screws is determined only with the use of the tensile capacity described in Section 4.6.

4.9 Brittle failure modes and group effects in dowelled joints

A joint is considered to exist if at least two fasteners are present in a connection. Oftentimes, several fasteners placed in a group are needed to carry the load in the connection. The fastener spacing needs to be kept at certain distances to prevent splitting between the fasteners. Rules for this are given in most codes, see Volume 2, Chapter 10.

4.9.1 Group effect

When a number of fasteners are loaded, they seldom reach their load-carrying capacity at the same time. This is due to local variations in timber strength, hole sizes, misalignment of holes and an uneven load transfer between the connecting members. Cramer (1968) and Lantos (1969) described what happens if a row of fasteners is loaded in tension and found that the first and last fasteners receive the highest load level, that is they will fail first, see Figure 4.23.

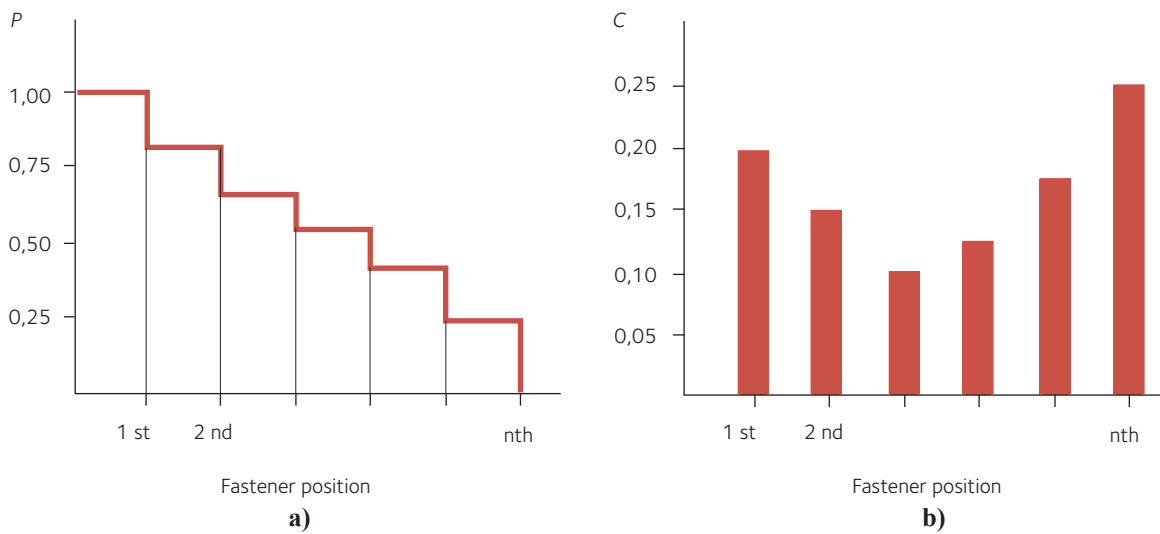


Figure 4.23: Load distribution in bolted joint, Lantos (1969). a) Force distribution in side member, b) force distribution along the row of fasteners.

The group effect is largest in joints with stout dowels, behaving nearly linear elastic, see further Section 4.12 on glued joints where the uneven stress distribution causes the same type of problem. The total capacity of such a joint is lower than the sum of the capacities of the individual dowels. In joints with slender dowels, for example nails, the plasticity of the fasteners enables redistribution of loads so that the total joint capacity is close to or equal to the sum of the individual fasteners' capacity, Blass (1990).

4.9.2 Eurocode 5 application

For dowel-type fasteners placed in a row parallel to grain a group effect is applied:

$$n_{\text{ef}} = n^{k_{\text{ef}}} \quad (\text{screws } d \leq 6 \text{ mm, nails and staples}) \quad \text{Eq. 4.23}$$

$$n_{\text{ef}} = \min \begin{cases} n \\ n^{0,9} \sqrt[4]{\frac{a_1}{13d}} \end{cases} \quad (\text{screws } d > 6 \text{ mm, bolts and dowels}) \quad \text{Eq. 4.24}$$

where a_1 is the distance between fasteners parallel to the grain. The group effect is strongly dependent on the distance a_1 between fasteners placed in a row parallel to the grain. For nails, the value of $k_{\text{ef}} = 1$ if $a_1 \geq 14d$, that is there is no group effect. For nails with the common spacing $10d$, $k_{\text{ef}} = 0,85$ and for dense spacing, $a_1 = 7d$, $k_{\text{ef}} = 0,7$, Eurocode 5: Section 8.3.1.1(8), see Figure 4.24. Reduction of the load-bearing capacity can be avoided by shifting the nails $\pm d$ in the direction of the fibres.

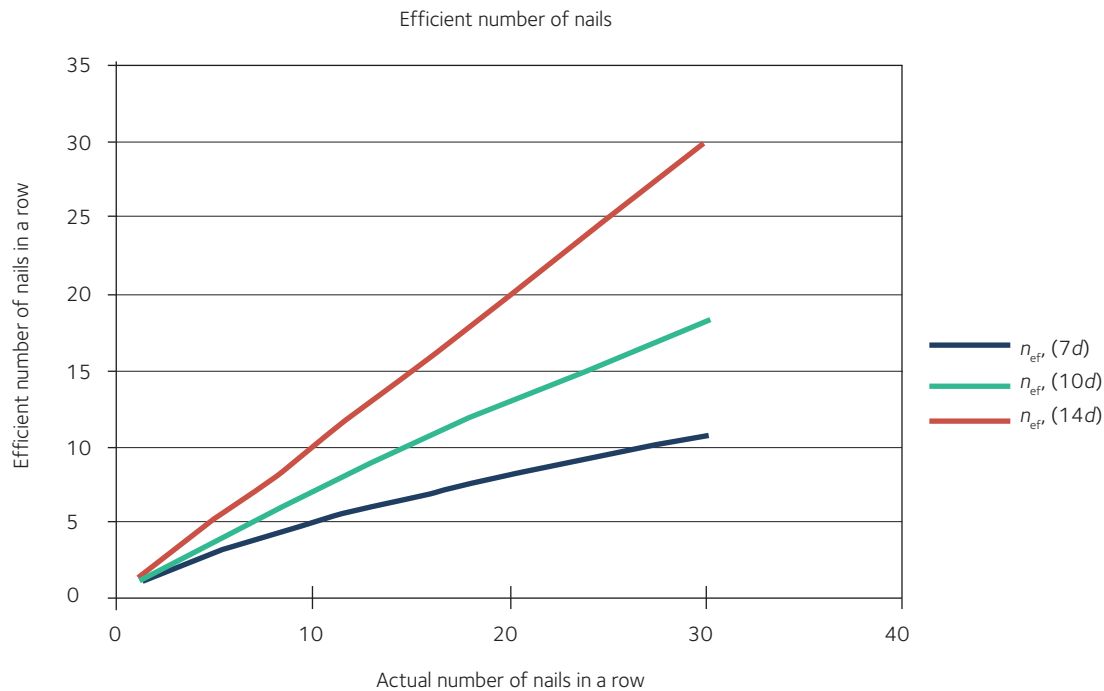


Figure 4.24: Efficient number of nails in a row dependent on spacing.

For bolts, the effect is also dependent on the distance between the fasteners and for the minimum spacing of $5d$ the effect is significant, see Figure 4.25. If the spacing is larger than $13d$, no group effect is expected.

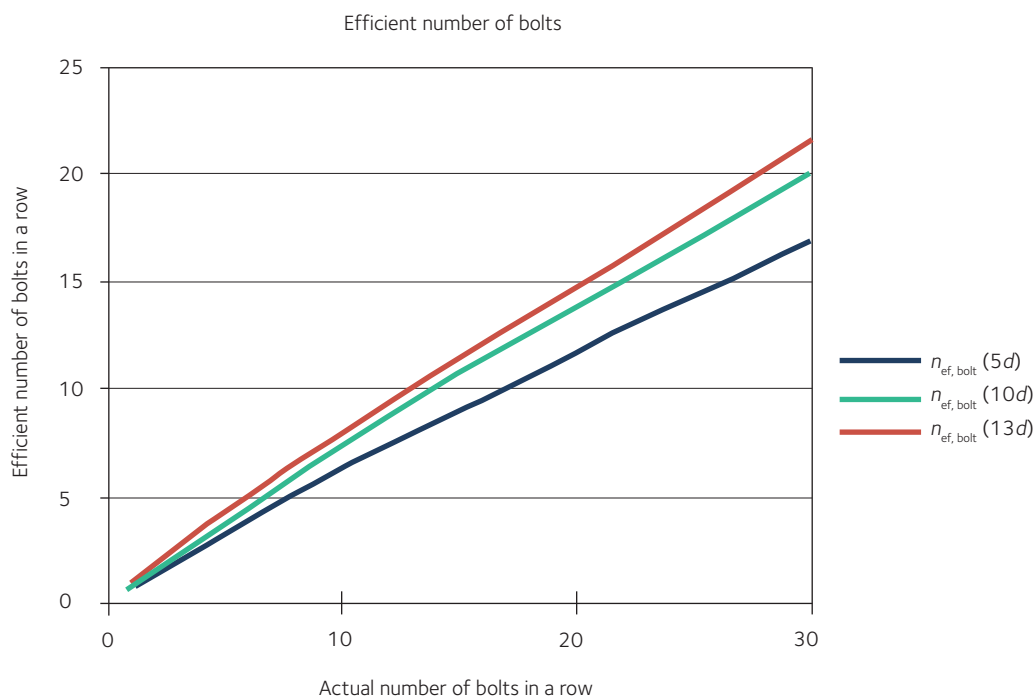


Figure 4.25: Efficient number of bolts in a row dependent on spacing.

The group effect shows a stronger dependency on spacing for nails than for bolts.

4.9.3 Brittle failure modes in dowelled joints

Failure according to Johansen theory occurs for single dowels placed at appropriate spacing to prevent splitting before bending of the dowel or embedding failure takes place. In many joints, large spacing is not possible to use since this would require large dimensions of timber members, ending up in uneconomical structures. However, when fasteners are closely spaced the risk for splitting increases. This can induce brittle failure in the joint before embedding failure occurs. Brittle failure in joints should be avoided and the capacity for the brittle failure modes must be checked along with the capacity of the single fastener and the group of fasteners. Brittle failures can be of different types; row shear, block shear, plug shear and splitting, see Figure 4.26.

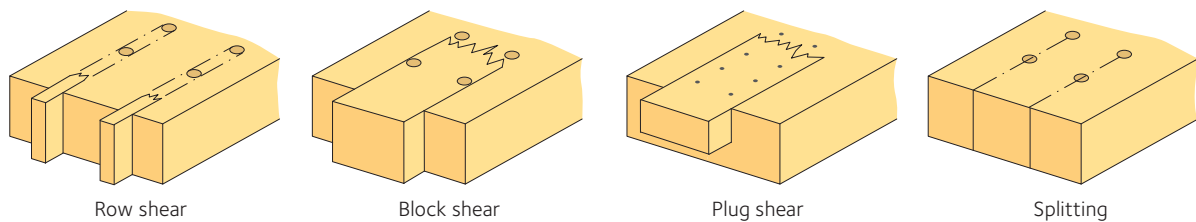


Figure 4.26: Brittle failure modes in mechanical timber joints.

If the joint consists of several rows of fasteners block/plug shear or row shear failure may occur. For nailed joints, row shear never occurs due to the small diameter of the fasteners. Furthermore, block shear is uncommon since nails seldom protrude through the timber member. Nailed joints should therefore be checked only for plug shear failure. Dowelled joints should be checked for block shear failure according to Eurocode 5, but row shear failure is left out. Row shear failure commonly shows a higher capacity than block shear, since splitting occurs along more faces compared to block shear failure. The faces can be designated as in Figure 4.27.

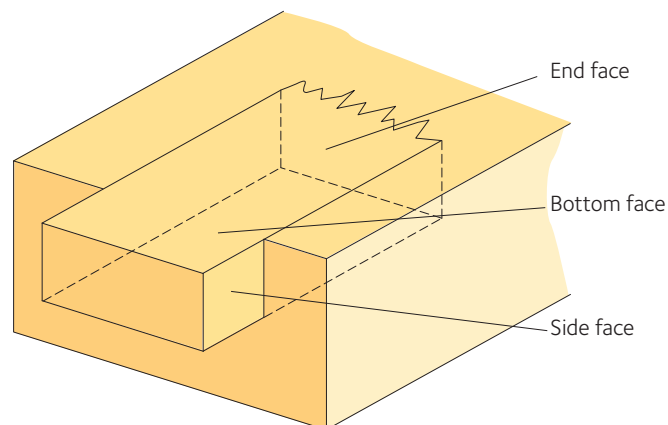


Figure 4.27: Faces along which splitting occurs in plug shear failure. In block shear failure no bottom face forms.

4.9.3.1 Block shear failure

The resistance in block shear consists of two possible contributions; the resistance in tension of the end face and the resistance in shear of the side faces. The tensile and shear capacities cannot be added since they are associated with different stiffnesses. The resistance is taken as the maximum value of the tensile and the shear capacities of the block:

$$F_{bs,Rd} = \max \begin{cases} 1,5A_{net,t} f_{t,0,d} \\ 0,7A_{net,v} f_{v,d} \end{cases} \quad \text{(dowel-type connectors)} \quad \text{Eq. 4.25}$$

The area $A_{net,t}$ is the net area at the end of the plug and the areas $A_{net,v}$ is the net area of the sides of the block. The factors 1,5 and 0,7 are empirically derived factors to accommodate for two different phenomena. In the case of the tensile capacity, the tensile strength can be higher locally at a joint, since the probability of having defects is lower in a specified area than in a full size member, Larsen (2003). The shear capacity is reduced due to a volume effect that affects the shear strength when the loaded area is large, Johnsson (2004). The areas should be reduced with the size of the fastener holes, as indicated by the subscript “net” in Equation 4.25. The method is described in Eurocode 5, Annex A.

4.9.3.2 Plug shear failure

Equation 4.25 can be used to establish both the block shear and the plug shear capacity. What differs is the calculation of the area $A_{net,v}$, which for plug shear failure can be taken as the sum of the net bottom shear area and the net shear area along the sides of the plug.

4.10 Forces acting at an angle to the grain

Wood is weak in tension perpendicular to the grain. Producing joints such as those in Figure 4.28 is not recommendable. A better solution is to transfer the forces in compression as in Figure 4.29.

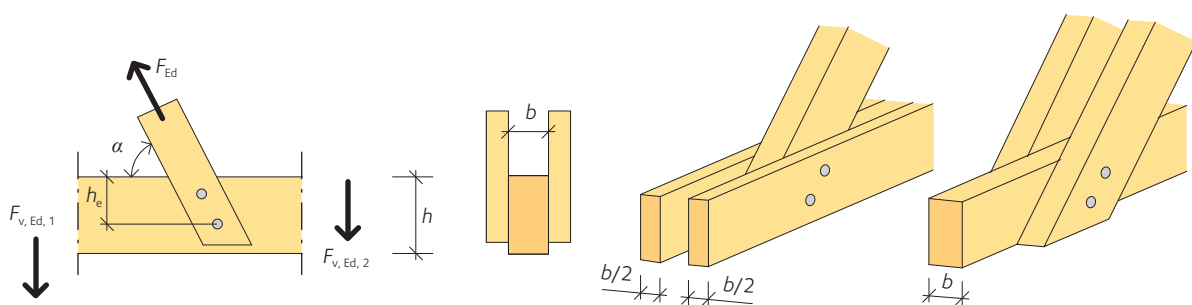


Figure 4.28: Joints loaded at an angle to the grain.

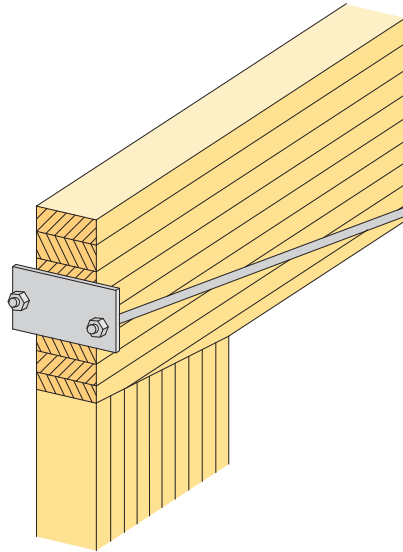


Figure 4.29: Load transfer in compression.

If it is impossible to transfer forces in compression it is important to place the fasteners as far from the loaded edge as possible to avoid splitting of the timber member. More or less sophisticated models exist to predict the capacity of a joint loaded in an angle to the grain, some of them based on linear fracture mechanics, Gustafsson et al. (2003). In Eurocode 5, Section 8.1.4, the phenomenon is taken into account by satisfying:

$$F_{v,Ed} \leq F_{90,Rd} \quad \text{Eq. 4.26}$$

where $F_{v,Ed} = \max\{F_{v,Ed,1}; F_{v,Ed,2}\}$, that is the largest of the two shear forces in the beam on each side of the connection as shown in Figure 4.28. The design resistance is determined on the basis of the characteristic resistance $F_{90,Rk}$ determined as:

$$F_{90,Rk} = 14b \sqrt{\frac{h_e}{1 - \frac{h_e}{h}}} \quad \left(\text{with } b, h_e \text{ and } h \text{ taken in mm and with } F_{90,Rk} \text{ in N} \right) \quad \text{Eq. 4.27}$$

where h , h_e and b are defined in Figure 4.28.

4.11 Punched metal plate fasteners

Punched metal plate fasteners are an integration of nails and nail plates. They are produced by punching nails directly from a steel plate, see Figure 4.4 b). Due to limitations in the punching process, the thickness of the original steel plate cannot exceed 2 mm, which has the consequence that the ‘nails’ have low capacity. The length of the ‘nails’ is commonly 8 – 15 mm, Nielsen (2003). The most common usage of punched metal plate fasteners is in the production of timber trusses, see Figure 4.30. In Sweden the application is prominently for the housing industry, with common spans of 9 – 15 m of these trusses, but they can reach towards 40 m span. Punched metal plate fasteners are installed through pressing them into wood by means of a hydraulic press. Therefore it is not recommendable to use punched metal plate fasteners for joints on the building site.

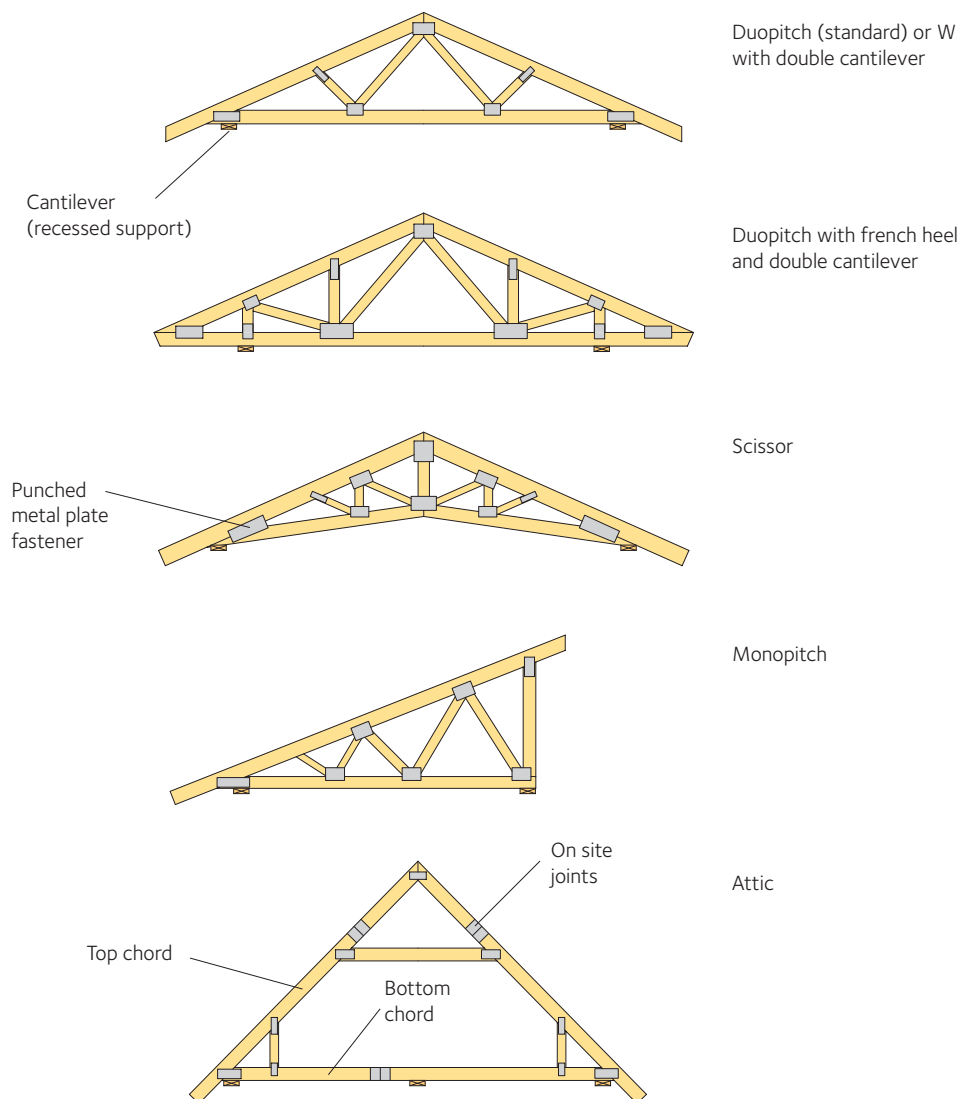


Figure 4.30: Trusses produced with punched metal plate fasteners, Nielsen (2003).

Due to the production process, the punched metal plate fastener will show an orthotropic behaviour. The strength of the fastener is dependent on the angle to the load direction and for a full characterisation of a punched metal plate fastener no less than 11 strength parameters need to be determined; shear, tensile, compressive and anchorage strengths in perpendicular directions and three constants which describe the anchorage behaviour, SS-EN 1075 and SS-EN 14545.

When designing a joint with punched metal plate fasteners, the tensile, compression and shear strength must be checked along with the risk for buckling of the plate at unsupported sections of the assembly. The procedure is described in detail in Eurocode 5, Section 8.8, but is seldom performed by hand calculations. Well developed software exists for the concurrent design of punched metal plate fasteners and trusses, taking into account the effect of movements in the joints, which affects the force distribution in the truss.

Punched metal plate fasteners can also be used to reinforce timber itself, but also the area around joints, Nielsen (2003). In that case, it is not the punched metal plate fastener in itself that is interesting to design, but rather the effect it creates by preventing failure perpendicular to the grain in timber. Examples of usage are, see Figure 4.31:

- increasing the compression strength perpendicular to grain by providing a secondary load path in the connection and stiffening the wood
- increasing the stiffness in bending by reinforcing the compressive side of the timber member
- increasing the tensile capacity perpendicular to grain in the joint area to prevent brittle failures such as block shear or splitting and also provide high embedding strength at the most heavily stressed section in a dowelled joint.

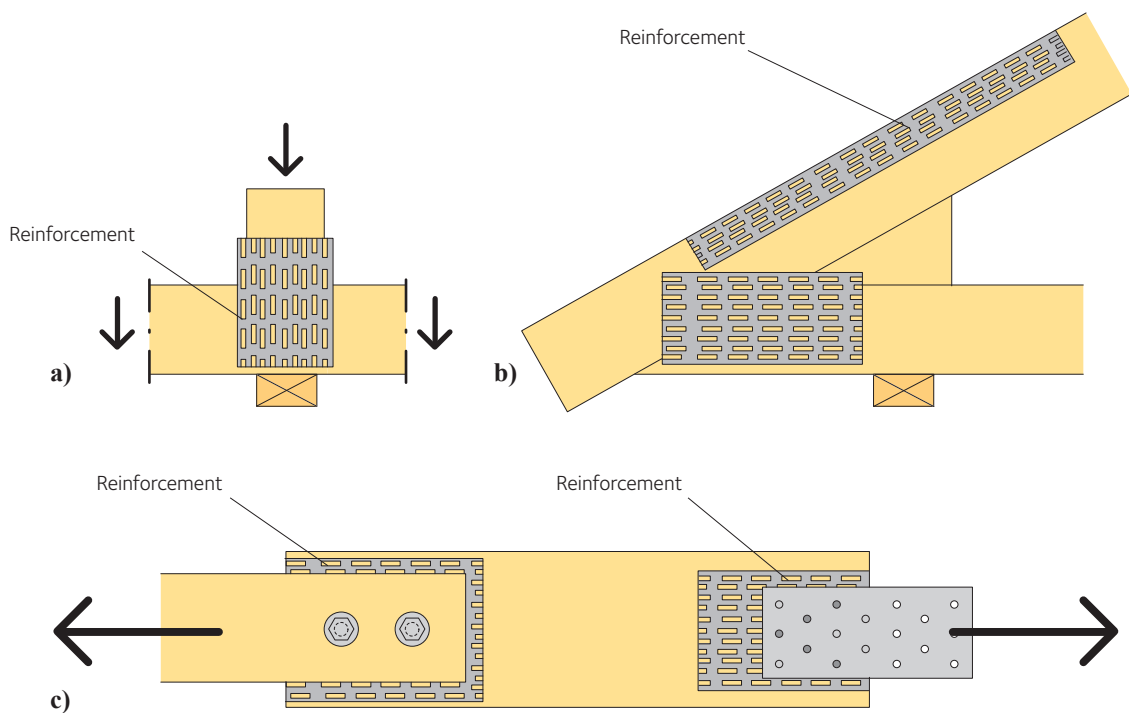


Figure 4.31: Strengthening of timber structures using punched metal plate fasteners, Nielsen (2003).

The usage of punched metal plate fasteners to strengthen dowelled joints can double the loadcarrying capacity, Blass (2003).

4.12 Glued joints

Glued joints are frequently used in timber engineering, especially when producing elements. They are not frequently used as a joining technique between members due to:

- a lack of precise design rules
- the need for controlled environmental conditions when gluing, making only factory production a feasible option.

Glued timber joints can be categorised according to Figure 4.32.

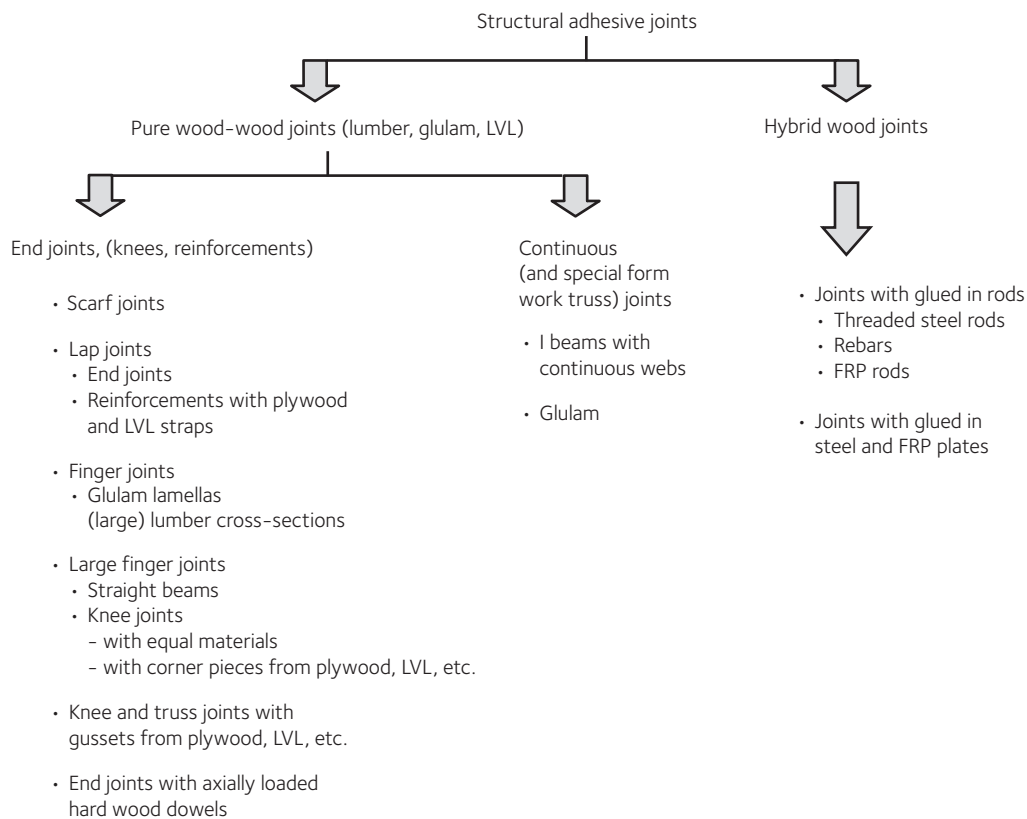


Figure 4.32: Classification of different types of structural adhesive joints, Aicher (2003).

The production of glued joints must take place under controlled conditions with a quality control to ensure consistent quality of the end result.

4.12.1 Glue characterisation

The strength of glued joints is difficult to describe analytically. If the glue is ductile, the shear strength describes its behaviour well. If it is brittle, the fracture energy, G_f , of the glue is the best descriptor. To characterise the brittleness, a brittleness ratio has been defined as f_v^2/G_f . A full characterisation from experiments of the shear strength f_v , the tensile strength f_t , the fracture energies in shear G_{ft} and in tension $G_{f\sigma}$ are available only for a few glues, see Table 4.9. One of the problems is that pure tension or shear is seldom present in a real joint, making it impossible to separate strength components even in testing.

Table 4.9 Characteristics of adhesives and comparison with wood, Aicher (2003).

Glue	f_v [N/mm ²]	G_{ft} [N/mm]	f_v^2 / G_{ft} [N/mm ³]	f_t [N/mm ²]	$G_{f\sigma}$ [N/mm]	$f_t^2 / G_{f\sigma}$ [N/mm ³]
Resorcinol/Phenol	8,4	0,7	99	6,4	0,36	114
PVAC	7,4	2,1	26	-	-	-
Polyurethane	2,8	0,6	14	1,0	0,23	4
Wood (Picea Abies)	8,9	1,0	83	2,9	0,3	28

From Table 4.9 one can conclude that the glue formerly often used for making glulam, resorcinol/phenol is more brittle than wood itself, while polyurethane is less brittle than wood.

4.12.2 Glue types

Resorcinol/phenol has two components and creates a reddish brown glue line with good mechanical properties also in fire. The glue can set at room temperature, but an elevated temperature (+40 °C) reduces the curing time. The glue line is resistant to moisture change. Resorcinol is expensive and a blend with 50 – 75 % phenol is therefore used. Up until a few years ago, resorcinol/phenol was the most common glue when producing glulam in Sweden. Currently Melamine-Urea-Formaldehyde (MUF) is used for glulam production, since this glue has a whitish glue line. It has the same structural properties as resorcinol/phenol. The curing time is an hour in room temperature, but only a few minutes if using a high frequency oven (microwave technology) during setting.

Polyurethane is also made up of two components, but is weaker than resorcinol although the demand for pressure during curing is not so high. Polyurethane is preferable to epoxy due to the risk for allergic reaction. Epoxy is on the other hand known to bond almost any material and can be used for gluing on site.

4.12.3 Strength

The strength of a lap joint was first described by Volkersen (1938) who arrived at the conclusion that the distribution of shear stress $\tau(x)$ in the joint can be described as:

$$\tau(x) = \tau_m \frac{\rho}{2} \left[\frac{\cosh(\rho x / \ell)}{\sinh(\rho / 2)} - \frac{(1 - \alpha) \sinh(\rho x / \ell)}{(1 + \alpha) \cosh(\rho / 2)} \right] \quad \text{Eq. 4.28}$$

$$\rho = \sqrt{\frac{G \ell^2}{E_1 t_1 d} (1 + \alpha)} \quad \alpha = \frac{E_1 A_1}{E_2 A_2} \quad \tau_m = \frac{P}{b \ell}$$

Refer to Figure 4.33 for notations. The coordinate x increases from left to right along the length ℓ .

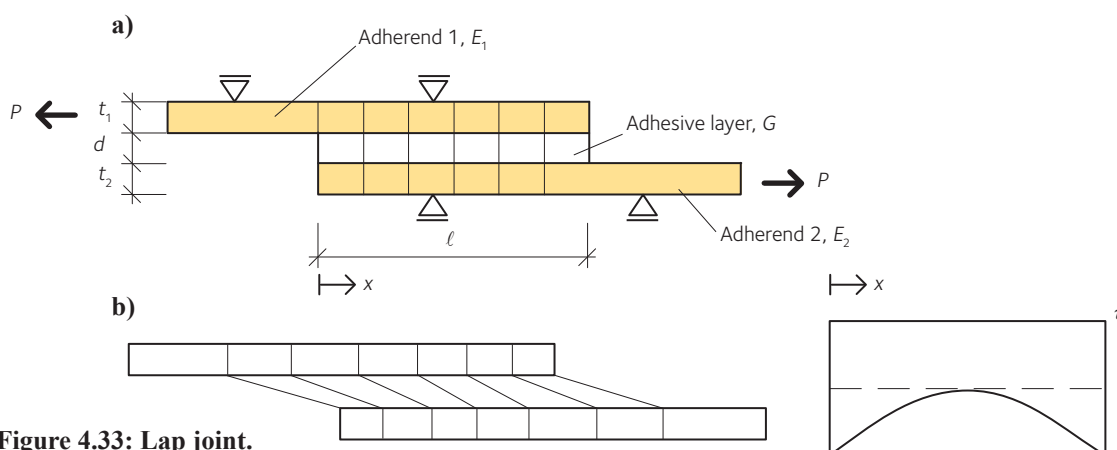


Figure 4.33: Lap joint.

The most important result from the Volkersen theory is that the stress distribution in the joint is non-uniform. The strength of the joint cannot be determined by calculating τ_m , which is the average shear stress value. There will be peak stresses at the ends of the glue line where failure will be initiated. The same phenomenon, but less prominent, is behind the group effect in dowels placed in a row. A deeper study of the shear stress $\tau(x)$ will reveal that above a certain

length, there is no change in peak stress or in other words, lengthening the connection will not increase its strength. Volkersen theory can be generalised to non-linear fracture mechanics, Gustafsson (1987). By studying the limit cases for non-linearity, fully plastic bond line and fully brittle bond line, Gustafsson (1987) arrived at upper and lower bounds for establishing the capacity of a joint:

$$\begin{aligned} P_{\max} &= f_v b \ell \\ P_{\min} &= \sqrt{2(1+\alpha)} b \sqrt{t_1 E_1 G_{f\tau}} \end{aligned} \quad \text{Eq. 4.29}$$

If a fully brittle bond line can be assumed, the capacity is not influenced by the length of the bond line. However, the range for the brittleness ratio where non-linear fracture mechanics is valid is dependent on the bond length through:

$$\begin{aligned} 0,1(1+\alpha) &\leq \omega \leq 10(1+\alpha) \\ \omega &= \frac{\ell^2 f_v^2}{t_1 E_1 G_{f\tau}} \end{aligned} \quad \text{Eq. 4.30}$$

For smaller ω , plastic conditions apply, while for larger ω , linear fracture mechanics should be used. Using $\ell = 400$ mm, $t = 45$ mm, and $E_1 = 13\,000$ MPa and f_v and $G_{f\tau}$ for a resorcinol-phenol glue according to Table 4.9, $\omega = 27$. For $\alpha = 1$ (symmetrical lap joint), the limit value for $\omega = 20$, which means that linear fracture mechanics can be applied. Changing glue to polyurethane makes $\omega = 3,8$, which means that non-linear fracture mechanics must be used.

4.13 Glued-in rods

A special application of a gluing is glued-in rods, Figure 4.34. This is an appealing solution since the joint is hidden inside the timber member and the connection is strong and stiff. Theoretically, there are no general design rules, but substantial testing have established good knowledge in the field, Gustafsson and Serrano (2001) and Tlustochowicz et al (2010).

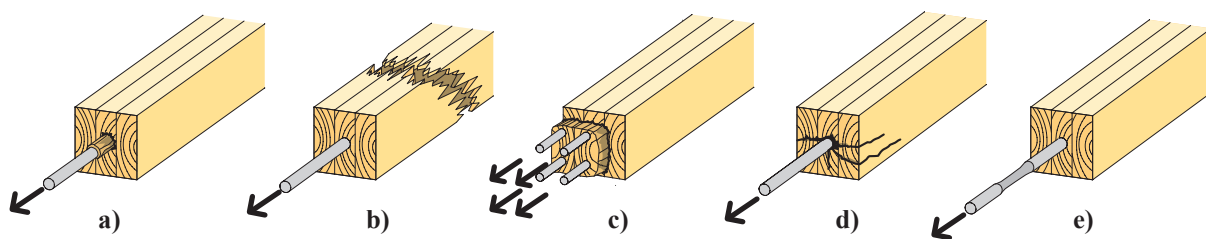


Figure 4.34: Failure modes for glued-in rods, Tlustochowicz et al. (2010).

4.14 Creating built-up column sections

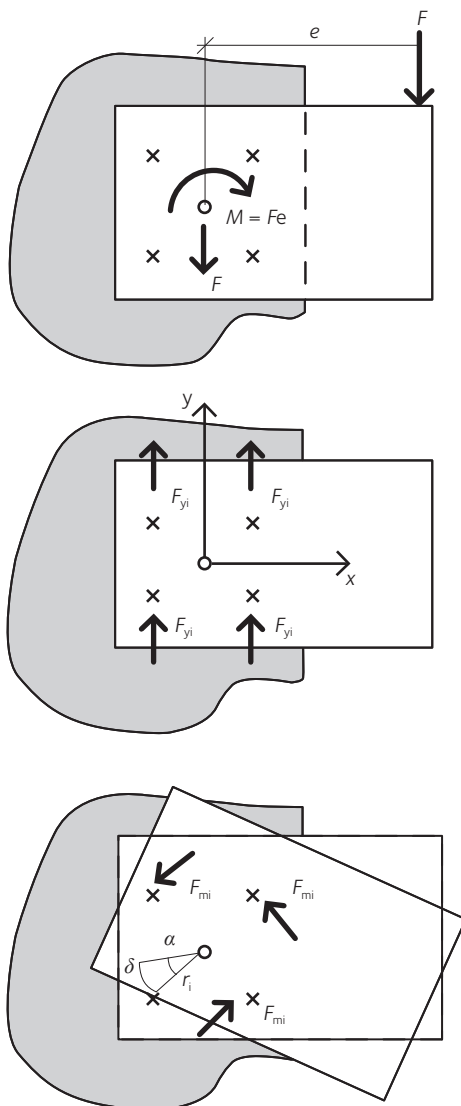
When designing timber columns and other members of a timber structure, it is not unusual to need other cross sections than rectangular ones (crosses and boxes are common). These can be built-up from several timber members and mechanically jointed as described in Eurocode 5, Annex C. Another option is to order this kind of structure directly from the glulam manufacturer, who then uses glue to fabricate the column, which leads to full composite action in the cross section. To obtain enough pressure during production, 1 – 2 nails/dm² is needed and the nails should have sufficient anchorage length and preferably be ringed to

further increase the anchorage. Alternatively screws can be used. Sufficient pressure can also be created through a distributed weight on the glued member.

4.15 Load distribution in joints

4.15.1 Elastic load distribution

In a joint, the individual fasteners will receive different loads depending on the global load applied to the joint. In tension and compression, it is a matter of calculating the load components and applying them to the individual fastener, but for moment resisting joints the actual geometry of the joint will produce different loads in different fasteners depending on their position in the joint. Assume a timber joint loaded with a vertical eccentric load with lever arm e with respect to the centre of gravity of the joint. The elastic load distribution can then be determined as shown in Figure 4.35:



1. Translate the eccentric forces to the centre of gravity of the fastener group.
2. The vertical force on each fastener is $F_{yi} = F/n$, where n is the number of fasteners.
3. From the moment $M = F \cdot e$ the load vector F_{mi} results. Assuming a linear force-slip relation, $F_{mi} = K\delta = Kar_i$, where K is the slip modulus of the connector, δ is the displacement due to rotation in the joint and α is the rotation angle, see bottom figure.
4. The external moment and the sum of the internal moments must be equal:

$$M = F \cdot e = \sum r_i F_{mi} = \sum r_i K \alpha r_i = K \alpha \sum r_i^2$$
5. The term $\sum r_i^2$ is a characteristic of the joint and is called the polar moment of inertia:

$$I_p = \sum r_i^2 = \sum (x_i^2 + y_i^2)$$
6. The load component F_{mi} can now be established through:

$$F_{mi} = K \alpha r_i = \frac{M r_i}{I_p}$$

$$F_{mxi} = -\frac{M y_i}{I_p}; \quad F_{myi} = \frac{M x_i}{I_p}$$

7. Vector summation gives the load effect on one fastener (including $F_{xi} = F_x/n$ in the general case when the external load F has a component F_x in the x -direction):

$$F_i = \sqrt{(F_{xi} + F_{mxi})^2 + (F_{yi} + F_{myi})^2}$$

8. F_i should be compared to the capacity of one fastener

Figure 4.35: Load distribution in a moment resisting joint.

4.15.2 Plastic load distribution

As dowelled joints show a plastic behaviour, there is a possibility to increase the total resistance of the joint, by allowing the outermost dowels to plasticize and thereafter redistribute their load to the dowels closer to the centre of rotation, see Figure 4.35. Assuming that the plastic behaviour in Figure 4.7 b) can be idealized to elasto-plastic behaviour, there is a possibility to establish a lower bound and an upper bound for the plastic resistance of the joint.

Note that the principle of superposition does not apply when assuming plastic behaviour. There is a limit on the displacement for one dowel, since the assumption of elastoplastic behaviour is limited by the possible displacement of the dowel. A reasonable limit could be that the maximum allowable displacement would be $u_{\max} \leq 4 u_y$ where u_y is the elastic displacement. The practical consequence is then that dowels closer to the rotational centre than $0,25r_{\max}$ (r_{\max} being the maximum distance from the centre of rotation to the outermost dowel) are not taken into account when establishing the plastic resistance of a joint.

4.15.2.1 Upper bound for plastic resistance

An upper bound F^+ is established through the principle of virtual work, (Larsen and Riberholt, 1999). A centre of rotation is assumed somewhere along a line perpendicular to the load. Often this line passes nearby the elastic centre of gravity, see Figure 4.35. Assuming that all dowels have equal yield load F_y , and reusing Figure 4.35, the internal and external works W_i and W_e are:

$$\begin{aligned} W_i &= \sum_{i=1}^n \alpha r_i F_y \\ W_e &= \alpha e F^+ \end{aligned} \tag{Eq. 4.31}$$

Equating W_i and W_e leads to:

$$F^+ = \frac{\sum_{i=1}^n \alpha r_i F_y}{\alpha e} = \frac{F_y}{e} \sum_{i=1}^n r_i \tag{Eq. 4.32}$$

To find the optimal value for F^+ , an iteration procedure is needed, where the centre of rotation is assumed and F^+ calculated. F^+ will have a minimum value when the exact plastic capacity is obtained. The optimum position of the centre of rotation is usually close to the elastic centre of gravity but on the side opposite the external load.

4.15.2.2 Lower bound for plastic resistance

The lower bound for the plastic resistance is established through dividing the fasteners into two groups; those that resist the moment and those that resist the normal force. The selection is made under the condition that equilibrium with the imposed load must be preserved.

If the joint is loaded with a force acting through the centre of gravity, the lower and upper bound will coincide as the direction of all reaction forces will be the same for all fasteners. The method is exemplified in Figure 4.36.

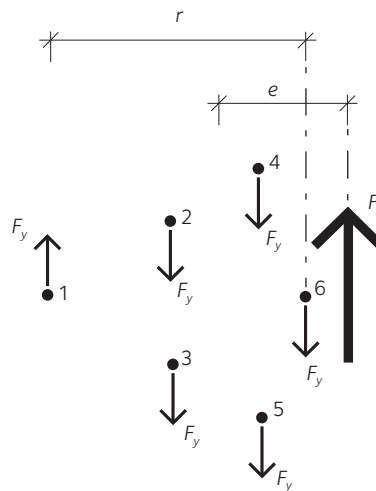


Figure 4.36: Example of lower bound establishment of plastic load distribution in joint, Larsen and Riberholt (1999).

In Figure 4.36, the lower bound F^- for the resistance is established through:

$$\text{Vertical equilibrium: } F - 4 \cdot F_y = 0$$

$$\text{Moment equilibrium: } F \cdot e - F_y \cdot r = 0$$

where F_y is the yield load for the fasteners in this case assumed equal for all of them.

The lowest value of F found from these two equilibrium relations constitutes the lower bound solution found from the assumed force distribution, that is:

$$F^- = \min \begin{cases} 4F_y \\ \frac{r}{e} F_y \end{cases} \quad \text{Eq. 4.33}$$

4.16 Stiffness of joints

The slip modulus K_u of connections in the ultimate limit state should be taken as two thirds of the slip modulus in the serviceability limit state, Eurocode 5, Section 2.2.2:

$$K_u = \frac{2}{3} K_{\text{ser}} \quad \text{Eq. 4.34}$$

The slip modulus in serviceability limit state, K_{ser} , in N/mm can be established through, Eurocode 5, Section 7.1:

$$K_{\text{ser}} = \begin{cases} \rho_m^{1.5} d / 23 & \text{(for dowels, bolts, screws and pre-drilled nails)} \\ \rho_m^{1.5} d^{0.8} / 30 & \text{(for non pre-drilled nails)} \end{cases} \quad \text{Eq. 4.35}$$

where ρ_m is the mean density in kg/m³ of wood and d the outer diameter in mm of the fastener. In joints between wood and steel and between wood and concrete, K_{ser} established using the wood density can be multiplied by 2.

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Chapter 5

Composite timber elements

Bert Norlin

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5 Composite timber elements

By composite timber elements here is meant I-beams, hollow section beams and stressed skin panels (SSP). As examples, the first two are typical beams having cross-sections as shown in Figure 5.2, while the last one is a kind of building block covering a large surface as in Figure 5.14.

Composite timber elements are usually, but not necessarily, made of more than one material. Common wood based materials used are structural timber, glued laminated timber, laminated veneer lumber (LVL), plywood, fibreboard, particleboard, oriented strand board (OSB), etc. Other materials such as steel, aluminium, concrete, plastics, and even glass are possible to use. But here we restrict ourselves to wood based materials.

Basically these elements carry the load through bending action, especially in horizontal structures like floors, roofs, bridge decks, etc. Of course axial forces must be considered for structures affected by such forces, like in wall elements and columns.

The common property of all these composite elements is that the section is divided into wide outer flange parts with an intermediate thin web part. Naturally, the flanges of an SSP-element are very wide and very thin, but the web panels are still quite thin in relation to the flanges. The same is also true for an ordinary I-beam, even though the flanges may have a small width the web panel will have a much smaller thickness. When these kinds of elements are subjected to bending, most of the bending moment is carried by the flanges while a minor part is carried by the web. The shear force on the other hand is mainly carried by the web panel or panels. The only exception is T-shaped sections for which no real distinction between a flange and web part can be made.

Consider now a small cut-out from the compression flange of an ordinary I-beam as shown in Figure 5.1 a). The axial stresses change slightly when moving from one end to the other end of the cut-out. The only thing that can cause this change is the shear stress affecting the web panel. All the shear stress must be transferred into the flange in order to build up the axial stresses or vice versa. Now if the web-to-flange connection is sensitive to shear deformation a small slip will result between the two materials, as shown in Figure 5.1 b). If this slip is negligible we have so called “full composite action” (fca) in which case the beam’s stiffness and stress magnitudes can be based on ordinary beam bending theory as explained in Section 5.1. If the slip is non-negligible the applied load will be counteracted differently such that with increasing slip the flanges will carry less of the applied bending moment through axial compression and tension of the flanges and more through individual bending of each flange and web panel. This will result in a decreasing transfer of shear stresses over the interface. In addition to this redistribution from axial flange forces to flange bending the stiffness will decrease leading to increased deflection. This state of non-negligible slip is referred to as “partial composite action” (pca) and is explained in Section 5.2.

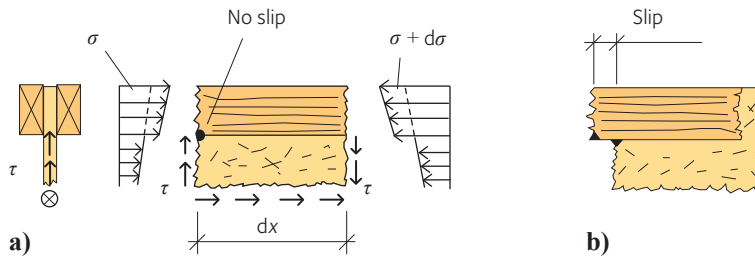


Figure 5.1: A small cut-out of the web-to-flange connection of an I-beam. a) No slip or negligible slip, b) non-negligible slip.

Typically, full composite action is usually assumed for glued connections having very small glue line thicknesses. Partial composite action must be assumed if the glue is replaced by mechanical fasteners and/or the interface have a thickness such that its shear deformation is not very small. Actually, the shear deformation of the web panel itself can be treated as a slip and can be accounted for through the theory explained in Section 5.2.

5.1 Structural elements with full composite action

I-beams and hollow section beams are treated in Section 5.1.1 while stressed skin panels are dealt with in Section 5.1.2. But bear in mind that most design issues are similar for these two types of composite elements.

5.1.1 Glued thin webbed beams

Examples of thin webbed beams are shown in Figure 5.2. Two basic shapes can be distinguished, that is open I-sections and closed hollow sections (box-sections). The flanges are usually made of structural timber, glued laminated timber or LVL, while web panels are made of hard fibreboard (hardboard), OSB, plywood or particleboard. In most commercial products sawn timber flanges in combination with hardboard, OSB or sometimes plywood are used.

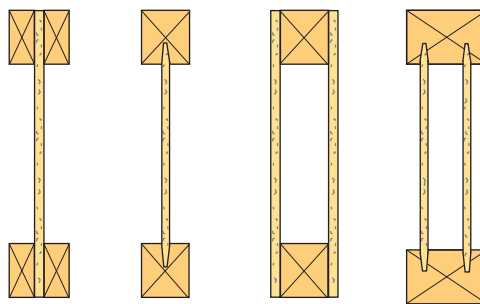


Figure 5.2: Typical cross-sections of thin webbed beams, I-sections as well as rectangular hollow sections (box-sections).

Figure 5.3 shows a side view of two typical I-beams (or hollow section beams), one having small web slenderness and one having large web slenderness. The slenderness limit between these two categories is approximately $h_w/b_w = 35$.

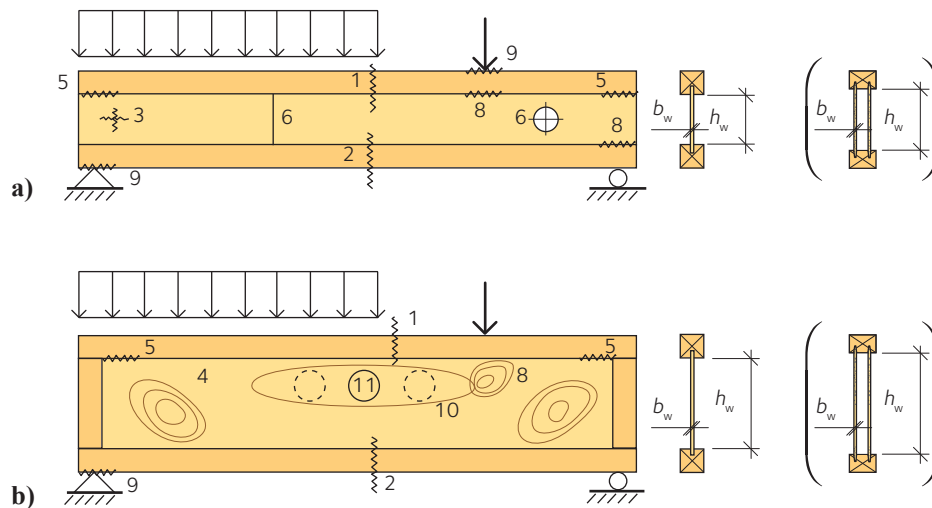


Figure 5.3: I-beam or hollow section beam having: a) small web slenderness or b) large web slenderness.

Possible failure positions are indicated symbolically and given a specific number in Figure 5.3. The type of failures indicated by the numbers are:

1. Compression failure of the compression flange, which in case of insufficient lateral bracing may involve the influence of lateral torsional buckling.
2. Tension failure of the tension flange.
3. Shear failure of the web panel.
4. Shear buckling failure of the web panel.
5. Shear failure of the flange-to-web interface, that is the glued connection between them.
6. Shear failure of open web panel joints and web panels with holes.
7. Buckling failure due to axial loading, **not** shown in Figure 5.3.
8. Patch loading failure, that is concentrated loading acting perpendicular to the flanges may cause failure in the neighbouring web panel region, either by a pure compression failure of the web panel in combination with some local flange bending (stocky web panels) or by a local buckling failure of the web panel that is also influenced by some flange bending (slender web panels).
9. Compression failure of the outer flange surfaces caused by concentrated forces acting perpendicular to the grain, which is checked using the rules described in Section 3.1.3.
10. Overall web buckling failure, which is quite rare for normal timber I-beams. Another type of buckling that may add to this buckling mode is flange induced web buckling due to increasing beam curvature.
11. Local web buckling caused by bending moments and/or axial force. This is more or less obscure for timber I-beams.
12. In the serviceability limit state one must check deflections, primarily caused by bending but sometimes significantly influenced by shear deformations in the web. **Not** shown in Figure 5.3.
13. The influence of vibrations and resonance frequencies in floors and bridge decks may also be important issues in the serviceability limit state, **not** shown in Figure 5.3.

Aspects 10 and 11, out of this extensive list of failure modes and requirements to be checked, are of little or no importance for beams of practical dimensions, and not treated in this book. The remaining aspects must at least be thought of whenever you attempt to design a beam of this kind. Usually some of them can be ruled out for a particular problem. Detailed design rules and recommendations are given in the following sections regarding most of the identified failure mechanisms and serviceability requirements.

5.1.1.1 Transformed or fictitious cross-sections

Here a transformed cross-section merely means a fictitious cross-section where the width of all parts except one has been changed such that the same modulus of elasticity can be assumed for the entire cross-section. The width is measured parallel to the relevant axis of bending. If a cross-section is made of two or more materials having different modulus of elasticity a transformed cross-section will simplify equations and actual design calculations. Using a transformed cross-section is by no means necessary from a theoretical point of view, but it brings practical simplifications. Figure 5.4 shows the transformation from a real cross-section into a fictitious cross-section.

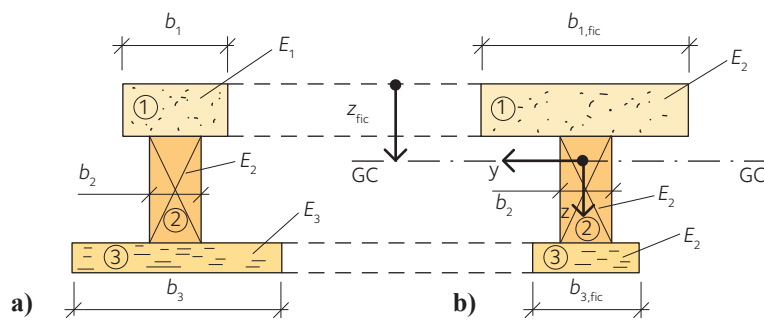


Figure 5.4: Going for a fictitious or transformed cross-section. a) The real physical cross-section, b) fictitious cross-section drawn to scale, if part 2 is taken as reference with $E_1 = 2$, E_2 and $E_3 = E_2/2$.

For a cross-section made up of 3 pieces as shown in Figure 5.4 the fictitious widths are determined as:

$$b_{1,\text{fic}} = \frac{E_1}{E_2} b_1, \quad b_{2,\text{fic}} = \frac{E_2}{E_2} b_2 = b_2, \quad b_{3,\text{fic}} = \frac{E_3}{E_2} b_3 \quad \text{Eq. 5.1}$$

if part 2 is taken as reference. A proper value of the modulus of elasticity must be used for each material making up the cross-section. For wood based materials this value will in the long run be affected by creep, as discussed in Section 2.4.7. The influence of creep is two folded for beams with a composite cross-section. First, the deflection and deformation of the beam increase with time. Second, the internal distribution of stresses between members will change with time. The outcome is that members that are more prone to creep will be unloaded, while members less sensitive to creep will carry more of the load with increasing time. Yet another problem is that two different moduli are given in material property tables for timber materials, for example in Eurocode 5. One value corresponds to a lower percentile value (usually 5 %) and one corresponds to the mean value of the modulus. Best practice is here to always use the mean values of the modulus for each material whenever a fictitious cross-section is needed. The simple reason for this is that we must try to predict the most likely distribution of stresses and avoid stress distributions affected by large scatters in material data. Note, however, that

this reasoning applies to finding the best stress distribution affecting a composite cross-section and may not be entirely correct for instability problems, which is discussed in connection to Equation 5.4 below.

Eurocode 5 uses the concepts of instantaneous and final conditions (“inst” and “fin”) to handle the creep problem. In a more general case we have n parts where $i = 1, 2, \dots, n$; and if one of the i -values is associated with the reference material, r , the fictitious width of the i^{th} part is:

$$b_{i,\text{fic}} = \mu_i b_i \quad \text{where} \quad \left\{ \begin{array}{l} \mu_{i,\text{ULS,inst}} = \mu_{i,\text{SLS,inst}} = \frac{E_{i,\text{mean}}}{E_{r,\text{mean}}} \\ \mu_{i,\text{ULS,fin}} = \frac{E_{i,\text{mean}}}{E_{r,\text{mean}}} \frac{(1 + \psi_2 k_{r,\text{def}})}{(1 + \psi_2 k_{i,\text{def}})} \\ \mu_{i,\text{SLS,fin}} = \frac{E_{i,\text{mean}}}{E_{r,\text{mean}}} \frac{(1 + k_{r,\text{def}})}{(1 + k_{i,\text{def}})} \end{array} \right. \quad \text{Eq. 5.2}$$

Here ULS and SLS refer to the ultimate limit state and the serviceability limit state, respectively. $k_{i,\text{def}}$ is the creep factor of the i^{th} member as if subjected to permanent loading and specified in Eurocode 5 for different materials. ψ_2 is the reduction factor for quasipermanent loading and should be determined for the load causing the largest stress. If that is a permanent action, a value of 1 should be used. When and how to use the different μ_i -values in Equation 5.2 is explained as:

$\mu_{i,\text{ULS,inst}} = \mu_{i,\text{SLS,inst}}$ is used in the ULS or SLS under instantaneous conditions, that is as if all design loads are applied during a period shorter than a year or so. This value should, in the ULS, be used to check the resistance of those members that are most prone to creep.

$\mu_{i,\text{ULS,fin}}$ is used in the ULS under final conditions, that is towards the end of a design life of some 50 years or so. This value should, in the ULS, be used to check the resistance of those members that are less sensitive to creep. The technique takes creep into account in an approximate way. Especially large variable loads act for short periods of time, under which not much creep will take place.

$\mu_{i,\text{SLS,fin}}$ is used in the SLS under final conditions to check deflections and deformations after a long time like 50 years. This value will give an estimation of creep deformations towards the safe side, as only the creep factor for permanent load k_{def} is used.

In practical design calculations the centre of gravity (GC), second area moment of inertia and bending stiffness are calculated for the fictitious cross-section, and in case of bending about the y-axis usually denoted as z_{fic} , $I_{y,\text{fic}}$ and $E_r I_{y,\text{fic}}$. Note that z_{fic} is measured from an arbitrarily chosen point of reference and that GC is the neutral axis in case of pure bending, that is no external axial loading. If you follow the above recommendations for μ_i , you will end up with as much as three different values of all relevant cross-sectional properties. As an example the second area moment $I_{y,\text{fic}}$ will take on three different values: $I_{y,\text{ULS,inst}} = I_{y,\text{SLS,inst}}$, $I_{y,\text{ULS,fin}}$ and $I_{y,\text{SLS,fin}}$.

Observe that when stresses are calculated for the fictitious section, such that they are assumed to be uniformly distributed along the width $b_{i,\text{fic}}$ for a given z -coordinate, they are also fictitious and must be smeared out over the real width using the appropriate value of μ_i from Equation 5.2 in the expression:

$$\sigma_{i,\text{real}} = \mu_i \sigma_{i,\text{fic}} \quad \text{where} \quad 1 \leq i \leq n \quad \text{Eq. 5.3}$$

For global stability problems such as column buckling and lateral torsional buckling of composite beams, the recommendation to always use the mean value of the elastic and shear moduli will be in conflict to the definition of slenderness parameters (see Sections 3.2.1 and 3.2.2), which are generally defined as:

$$\begin{array}{ll} \text{For column buckling} & \text{For lateral torsional buckling} \\ \lambda_{\text{c,rel}} = \sqrt{\frac{\text{Axial resistance}}{\text{Critical axial load}}} & \lambda_{\text{crit,rel}} = \sqrt{\frac{\text{Bending resistance}}{\text{Critical bending moment}}} \end{array} \quad \text{Eq. 5.4}$$

The conflict is that the slenderness parameter should use the most probable ratio, which is achieved if both the numerator and denominator are based on either characteristic or mean values of the involved material parameters. Only characteristic material properties are available for the numerator (the resistance is based on $f_{m,k}$, $f_{c,k}$, $f_{t,k}$, etc.), while both characteristic and mean values are available for the denominator (the critical load depends on E , G , etc. and not on any strength parameters). It is therefore recommended that E - and G -values used in stiffness expressions are taken as (lower) characteristic values, while the fictitious section is still based on mean values of the moduli. Typical stiffness values involved in finding critical loads are $E_r I_{y,\text{fic}}$, $E_r I_{z,\text{fic}}$, $E_r I_{w,\text{fic}}$ and $G_r I_{t,\text{fic}}$ for y -axis bending, z -axis bending, warping and torsion, respectively.

The difference is usually smaller than 5 percent in the final design resistance of a typical composite cross section, if comparing results obtained using either characteristic or mean values of the elastic moduli involved in determining the dimensions of the fictitious cross-section. If global instability is a part of the analysis the difference usually stays within 10 percent, if mixing characteristic and mean values when determining the slenderness parameter according to Equation 5.4. Hence, we conclude that a principal mistake regarding the used moduli will have a small impact on the final result, if compared to the large scatter of material data and the magnitude of partial coefficients used. The same also holds if mixing up the μ_i -values given in Equation 5.2, which in general will have a small influence on the final result. One can really ask if all the extra work associated with these different μ_i -values are worth the effort in practical design situations.

5.1.1.2 Bending resistance

Here a typical I-section is taken as an example, other sections (like box-sections) are assessed in the same principal way. In Figure 5.5 an I-section having flanges of equal dimension but different strength grades is illustrated. The stress distribution for pure bending is shown. It is assumed that the area moment of inertia $I_{y,\text{fic}}$ is determined according to Equation 5.2 using the timber compression flange as reference and that the tension flange has a much better strength grade than the compression flange. Note that this kind of asymmetry should be avoided in practical design, because the beam may accidentally be placed upside down.

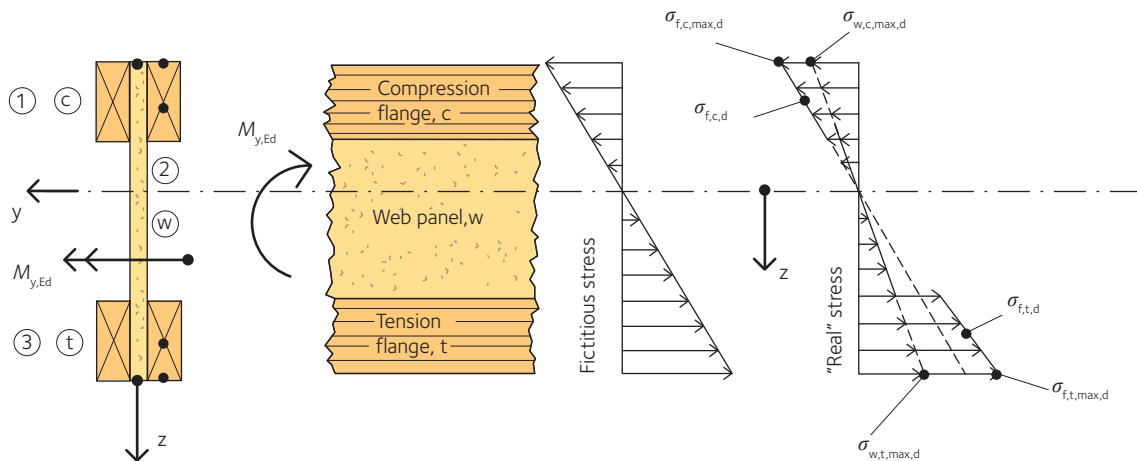


Figure 5.5: Stress distributions on an I-section (or box-section) used to determine its bending moment resistance $M_{y,Rd}$. Note that the “real” stress distributions indicated is valid if $E_w < E_{f,c} < E_{f,t}$.

In Figure 5.5 six dots indicate positions on the cross-section where the bending stress may become critically large in comparison to the strength of the relevant material. The absolute value of these stresses are determined using Navier’s equation for the fictitious cross-section, but also multiplying by the appropriate E_i/E_r -ratio as in Equation 5.3. The stress for any of the dots is:

$$\sigma_{\text{dot},d} = \frac{E_{\text{dot}}}{E_r} \frac{M_{y,Ed}}{I_{y, \text{fic}}} |z_{\text{dot}}| \quad \text{Eq. 5.5}$$

where z_{dot} is the distance from the neutral axis to the dot and E_r is the modulus of elasticity of the reference material (in this case the timber of the compression flange). It is now tempting to use Equation 3.6 in Section 3.1.2 to verify the resistance of the flanges. This is because the total stress affecting a timber flange can be split into two parts: one pure compressive or tensile stress at the centre of each flange and one pure bending stress, taken as the difference between the outermost edge stress and the centre stress. Finally, substituting these stresses into the appropriate interaction formula will verify the resistance. It has however turned out that such an approach is on the safe side compared to experimental results and also a bit cumbersome. The recommended approach is much simpler. Usually the flange depth is much smaller than the beam depth, in which case their resistance is almost entirely controlled by the compression or tensile strength of the flange material. Hence a reasonable design criterion would be to avoid stresses at the flange centres that are greater than the material strength at these points. But this approach is too optimistic if the flange depth becomes large in relation to the beam depth. This obstacle is overcome by realising that the bending strength is greater than the compression strength and much greater than the tensile strength for most timber materials. So the problem is solved by adding the additional requirement that the stress at the outermost fibres of the flanges must be smaller than the bending strength. For the dots in Figure 5.5 the following six verifications must be made.

$$\begin{aligned} \text{Compression flange: } & \sigma_{f,c,d} \leq f_{c,0,d} & \sigma_{f,c,\max,d} \leq f_{m,d} & \sigma_{w,c,\max,d} \leq f_{c,w,d} \\ \text{Tension flange: } & \sigma_{f,t,d} \leq f_{t,0,d} & \sigma_{f,t,\max,d} \leq f_{m,d} & \sigma_{w,t,\max,d} \leq f_{t,w,d} \end{aligned} \quad \text{Eq. 5.6}$$

where all σ -stresses are calculated using Equation 5.5; $f_{c,0,d}$ and $f_{m,d}$ are the compressive and bending strength of the timber in the compression flange, respectively; $f_{t,0,d}$ and $f_{m,d}$ are the tensile and bending strength of the timber in the tension flange, respectively; $f_{c,w,d}$ and $f_{t,w,d}$ are the compressive and tensile strength of the web, respectively.

Using $f_{c,w,d}$ and $f_{t,w,d}$ will slightly underestimate the resistance of the web panel/panels. The three main reasons for this are: 1) The strength values have been determined based on testing of fairly wide and uniformly stressed specimens. But in a beam just a small volume close to the edge is stressed to this level. 2) A lamination effect in the flange, where the timber will allow for some stress redistribution from the web into the flange before final web failure. 3) Web material of wood based sheeting has a tendency to break away the surface material if subjected to compression. This is more or less prevented for a surface that is glued to the timber. 4) The shear deformation of the web panel actually leads to a small increase of the axial stresses in the web, which slightly counteracts the favourable effects of remarks 1 to 3. By considering all these effects it should still be possible to increase these particular strength values by 10 percent, without hesitation. Such an increase is presently not promoted by the Eurocode.

Note that the verification criterion (Equation 5.6) does not cover the risk of lateral torsional buckling. The background to lateral torsional buckling of ordinary rectangular timber beams are treated in Section 3.2.3. Here some additional comments regarding thin webbed composite beams will be given. One very simple and easy to use method is to consider the compression flange to be an ordinary column that can buckle sideways without having to drag the rest of the cross-section with it. The column is cut out between two points at which the compression flange is sufficiently braced laterally. Pinned boundary conditions are assumed at these two points, that is the flange is treated as an Euler 2 column. Verification of the resistance is given by:

$$\sigma_{f,c,d} \leq k_c f_{c,0,d} \quad \text{Eq. 5.7}$$

where k_c is the reduction factor for flexural column buckling as described in Section 3.2.1. The bad thing with this model is that it grossly underestimates the bending resistance of the beam, especially for beams with large torsional stiffness $G I_{t,fc}$, like box-sections. Yet another problem is that Equation 5.7 is designed to work for one massive section of solid timber, but a flange usually has some web panels with different material properties attached to it. This is, however, a small and negligible problem in relation to neglecting the rest of the cross-section. A better approach to find the “true” resistance with regard to lateral torsional buckling is to rewrite Equation 5.7 as:

$$M_{y,Ed} \leq M_{y,LT,Rd} = k_{crit} M_{y,Rd} \quad \text{Eq. 5.8}$$

where $M_{y,LT,Rd}$ is the moment resistance due to lateral torsional buckling, k_{crit} is the reduction factor covering up for lateral torsional buckling and $M_{y,Rd}$ is the design resistance not influenced by instability and taken from Equation 5.6. The reduction factor k_{crit} is obtained following the sequence: critical value \rightarrow slenderness parameter \rightarrow reduction factor, that is:

$$M_{y,crit} \rightarrow \lambda \rightarrow k_{crit}$$

The designation λ refer in the following sections to $\lambda_{crit,rel}$ unless otherwise stated.

In determining λ the bending moment resistance $M_{y,Rk}$ is needed, which is the characteristic resistance without taking lateral instability into account. This value is easily obtained using Equation 5.6 with characteristic values rather than design values. Next step is to find the critical bending moment $M_{y,cr}$ for the actual support and loading conditions at hand. Note that this is a theoretical value obtained for a beam without any imperfections whatsoever. The basic idea is that the critical value captures the influence of geometry, boundary conditions and how the load is entering into the beam. Normally, $M_{y,crit}$ is calculated based on some elementary load case from the literature. Cross-sectional properties needed are typically $E I_{z,fi,c}$, $E I_{w,fi,c}$ and $G I_{t,fi,c}$ for z-axis bending, warping and torsion, respectively. They should be determined as discussed in Section 5.1.1.1, that is $I_{z,fi,c}$, $I_{w,fi,c}$ and $I_{t,fi,c}$ are calculated based on mean values of elastic and shear moduli, while E and G in front of them should reflect the characteristic value. It is not straightforward to find $M_{y,crit}$, but not unrealistically problematic. Guidance is given in many handbooks such as StBK-K2 (1973). Next, the slenderness parameter for lateral torsional buckling is obtained as:

$$\lambda \leq \sqrt{\frac{M_{y,Rk}}{M_{y,crit}(x_{dim})}} \quad \text{Eq. 5.9}$$

The value of $M_{y,crit}$ should be taken as close as possible to the design section x_{dim} where the beam is subjected to a combination of stiff axis bending, weak axis bending and torsion. Sometimes it is obvious where x_{dim} is located, sometimes one has to make a qualified estimation. It is of course always on the safe side to choose x_{dim} such that $M_{y,crit}$ has its maximum, but that may not always be the same position as where the beam section will fail.

Finally, k_{crit} can be read from a suitable design curve of an instability chart. Presently, only one design curve exists for lateral torsional buckling of timber beams, and that curve is actually valid for sections of rectangular structural timber and glulam. No specific curve exists for more complicated sections. In Figure 5.6 the one and only curve for lateral stability (as given in Section 3.2.3.3) is brought together with the two curves used for ordinary column buckling of structural timber and glulam (as given in Section 3.2.1). A fourth extremely important curve $1/\lambda^2$ is also depicted. This curve exactly represents the theoretical critical load of a beam or column without any imperfections, that is critical bending moment in case of a beam and critical axial load in case of column buckling. You may easily verify this by rewriting $1/\lambda^2$ using Equation 5.9 and then substituting into Equation 5.8, which results in the critical bending moment. It works the same for flexural column buckling, except that the bending moment is replaced by axial force. In either case it is important to realise that the critical curve $1/\lambda^2$ represents an absolute upper limit of the resistance, no postbuckling resistance is possible.

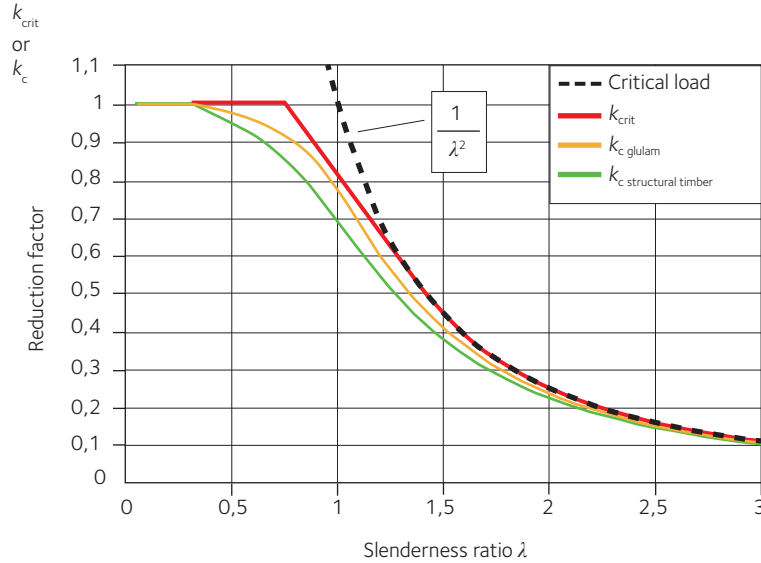


Figure 5.6: Buckling curves for lateral torsional buckling and flexural column buckling.

The whole idea behind the chart of Figure 5.6 is that the influence of geometry, support and loading arrangements enters via λ , while the buckling curves only corrects the resistance with respect to all relevant imperfections, where the most important ones for wood based materials are non-linear material and bow imperfections. The critical load is determined under the assumption of a linear elastic material that can sustain infinitely large stress. In reality k_{crit} or k_c can never exceed unity. We see that the two k_c -curves give a slightly greater reduction than the k_{crit} -curve. It can also be shown that a curve for lateral torsional buckling must give smaller reductions than for flexural column buckling. Since no specific curve exist for lateral torsional buckling of composite I- and box-sections it is here proposed that the present k_{crit} -curve can be used also for composite beams, especially since most composite beams have smaller bow imperfections than timber beams with rectangular cross-sections. And if one for some reason feel uncertain about the applicability of the k_{crit} -curve it should be safe to use the upper k_c -curve for flexural buckling of glulam.

The procedure to find the resistance with regard to lateral torsional buckling is summarized as follows: The characteristic short term properties should be used when calculating the slenderness according to Equation 5.9, since the slenderness only reflects geometry, boundary conditions and load arrangement. But after determining the reduction factor k_{crit} as a function of λ the reduction should be applied in Equation 5.8, where $M_{y,Rd}$ should be calculated based on design values modified for duration of load with k_{mod} according to section 2.4.4, also see Section 3.2 in Volume 2. It is recommended to determine the slenderness parameter λ under instantaneous (short term) conditions only, since a determination under final conditions (long term) would have a small and insignificant impact on the ultimate resistance. But note that in finding $M_{y,Rd}$ itself both “inst” and “fin” conditions may have to be considered.

5.1.1.3 Shear resistance of web panels without open web joints

Two kinds of failures are possible for a uniform web panel without any open joints, either a pure panel shear failure or an out of plane bending failure caused by shear buckling. For web panels with open joints the reader is referred to Thielgard and Larsen (1978). Approximately, we may assume panel shear failure if $h_w/b_w \leq 35$ and shear buckling failure if $35 < h_w/b_w \leq 70$.

Other kinds of buckling phenomena may limit the resistance for values above 70. But web panels having a slenderness above this limit are rarely used in practical applications. Actually, a slenderness h_w/b_w above 70 should be avoided unless the designer is experienced enough to handle all instability scenarios that may occur. Note that the limit 35 is supposed to be valid for all board materials, but values adjusted to better match the properties of individual board materials are given in Table 5.2 on page 176 in this volume (Volume 1).

Let us start with a pure panel shear failure under linear elastic conditions. The shear stresses counteracting the shear force $V_{z,Ed}$ are then distributed over the cross-section as shown in Figure 5.7.

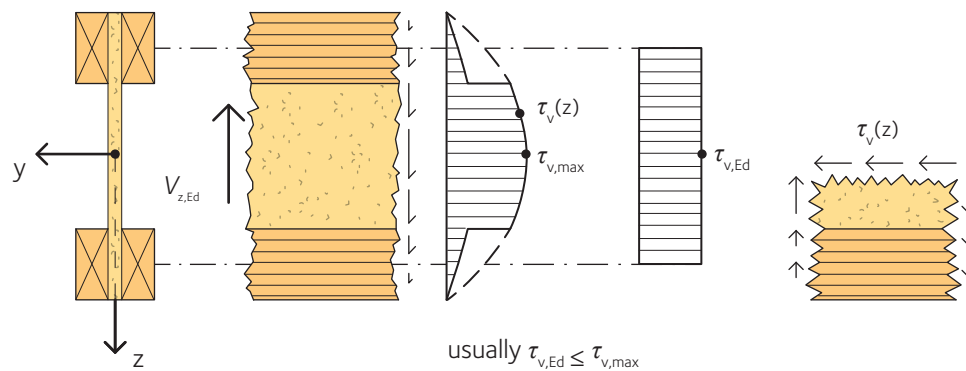


Figure 5.7: A piece of an I-beam subjected to shear force.

The simple failure criterion is that $\tau_{v,Ed} \leq f_{v,min,d}$. The shear stress can be obtained using the well-known shear formula as given by:

$$\tau_v(z) = \frac{V_{z,Ed} S_{y, fic}(z)}{I_{y, fic} b_w} \quad \text{but} \quad \tau_{v,Ed} \approx \frac{V_{z,Ed}}{A_w} = \frac{V_{z,Ed}}{b_w (h_w + h_f)} \leq f_{v,min,d} \quad \text{Eq. 5.10}$$

where the left expression gives the shear stress parallel to the x- or z-axis at any point z in the web panel. $S_{y, fic}(z)$ is the static moment of all area on either side of the z-coordinate, which is measured from the centre of the fictitious cross-section. The only problem with this formula, aside from its complexity, is that it overestimates the magnitude of the shear stress by some 5 to 10 percent for ordinary I-beams. The reason is that it is derived under the assumption that no deformations are caused by the shear stresses and that only axial strains caused by the bending moment contribute. In reality shear stresses will cause shear straining, which in turn will reduce the shear stress below the value given by the left part in Equation 5.10. If we instead assume all shear stresses to be uniformly distributed over the cross-sectional area of the web panel, A_w measured from flange centre to flange centre, a much simpler relation is obtained. We have in this simpler failure criterion neglected the slightly parabolic shape of the stress distribution τ_v in the web panel. But this is justified by neglecting the shear contribution from both outer flange parts. And it is also justified by the fact that the actual shear stresses are smaller, as described above. The simplified version of Equation 5.10 is recommended for practical design, as in Eurocode 5. Note that $f_{v,min,d}$ is the minimum value of the panel shear strength taken parallel or perpendicular to the beam axis. The shear strength of plywood varies considerably with the angle between its fibre orientation and the longitudinal beam axis.

The shear strength of plywood is more than twice as large as the tabulated value for an angle of 45°. In Eurocode 5 $f_{v,min,d}$ is denoted $f_{v,0,d}$ and no distinction is made between panel shear stresses acting parallel and perpendicular to the beam axis.

Figure 5.8 shows two side views and one end view of typical I-beams for which shear buckling is limiting the resistance. The buckling pattern is clearly indicated. Note that the web panel must under all circumstances be furnished with transverse web stiffeners at the supports, whenever shear buckling is limiting the resistance. But more stiffeners are rarely needed, unless large concentrated forces are applied to the beam in which case additional stiffeners may be needed at those locations. Note that web stiffeners may be needed at the supports even if shear buckling is no problem. This is because of crushing or local buckling of the web panel due to concentrated loading, that is patch loading may limit the resistance. The best practice to avoid patch loading failure where concentrated force are applied, like at supports, is to always use web stiffeners at these locations. How to check the patch load resistance if no stiffeners are used is not explicitly covered in this book.

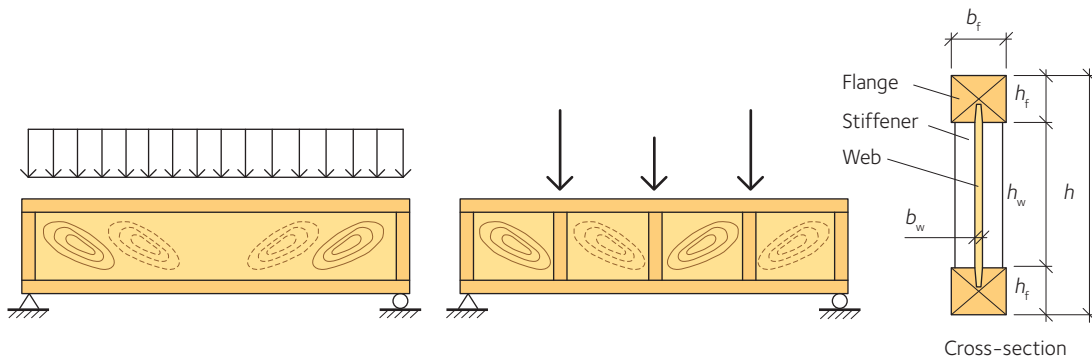


Figure 5.8: I-beams for which shear buckling is limiting the resistance, that is $35 < h_w/b_w \leq 70$.

By rewriting the right part of Equation 5.10 and introducing a reduction factor χ_v covering the shear buckling phenomenon we have:

$$V_{z,Ed} \leq V_{z,Rd} = \chi_v A_w f_{v,min,d} = \chi_v b_w (h_w + h_f) f_{v,min,d} \quad \text{Eq. 5.11}$$

Since shear buckling is an instability problem we may proceed in the same way as for lateral torsional buckling in the previous section, that is follow the chain: critical value \rightarrow slenderness parameter \rightarrow reduction factor, that is using $\tau_{cr} \rightarrow \lambda_v \rightarrow \chi_v$, where τ_{cr} is the critical shear buckling stress that will cause buckling of a web panel without imperfections, that is a linear elastic and perfectly flat plate having no residual stresses. The value of τ_{cr} catches the influence of the plate's geometry, boundary conditions and the shape of the in-plane stress field acting across the plate. The critical shear buckling stress should be determined based on characteristic values of the web panel's elastic properties, like E and G . Formulas and accompanying values of the buckling coefficient k_τ used to calculate τ_{cr} may be taken from some relevant literature, for instance Lekhnitsky (1968). However, two cases that cover most practical applications are given here. In both cases the plate is rectangular, simply supported around all 4 edges and subjected to a uniform field of shear stress as shown in Figure 5.9. These conditions are similar to those experienced by a cut-out part from any of the web panels indicated in Figure 5.8.

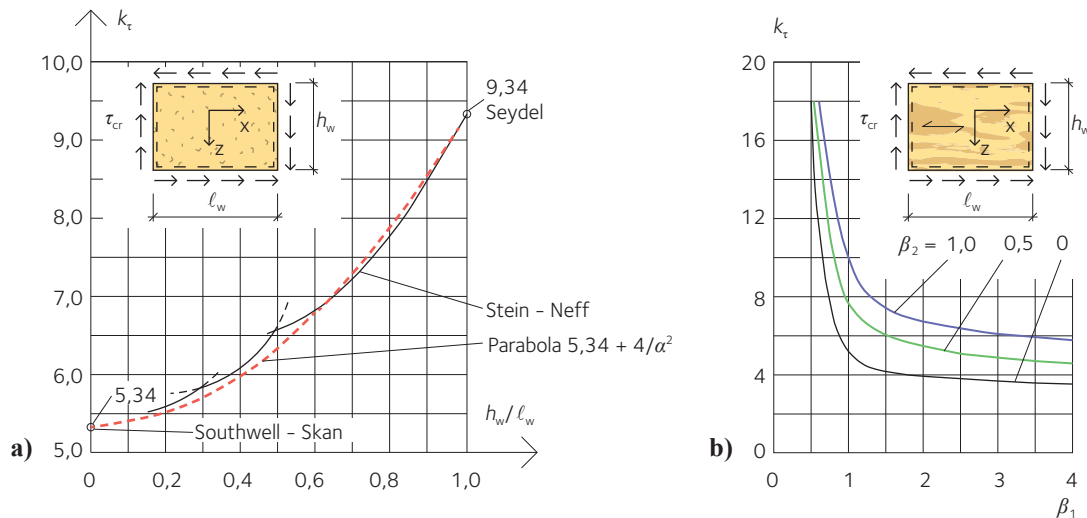


Figure 5.9: Buckling coefficient k_τ for a simply supported plate subjected to uniform shear stress around all four edges. a) Isotropic plate, b) orthotropic plate.

For isotropic plates, see Figure 5.9 a), the buckling coefficient and its corresponding shear buckling stress can be determined as:

$$k_\tau = 5,34 + 4,0 \left(\frac{h_w}{l_w} \right)^2 \quad \text{and} \quad \tau_{cr} = k_\tau \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{b_w}{h_w} \right)^2 \quad \text{Eq. 5.12}$$

where ν is the Poisson's ratio, which for hardboard can be set to 0,20 and to zero for particle-board if no better values are available. Note that wood based board materials can be viewed as isotropic if the fibre orientations are random within the plane of the plate (like for particle-board and hardboard, but not for plywood and OSB which have a clear grain direction). The buckling coefficient is simply a parabolic approximation of the theoretically "true" result shown in Figure 5.9 a). For orthotropic plates it is a bit more complicated but the buckling coefficient can be taken from Figure 5.9 b), where parameters β_1 and β_2 together with the shear buckling stress are given by:

$$\left\{ \begin{array}{l} \beta_1 = \frac{l_w}{h_w} \sqrt[4]{\frac{D_x}{D_z}} \\ \beta_2 = \frac{D_{xz}}{2\sqrt{D_x D_z}} \end{array} \right\} \text{and Figure 5.9 b)} \Rightarrow k_\tau \quad \text{and} \quad \tau_{cr} = k_\tau \frac{\pi^2 \sqrt[4]{D_x^3 D_z}}{b_w h_w^2} \quad \text{Eq. 5.13}$$

In Equation 5.13 D_x is the plate bending stiffness about the x-axis, D_z is the plate bending stiffness about the z-axis and D_{xz} is the torsional stiffness of the web panel. The critical buckling stresses predicted by Equations 5.12 and 5.13 are in general greater because of the torsional restraints provided by the flanges and the stiffeners. For an isotropic web panel having all four edges fully built in (infinite rotational stiffness of flanges and stiffeners) the critical stress is 60 to 70 percent greater than calculated by Equation 5.12, but that is in practice not to achieve.

The slenderness parameter is defined as:

$$\lambda_v = \sqrt{\frac{f_{v,\min,k}}{\tau_{cr}}} \quad \text{Eq. 5.14}$$

where $f_{v,\min,k}$ is the characteristic minimum panel shear strength taken parallel to the x- or z-axis. Finally the reduction factor χ_v , accounting for all imperfections, is obtained using a proper design curve. In Höglund (1990) it is shown that:

$$\chi_v = \min\left(\frac{0,8}{\lambda_v}; 1,0\right) \quad \text{Eq. 5.15}$$

is a simple and suitable curve, which compared to experimental results and advanced numerical simulations give results slightly on the safe side. The design curve is shown in Figure 5.10 together with the critical curve for a plate without imperfections. A comparison of the two curves indicates large post buckling resistance for $\lambda_v \geq 1,25$ and no influence of shear buckling for $\lambda_v \leq 0,8$, for $\lambda_v < 1,25$ the resistance is smaller than the critical value. Note that the post buckling resistance can only be utilised if the web panel is equipped with transverse web stiffeners at all supports, but more stiffeners are generally not needed. However, if more and closely spaced stiffeners are added an increase in resistance is possible, because of a larger critical stress and contribution from flange bending (vierendeel beam effect), as described later in this section, see Figure 5.11.

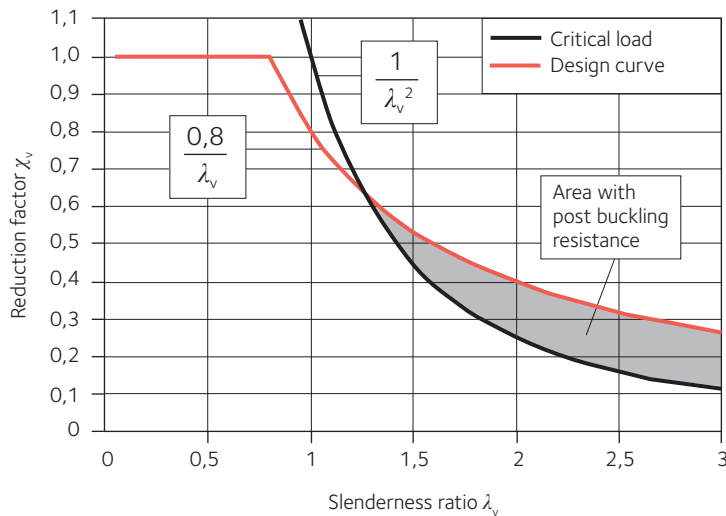


Figure 5.10: Design shear buckling curve $0,8/\lambda_v$, compared to the critical curve $1/\lambda_v^2$.

The reduction factor χ_v can for many commonly used designs be further simplified, as to obtain a lower limit value of the resistance. Let us check an I-beam having an 8 mm thick hard-board web of strength quality HB.HLA2 according to SS-EN 13986. The characteristic panel shear strength, modulus of elasticity and shear modulus are $f_{v,\min,k} = 16$ MPa, $E = 3\ 680$ MPa and $G = 1\ 520$ MPa, respectively. Poisson's ratio is, from linear elasticity, estimated to $\nu = E/(2G) - 1 = 0,21$. Substituting Equations 5.12 and 5.14 into Equation 5.15 and at the same time introducing numerical values for $f_{v,\min,k}$, E and ν gives Equation 5.16. But before substitu-

tion, the critical stress according to Equation 5.12 is increased by 25 percent to account for some torsional rigidity coming from the flanges and the buckling coefficient k_τ is set to 5,34. For a long plate (like $\ell_w > 2h_w$) k_τ is rapidly approaching 5,34 according to Figure 5.9 a), hence 5,34 is a safe assumption for an ordinary I-beam having transverse stiffeners only at the supports:

$$\chi_v = \frac{0,8}{\lambda_v} = 0,8 \sqrt{\frac{1,25 \tau_{cr}}{f_{v,min,k}}} = 0,8 \sqrt{\frac{1,25 k_\tau \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{b_w}{h_w}\right)^2}{f_{v,min,k}}} =$$

Eq. 5.16

$$0,8 \sqrt{\frac{1,25 \times 5,34 \frac{\pi^2 \times 3680}{12(1-0,21^2)} \left(\frac{b_w}{h_w}\right)^2}{16}} = 29,1 \frac{b_w}{h_w}$$

If repeating this process for different panel materials, isotropic as well as orthotropic, the number in place of 29,1 will end up between 28 to 73 and in the majority of cases between 30 and 40. More detailed values of this coefficient are given in the two rightmost columns of Table 5.2 on page 176. In Eurocode 5 a value of 35 has been chosen to be applicable to all wood based panel materials. The only difference being that for orthotropic materials Equation 5.13 is used for the critical load and the buckling coefficient k_τ is taken as 3,81, which is the smallest value in Figure 5.9 b). Finally, replacing 29,1 in Equation 5.16 by 35 and substituting into Equation 5.11 gives:

$$V_{z,Ed} \leq V_{z,Rd} = 35 b_w^2 \left(1 + \frac{h_f}{h_w}\right) f_{v,min,d} \quad \text{Eq. 5.17}$$

in which $35 b_w$ can be thought of as an effective depth of the web panel with regard to shear buckling. Note also that from Equation 5.16 it immediately follows that shear buckling is no problem if $h_w/b_w \leq 35$, that is if 29,1 is replaced by 35. Note also that avoiding slender web panels for which $h_w/b_w > 70$ means that χ_v is always greater than 0,5, alternatively λ_v is always smaller than 1,6. It is possible to use more slender web panels, but then local buckling from axial stresses (bending stresses) and web buckling caused by transverse compressive stresses from distributed loads and beam curvature must be thoroughly checked. Concentrated transverse loads are no problem because web stiffeners must always be used at such locations if the web panel is so slender that shear buckling is possible, that is $h_w/b_w > 35$.

Beams having closely spaced transverse stiffeners have greater resistance than predicted by Equation 5.17. The reasons are two: first, the resistance should be based on Equation 5.11 including the increased shear buckling resistance gained by the additional stiffeners, and second, the flanges will now contribute more to the shear resistance because of local bending between adjacent stiffeners as shown in Figure 5.11.

A greater portion of the shear force is carried by the flanges after buckling of the web panel. It is as if the shear modulus of the web panel gets smaller due to the growing shear buckles.

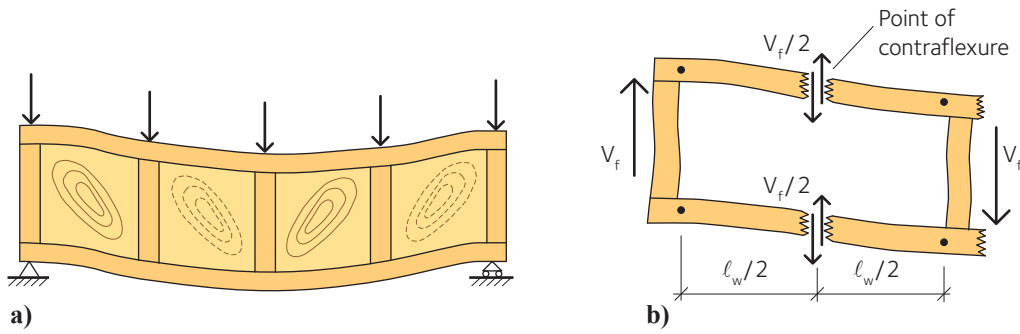


Figure 5.11: Local flange bending between transverse web stiffeners contributes to the shear resistance of the web, if shear buckling is limiting the resistance.

Stiffeners and flanges constitute an ordinary framework with considerable stiffness, greater stiffness with decreasing spacing of stiffeners. The web panel connects flanges and stiffeners to fairly stiff frame corners. The maximum shear force that can be carried by the flanges alone, can be estimated from the bending resistance of a flange. Half the shear force is acting at the point of contraflexure, where the bending moment in the flange is zero, see Figure 5.11 b). At any of the 4 points indicated we have:

$$\sigma_f = \frac{M_f}{W_f} = \frac{\frac{V_f \ell_w}{2}}{\frac{b_f h_f^2}{6}} \leq f_{f,md} \quad \Rightarrow \quad V_f \leq \frac{2}{3} f_{f,md} \frac{b_f h_f^2}{\ell_w} \quad \text{Eq. 5.18}$$

The bending moments can as a rule not reach the full resistance of the flange, because the shear deformation of the web panel will not be large enough. It is, therefore, reasonable not to use more than a little less than half of $2/3$, say $0,3$. The total shear resistance is now the sum of Equations 5.11 and 5.18. But the panel shear strength according to the right part of Equation 5.10 cannot be exceeded. Hence we get:

$$V_{z,Rd} = \min \left\{ \begin{array}{l} \chi_v V_{z,max,Rd} + 0,3 f_{f,md} \frac{b_f h_f^2}{\ell_w} \\ V_{z,max,Rd} = b_w (h_w + h_f) f_{v,min,d} \end{array} \right. \quad \text{Eq. 5.19}$$

5.1.1.4 Shear strength of flange-to-web connections

For a composite beam it is important that the shear stresses in the web panel can be transferred into the flanges without any failure of the connecting glue line. Only glued connections are considered in this section, mechanical connections are treated in Section 5.2. Shear stresses to be transferred across sections 1–1 and 2–2 in Figure 5.12 a) can be estimated by the traditional shear formula, which if expressed as a failure criterion takes the form:

$$\tau_{v,Ed} = \frac{V_{z,Ed} \Delta S_{y,i, fic}}{I_{y, fic} h_{gl}} \leq f_{gl,min,d} \quad \text{if} \quad h_{gl} \leq \frac{4b_w}{n_{gl}} \quad \text{Eq. 5.20}$$

where $f_{gl,min,d}$ usually is the planar (rolling) shear strength of the web panel if not for some odd reason the shear strength of the flange material in the longitudinal direction is smaller. Equation 5.20 is valid if the shear stresses are assumed to be uniformly distributed along the depth of each glue line h_{gl} . The parameter n_{gl} is the number of glued surfaces per web panel and flange, that is 1 for a box-section and 2 for an I-section. In this case all the glue lines are of the same depth, which is not a requirement. The parameter $\Delta S_{y,i, fic}$ is the first area moment of the flange part belonging to each glue line i , $i = 1$ or 2 taken with respect to GC. It should be calculated for a part of the flange extending from the glue line to a point of zero shear stress, usually a free edge or a line of symmetry. In the upper part of Figure 5.12 a) is shown how the shear stresses are distributed along the flange width, that is stresses on any sectional cut parallel to the z-axis. Note also that Equation 5.20 gives the “real” stress and not a fictitious stress, because the stress is uniformly smeared out along the real glue line h_{gl} as depicted in Figure 5.12 b). The assumption of uniformly distributed shear stresses works fine as long as $h_{gl} \leq 4b_w / n_{gl}$, which was shown by Foschi (1970). For greater glue line depths there is a non-negligible concentration of the shear stresses towards the re-entrant corners where the flange meets the web panel. The concentration is more severe for a double sided glue line (I-section) compared to a single sided glue line (box-section). For $h_{gl} > 4b_w / n_{gl}$ Foschi (1970) has suggested a modified version of Equation 5.20, which reads:

$$\tau_{v,Ed} = \frac{V_{z,Ed} \Delta S_{y,i, fic}}{I_{y, fic} h_{gl}} \left(\frac{n_{gl} h_{gl}}{4b_w} \right)^{0,8} \leq f_{gl,min,d} \quad \text{if } h_{gl} > \frac{4b_w}{n_{gl}} \quad \text{Eq. 5.21}$$

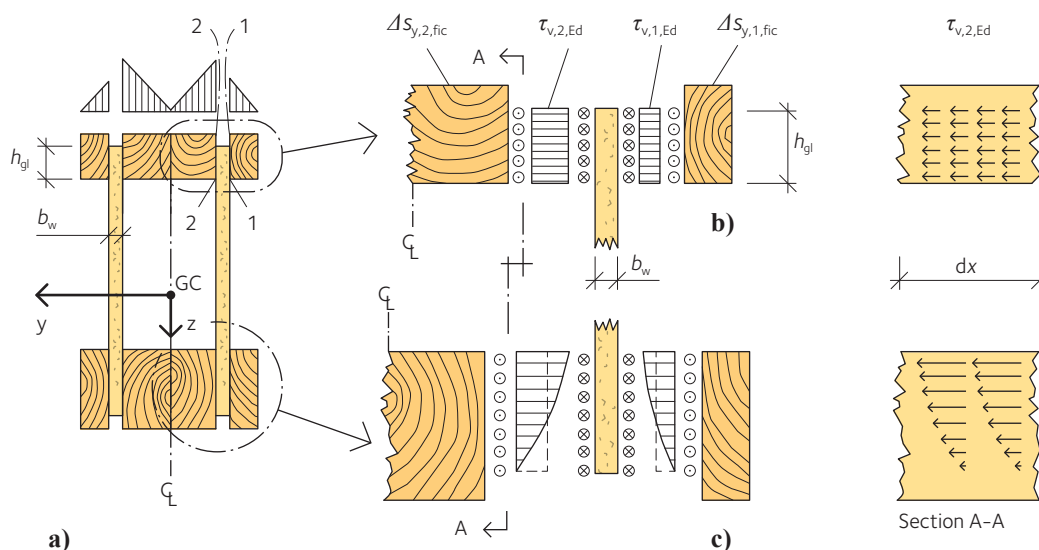


Figure 5.12: Shear stresses in glued flange-to-web connections. a) Entire cross-section, b) sections 1-1 and 2-2 cut open indicating distribution and direction of shear stresses, c) as in (b), but for the bottom flange having a deep glue line.

A correction is possible to utilise for plywood-webbed beams having the grain direction of the face veneers parallel to the flanges. The reason is that the veneer which is sensitive to planar (rolling) shear is located one veneer thickness away from the glue line surface where the shear stress is smaller than at the surface. The shear stress obtained from Equations 5.20 or 5.21 can be reduced as shown in Figure 5.13 by assuming a linear decline of the shear stress, which results in:

$$\tau_{v,red,Ed} = \tau_{v,Ed} \left(\frac{\frac{b_w}{n_{gl}} - b_{sv}}{\frac{b_w}{n_{gl}}} \right) \quad \text{where} \quad \begin{cases} n_{gl} = 1 & \text{single sided glue line} \\ n_{gl} = 2 & \text{double sided glue line} \end{cases} \quad \text{Eq. 5.22}$$

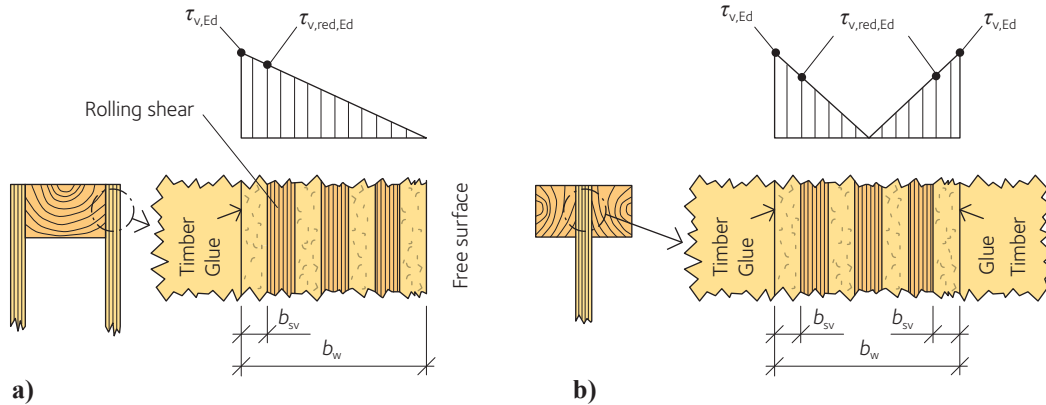


Figure 5.13: A possible reduction of shear stresses for plywood. a) One glued and one free surface, b) two glued surfaces.

Note that the reduction given by Equation 5.22 might be a bit too optimistic for a double sided glue line if the number of veneers is smaller than 6.

5.1.1.5 Bending and shear deflections

The shear deflection is not negligible for full composite beams having webs of relatively slender wood based sheet materials, and must be added to the usual bending deflection. In addition to this the different flange and web materials have different creep properties, making it necessary to distinguish between “instantaneous” and “final” deflection. Instantaneous means that no creep has yet occurred and final means full creep deformations accumulated during a period of about 50 years. This means that two different second area moments have to be calculated for the transformed cross-section using “inst” and “fin” values of the elastic moduli in Equation 5.2. The proper elastic modulus to use for each case and each material is given in Table 5.4 on page 196. Finally, the deflections should be calculated as described by the principles given by Equations 5.85 and 5.86 on page 197. These expressions are, however, indexed as to make sense for beams subjected to partial composite action. But by changing index “fca” to “bending”, “slip” to “shear” and erasing factor “2” from the last three terms of Equation 5.86 they are ready to use for full composite beams as well. Make sure to use k_{def} for the reference material used in Equation 5.2 in all the bending terms and k_{def} for the web material in all the shear terms of Equation 5.86.

Once $E_{0,mean} I_{y,fig,inst}$ and $E_{0,mean} I_{y,fig,fin}$ are known the bending deflection is easily calculated using traditional elementary cases for beam bending found in the literature. $E_{0,mean}$ is the mean modulus of elasticity of the reference material taken directly from material standards, see for instance Section 3.4 in Volume 2. Unfortunately, formulas for shear deflections are seldom given in such literature. The shear deflection can, for statically determinate beams, be found by direct integration of the shear angle caused by the shear force. It is assumed that all shear deformation is concentrated to the web panels and that the entire shear force is carried by them. The differential equation to integrate then becomes:

$$\frac{d}{dx} w_{\text{shear}}(x) = \frac{V(x)}{G_w A_w} \quad \text{Eq. 5.23}$$

where G_w is the mean shear modulus of the web directly from material standards, A_w is the total web area taken between flange centres and $V(x)$ is how the shear force varies along the beam. Direct integration of Equation 5.23 and determination of integration constants with regard to the boundary conditions result in a function for the shear deflection. For a simply supported beam subjected to uniform loading q , that is the most common support and loading conditions in the world, the maximum shear deflection at midspan becomes:

$$w_{\text{shear,max}} = \frac{q L^2}{8 G_w A_w} \quad \text{Eq. 5.24}$$

5.1.1.6 Advantages and disadvantages with I- and box-sections

As with all structural solutions there are advantages and disadvantages, some are of a statical nature and some of a practical or economical nature. Some advantages of I-sections in contrast to hollow sections are:

- easy to put web stiffeners in place at end and intermediate supports to counteract the influence of patch loading and shear buckling (actually this holds for all kinds of web buckling)
- if necessary, easy to reinforce open web joints after manufacturing
- slightly easier to reinforce or to arrange flange joints if needed
- easier to make holes in the web panel for installations.

Some advantages of hollow sections in contrast to I-sections are:

- much better torsional stiffness and bending stiffness about the minor axis, which is good for avoiding lateral torsional buckling
- easier to put thermal or sound insulation in place and generally more suitable in production
- if the web panels are made of hardboard it is somewhat more difficult to nail using an ordinary hammer, that is whenever the nail must first penetrate the hardboard.

5.1.2 Glued thin flanged elements (SSP)

Examples of SSP-elements are shown in Figure 5.14. Two basic shapes can be distinguished, open T-sections and closed hollow sections (H-sections). The flanges are usually made of particleboard, plywood, OSB or hardboard; but tension flanges made of steel sheeting are not uncommon. The web parts are usually made of sawn timber, glued laminated timber or LVL; but web parts that are themselves composite beams are not unusual.

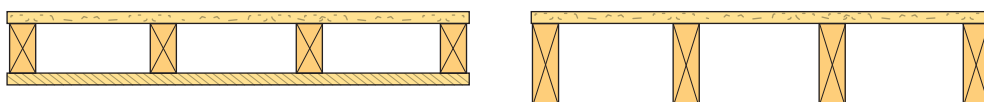


Figure 5.14: Typical cross-sections of stressed skin panels (SSP), H- as well as T- sections are shown.

Figure 5.15 shows one typical SSP-element, an H-section having two thin flanges and webs of rectangular timber pieces.

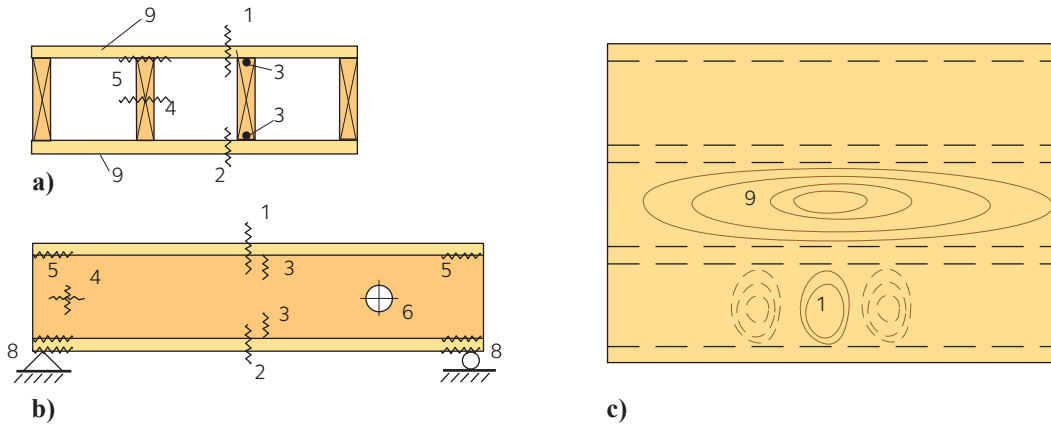


Figure 5.15: Double flanged SSP-element. a) Cross-section, b) elevation, c) top view.

Some relevant failure locations are indicated symbolically and given a specific number in Figure 5.15. The indicated failures refer to a section with stocky web parts that are not sensitive to any kind of buckling. If, however, the web parts are made of thin sheeting materials the relevant failure modes from Figure 5.3 and Section 5.1.1 should also be checked. The most relevant failure modes of elements with stocky webs are:

1. Compression failure of the compression flange, which includes the effect of local buckling, shear lag and if necessary flange curling.
2. Tension failure of the tension flange, including the influence of shear lag and possibly flange curling.
3. Bending failure of the web parts.
4. Shear failure of the web parts. The resistance is verified as for structural timber, for which design rules are given in Section 3.1.1. If the webs are made of sheet material, the rules in Section 5.1.1.3 apply.
5. Shear failure of flange-to-web interface, that is the glued connection between them. The resistance is basically checked as for I-girders, which is described in Section 5.1.1.4. Note, that if the interface itself is strong enough, there is still a risk for a pure panel shear failure of the flange material on either side of the web members both for the tension and compression flanges. The exact points for this failure mode are not indicated with numbers in Figure 5.15, but the position is almost as for point 5 but in the flanges themselves.
6. Failure of web parts with holes. No detailed design rules are given in this book, see for example instructions from the manufacturers.
7. Flexural buckling failure caused by axial loading, **not** shown in Figure 5.15. Flexural buckling is treated as for ordinary columns in Section 3.2.1.
8. Compression failure of the outer flange surfaces or the web parts in contact with the flanges caused by concentrated forces acting perpendicular to the grain, which is basically checked using the rules given in Section 3.1.3. But the contact area between flange and web can be slightly increased if the sheeting material has good compression properties in its thickness direction.
9. The influence of flange curling, that is transverse bending of both the compression and tension flanges due to the element curvature caused by the external bending moment. Of course some flange curling may also be induced by transverse loading to a flange in the regions between the webs. The basic influence of both flange curling and transverse loading is reduced bending stiffness as the distance between the flanges decreases. As a consequence, the axial loading of the flanges as well as the bending deflection of the element will increase.

10. In the serviceability limit state one has to check deflections primarily caused by bending. Shear deformations can at least for stocky webs be neglected, but should be added for slender web panels. The influence of vibrations and resonance frequencies in floors and bridge decks may be of profound importance. More information about these topics is given in Chapter 7.

Detailed design rules and recommendations are given in the following sections regarding the identified failure mechanisms under 1, 2, 3 and 9, also the deflection part of 10 is briefly discussed.

5.1.2.1 Bending resistance

The bending resistance may at the same time be influenced by shear lag, local flange buckling and flange curling. For a long element with flanges having a low width-to-thickness ratio none of them may have any substantial influence on the resistance, while the opposite is true for short elements with slender flanges. The common denominator of these influences are that they are all accounted for by changing the physical cross-sectional dimensions of the SSP-element into an effective cross-section; for which the resistance check is carried out in exactly the same way whether or not the resistance is affected by shear lag, local buckling and flange curling. The shape and basic dimensions of the effective cross-section for H- and T-shaped elements are given in Figure 5.16.

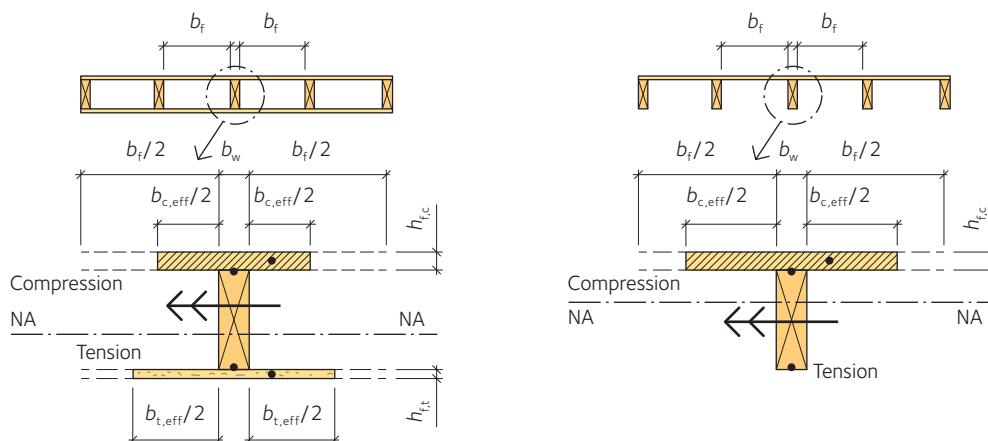


Figure 5.16: Effective cross-sections of stressed skin panels (SSP), H- as well as T-sections.

How the effective cross-section is obtained will be shown later. First the resistance checks to be made are given, if the dimensions of the effective cross-section are known. The resistance is simply verified by checking the stress levels at the locations of the dots in Figure 5.16, that is in almost the same way as for I- or box-sections. But before making these checks the effective cross-section is for convenience usually changed into a “transformed” section, to account for different elastic moduli and creep properties of the web and flange materials, as described in Section 5.1.1.1. The axial stress at each dot in Figure 5.16 is obtained by Equation 5.5 and then compared to the appropriate design strengths as:

$$\begin{aligned}
 \sigma_{f,c,d} &\leq f_{c,0,d} \text{ or } f_{c,90,d} && \text{(Compression flange)} \\
 \sigma_{f,t,d} &\leq f_{t,0,d} \text{ or } f_{t,90,d} && \text{(Tension flange)} \\
 \sigma_{w,\max,d} &\leq k_h f_{m,d} && \text{(Web edge)}
 \end{aligned}
 \tag{Eq. 5.25}$$

The last inequality of Equation 5.25 applies to webs of structural timber having a rectangular shape. Comparing the maximum edge stress to the bending strength is only relevant if the stress distribution over the web is sufficiently close to a pure bending distribution, which is usually the case for H-shaped SSP-elements. If, however, the stress distribution can be split into one tension or compression part plus one bending part the checking of the web should be made through the interaction criteria described in Chapter 3 for tension and compression. This is usually the case for T-shaped SSP-elements.

If the webs are made of wood based sheeting, then the last inequality of Equation 5.25 is replaced by a verification of the edge stresses being smaller than the tensile and compressive strengths of the board material, as appropriate.

If glued thin webbed beams are used as webs in an SSP-element the last inequality of Equation 5.25 is replaced by Equation 5.6.

Now, the methods of finding the effective cross-section with regard to the influence of shear lag, local buckling and flange curling will be discussed. We start by discussing shear lag in combination with local buckling, whereafter flange curling is treated separately.

Both shear lag and local buckling cause a non-uniform stress distribution along the flange width. The maximum stress appears around the web panels, while the minimum stress appears between them. Shear lag may influence both compression and tension flanges, while local buckling is only relevant for compression flanges.

The flanges of any beam are subjected to in-plane shear stresses, that is acting in a plane “perpendicular” to the webs. It is these shear stresses that gradually build up the axial stresses in the flange, but at the same time they will cause some small shear deformation of the flange. This shear deformation will consume some of the axial straining needed to build up full axial stress in the flange material between the webs, resulting in a non-uniform stress distribution as shown in Figure 5.17 a). From a designers point of view this fairly complicated stress distribution is handled through the concept of effective width. The real flange width is replaced by an effective width for which $\sigma_{\text{mean}} b_f = \sigma_{\text{max}} b_{\text{eff}}$ is supposed to be valid.

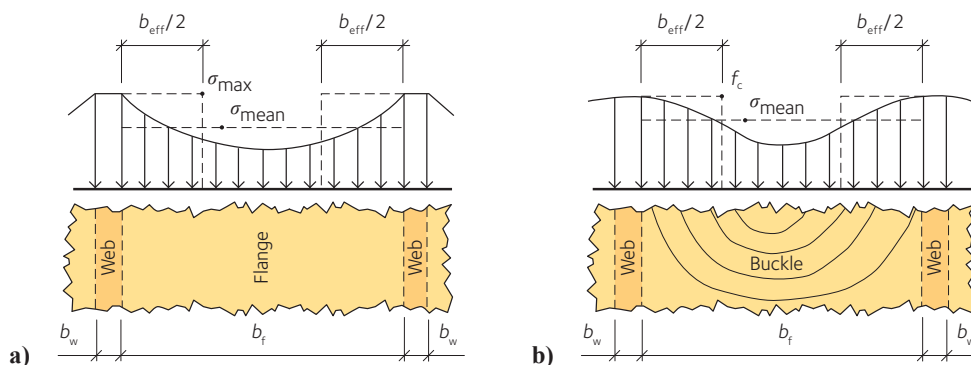


Figure 5.17: Non-uniform stress distributions, in a wide compression flange between two webs, caused by: a) shear lag only, b) local buckling only.

To find the best effective width in each particular case is difficult because it is influenced by many factors, out of which the most important are: shear properties of the flange material,

loading configuration, boundary conditions, ratio of flange width to span length and local buckling. In Eurocode 5 the effective width is simply given as a constant fraction of the span regardless of this complexity. The span is the distance between points of contraflexure or zero bending moment. In Figure 5.18 a comparison between the simple Eurocode expression and theoretically more correct relations are given for a flange made of OSB loaded either perpendicular or parallel to its main grain direction, here denoted by \perp or \parallel , respectively. The SSP-element is simply supported and loaded by either a uniformly distributed load acting along the entire span or a concentrated load at midspan, labelled q or P . It is clear that the Eurocode expression of $0,15 \ell$ gives a good effective width only if the ratio of flange width to span is about 0,16 for a q -load and 0,20 for a P -load. For longer spans the effective width is overestimated by as much as 20 %, while the effective width is underestimated for shorter spans. Flanges of hardboard, particleboard and plywood show similar results, but plywood shows larger scatter and deviations between the Eurocode curve and theoretical results. The theoretical curves were obtained by FE-analysis under linear elastic conditions.

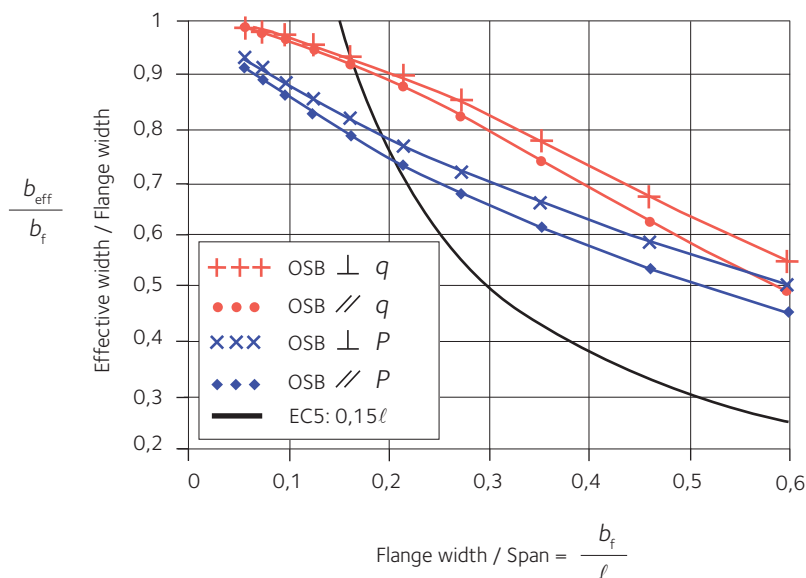


Figure 5.18: The influence of shear lag for an OSB flange in a simply supported SSP-element subjected to two different loading conditions.

More accurate predictions of the effective width are obtained by approximating the theoretical curves with straight lines on the interval of practical interest $0,05 \leq b_f/l \leq 0,30$. The function expression for such fitted results is given by:

$$\frac{b_{\text{eff}}}{b_f} = \beta_0 - \beta_1 \left(\frac{b_f}{\ell} \right) \quad \text{Eq. 5.26}$$

where the coefficients β_0 and β_1 are given in Table 5.1 together with the traditional Eurocode expressions.

Table 5.1 Effective width due to shear lag in wide compression or tension flanges. Widths according to both Eurocode 5 and Equation 5.26 are given.

Board material	Eurocode 5 b_{eff}	Type of loading ^{2) 3)}	Coefficient β_0 in (5.26)	Coefficient β_1 in (5.26)
OSB	0,15 ℓ	q	1,03	0,72
		P	0,97	1,05
Plywood, 3L ¹⁾	0,10 ℓ	q	1,05	1,15
		P	0,93	1,35
Plywood	0,10 ℓ	q	1,06	1,80
		P	0,83	1,60
Particleboard or fibreboard	0,20 ℓ	q	1,03	0,61
		P	0,98	0,90

¹⁾ 3L denotes a three veneer plywood with the face veneers oriented perpendicular to the axial stresses.

²⁾ q is a uniformly distributed load on a simply supported beam.

³⁾ P is one concentrated load at midspan on a simply supported beam.

Buckling of compression flanges gives a non-uniform distribution of axial stresses which is similar in shape to the distribution caused by shear lag, as indicated in Figure 5.17 b). Shear lag reduces the influence of local buckling because it leads to smaller compressive stresses in the part of the flange that is sensitive to buckling. The other way around local buckling will reduce the influence of shear lag. But we start to discuss local flange buckling as a stand-alone problem. As an instability problem it can be accounted for using the same principal approach as outlined for shear buckling in Section 5.1.1.3 resulting in an effective flange width ($b_{\text{eff}} = b_f \chi_c$). This process may be summarised as: critical compressive stress \rightarrow slenderness parameter \rightarrow reduction factor, that is using $\sigma_{\text{cr}} \rightarrow \lambda_c \rightarrow \chi_c$, where index c indicates compression. The slenderness parameter is defined in the same way as Equation 5.14 and the design curve is chosen as in Equation 5.15, but index v is replaced by c to indicate compression. This results in a reduction factor expressed as:

$$\chi_c = \frac{0,8}{\lambda_c} = 0,8 \sqrt{\frac{1,25 \sigma_{\text{cr}}}{\sigma_{f,c}}} = 0,9 \sqrt{\frac{\sigma_{\text{cr}}}{\sigma_{f,c}}} \quad \text{Eq. 5.27}$$

where σ_{cr} is the critical buckling stress for a long plate simply supported along all four edges and subjected to uniform compression in its longitudinal direction. The factor 1,25 increases the buckling stress by 25 % to approximately account for some of the rotational stiffness provided by stocky web members (total rotational constraint along the webs would give about 60 %). If, however, the web members are thin and slender 1,25 should be replaced by 1,0, as thin board members provide little rotational stiffness. $\sigma_{f,c}$ is the compressive stress in the flange at the load level where the bending resistance of the entire SSP-element is exhausted, under the assumption that no local buckling affects the resistance of the flange. It is always on the safe side to set $\sigma_{f,c} = f_{c,0,d}$ or $f_{c,90,d}$, as appropriate. But if the bending resistance is governed by the tension flange or webs, rather than the compression flange, a proportionately reduced value can be used. Also, if shear lag is affecting the compression flange the above value of $\sigma_{f,c}$ can be reduced by the factor $b_{\text{eff, shear lag}}/b_f$ to take the beneficial effect of shear lag into account.

Note that it is not self-evident that the design curve should be $0,8/\lambda_c$, but it must under all circumstances have a similar shape. A comparison to the more complicated curves used for steel in Eurocode 3, observing that those curves also take the influence of residual stresses into account, indicates that $0,9/\lambda_c$ might be a better choice. But choosing $0,8$ is slightly on the safe side, especially for slender flanges, and can also be said to implicitly account for some small influence of flange curling. This simple formula for the design curve makes it possible to express the effective width as a **constant** \times **flange thickness**, as we shall see next.

The critical stress σ_{cr} is calculated based on linear elastic theory and a lower limit, on the safe side, is obtained by assuming the flange to be a long plate simply supported along all four edges. But the value can easily be increased or decreased if other boundary conditions are more relevant, for example increased if the flange is equipped with closely spaced transverse stiffeners. Compression flanges in SSP-elements are either made of isotropic or orthotropic sheet materials and the critical stress is conveniently expressed as:

$$\sigma_{cr} = k_{\sigma,iso} \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{h_f}{b_f} \right)^2 \quad \text{Isotropic plate}$$

Eq. 5.28

$$\sigma_{cr} = k_{\sigma,orth} \frac{\pi^2 \sqrt{D_x D_y}}{h_f b_f^2} \approx k_{\sigma,orth} \frac{\pi^2 \sqrt{E_x E_y}}{12} \left(\frac{h_f}{b_f} \right)^2 \quad \text{Orthotropic plate}$$

where D_x and D_y is the bending stiffness of the flange about its x- and y-axis, respectively. E_x and E_y are the elastic moduli for bending about the two main axes of the sheet material as if it would be homogeneous over its thickness, that is the moduli usually given in various standards and test reports for timber based sheet materials. The buckling coefficient $k_{\sigma,iso} = 4,0$ applies to long isotropic plates. For orthotropic plates the buckling coefficient $k_{\sigma,orth}$ depends on two parameters:

$$\beta_1 = \frac{\ell}{b_f} \sqrt[4]{D_x D_y} \quad \text{and} \quad \beta_2 = \frac{D_{xy}}{2\sqrt{D_x D_y}} \approx \frac{2G}{\sqrt{E_x E_y}} \quad \text{Eq. 5.29}$$

where G is the shear modulus. But, for long plates ($\ell > 2b_f$) that are only loaded by uniform compression in their longitudinal direction, the value of β_1 does not matter and a fairly good approximation of the buckling coefficient is:

$$k_{\sigma,orth} = 2,0 + 2,0\beta_2 = 2,0 + \frac{4,0G}{\sqrt{E_x E_y}} \quad \text{Eq. 5.30}$$

By using $k_{\sigma,iso} = 4,0$ and $k_{\sigma,orth}$ from Equation 5.30 in Equation 5.28 and substituting the obtained critical stresses into Equation 5.27 the reduction factor for the effective width is a **constant** times h_f/b_f and the effective width itself is just a **constant** \times h_f . Obviously the **constant** is a unique number for every sheet material on the market. In Table 5.2 the results of such calculations are summarised for some common sheet materials together with the effective widths recommended in Eurocode 5. It appears that the precision of the Eurocode 5 constant is rather poor and too much on the unsafe side for OSB, particleboard

and fibreboard. The reason for this difference is that the design curve is taken as $1,0/\lambda_c$ rather than $0,8/\lambda_c$ for the Eurocode 5 constant, which is definitely too optimistic if $\lambda_c < 1,30$ and the unfavourable effect of flange curling is not considered; or if the favourable effect of shear lag is not implicitly taken into account.

Table 5.2 Effective width due to buckling of compression flanges according to Eurocode 5 compared to values obtained by Equations 5.30, 5.28 and 5.27 applied to 139 different board products for which material properties are available. *Effective width* = $const \times h_f$. The two rightmost columns show the effective width ($const \times h_w$) in case of shear buckling, as described in Section 5.1.1.3. Minima and maxima obtained for different products in each category are given.

Board material ^{1) 2) 3)}	Eurocode 5 <i>const</i>	Compression ⁴⁾		Shear	
		Min <i>const</i>	Max <i>const</i>	Min <i>const</i>	Max <i>const</i>
OSB, //	25	19	21	32	36
OSB, ⊥	25	21	23	40	45
Plywood, 3, //	20	12	15	22	27
Plywood, 3, ⊥	25	17	22	44	61
Plywood, //	20	17	24	35	63
Plywood, ⊥	25	19	32	44	73
Particleboard	30	23	26	35	40
Fibreboard	30	20	28	28	44

¹⁾ // The flange major axis of bending oriented transversely to the SSP-element, for example face veneers of plywood oriented parallel to the webs.

²⁾ ⊥ The flange major axis of bending oriented longitudinally along the SSP-element, for example face veneers of plywood oriented perpendicular to the webs.

³⁾ 3 Denotes plywood made of only 3 veneers.

⁴⁾ It is assumed that the compression flange governs the resistance such that $\sigma_{f,c} = f_{c,0,d}$ or $f_{c,90,d}$, as appropriate.

Following Eurocode 5, the effective width should be taken as the smallest width obtained either from the Eurocode column of Table 5.1 or Table 5.2 for compression flanges and only from Table 5.1 for tension flanges. In addition, the influence of flange curling is not even mentioned. The main advantage of this approach is its simplicity, but its precision might be very poor. It is pointless to perform complicated and tedious analyses of SSP-elements with regard to the creep properties and moduli of elasticity of the materials making up the cross-section if the effective width can be off target by as much as 50 %, while the influence of creep gives a variation of usually no more than 10 % in the calculated bending resistance.

Instead, the bending resistance can be more precisely targeted by following the recipe:

1. Calculate, with regard to shear lag, the effective width of both the tension and compression flanges through Equation 5.26 using coefficients from Table 5.1.
2. Reduce, if relevant, the effective width of the compression flange further with regard to buckling according to Equation 5.27. In Equation 5.27: use Equation 5.28 to find the critical stress based on buckling coefficients $k_{\sigma,iso} = 4,0$ and/or $k_{\sigma,orth}$ from Equation 5.30, and set $\sigma_{f,c} = (f_{c,0,d}$ or $f_{c,90,d}) \times b_{eff,shear\ lag}/b_f$, as appropriate. If after this reduction the resistance is still governed by the tension flange, $\sigma_{f,c}$ can be reduced even further to reflect the “real” stress level in the compression flange. But note that such an extra reduction is iterative and usually quite impractical for ordinary hand calculations, especially since the effect on the final result is quite small.
3. Calculate the position of the neutral axis and the second area moment of inertia of the effective and transformed cross-section. Determine the influence of flange curling

according to the procedure given below. Move each flange for which flange curling must be considered towards the neutral axis by a distance estimated as a fraction of u_{\max} according to Equation 5.31. Recalculate the position of the neutral axis and the second area moment of inertia with regard to the new flange positions.

4. Check the bending resistance according to Equation 5.25.

Flange curling is a problem for thin and wide flanges and affects both tension and compression flanges. The axial bending stresses in the flanges will always have a very small component acting towards the neutral axis of the SSP-element, which is a consequence of the bending curvature. It is equivalent to a distributed load applied to the flange surface, causing transverse bending of the flange. This results in a reduced distance between the flanges, which in turn reduces the second area moment and increase the axial flange stresses. The maximum transverse bending deflection of the flange in between two neighbouring web panels can be estimated from elementary beam bending theory applied to a flange strip supported on a number of webs. Following the approach taken in StBk-N5 (1980) the maximum flange deflection can be estimated to:

$$u_{\max} = K \left(\frac{\sigma_{f,\text{mean}}}{E_f} \right)^2 \frac{b_f^4}{h_f^2 z_f} \quad \text{Eq. 5.31}$$

where $\sigma_{f,\text{mean}}$ is the average axial stress in the flange calculated as $(\sigma_{f,c,d}$ or $\sigma_{f,t,d}) \times b_{\text{eff}}/b_f$, that is the average stress with respect to shear lag and buckling. E_f is the modulus of elasticity with respect to flange bending about an axis parallel to the webs. Set E_f to E_{mean} under instantaneous conditions in service class 1, $0,8 E_{\text{mean}}$ in service class 2 and $0,4 E_{\text{mean}}$ for plywood that can be used in service class 3. Set E_f to $E_{\text{mean}}/(1 + \psi_2 k_{\text{def}})$ under final conditions with ψ_2 determined for the load causing the largest stress. z_f is the absolute value of the distance from the neutral axis to the centre of the flange. K is a modification factor taking the support conditions of the flange into account and is given below.

If u_{\max} is greater than 5 % of the distance between flange centres it is recommended that flange curling is considered by moving the flange a fictitious distance, that is a fraction of u_{\max} , towards the neutral axis. The recommended value of K and its corresponding fictitious distance is:

$K = 0,15$ if the flange is simply supported on two neighbouring webs. If needed move the flange $0,64 u_{\max}$

$K = 0,065$ if the flange is continues over intermediate webs and supported by at least three equally spaced webs. If needed move the flange $0,58 u_{\max}$

Note that the above recipe for finding the moment resistance is applicable to SSP-elements for which the transverse load enters into the element right on top of the webs. No external transverse loading can be applied directly to the flanges in between the webs, which would cause severe transverse bending of them. In such cases, like in floors and outer roof surfaces, the flange must first be designed such that it can carry this transverse bending from distributed and/or concentrated forces. After that the recipe can be applied, but the flange slenderness is then usually so small that only shear lag needs attention.

5.1.2.2 Bending and shear deflections

The bending deflection is determined exactly as for full composite I-beams as discussed in Section 5.1.1.5. The shear deflection is usually quite small and is negligible for elements having rather stocky webs of structural timber, glulam, etc. If, however, the webs are made of thin sheet materials the shear deflection should be calculated in the same way as described for I-beams in Section 5.1.1.5. Note that effective flange widths need only be determined with respect to shear lag when calculating deflections in the SLS. Flange buckling and flange curling need not be considered, because the stress levels are much smaller in the SLS than in the ULS, making the effect of these phenomena negligible, at least for normal SSP-elements.

5.1.2.3 Advantages and disadvantages with T- and H-shaped SSP-elements

Here some statical as well as practical advantages and disadvantages of SSP-elements are discussed.

The most difficult problem to solve for SSP-elements is the continuity of a thin tension flange. Wood based sheeting are usually available in lengths much smaller than the element length and must therefore be jointed in regions transferring large tensile stresses, which is possible but complicated and expensive. One solution is to use LVL or steel sheeting for the tension flange and thereby avoiding unwanted joints.

Some advantages of T-sections compared to H-sections are:

- Easy to add extra local reinforcement to secure load transfer of large concentrated forces that may otherwise cause local damage.
- Simpler to connect the elements to each other and to the supports.
- Easier to install and hide installations within the elements, when carried out on site.
- Combining two similar elements with limited or no contact in between them may result in very good sound insulation properties, which may also be good for fire protection.

Two advantages of H-sections compared to T-sections are:

- Better bending resistance and bending stiffness, which makes it possible to span greater distances without using much more material.
- A higher degree of prefabrication is possible, avoiding costly completions on site. Execution quality of insulation and installation works will be better due to more suitable factory conditions regarding tools, climate and control.

5.2 Structural elements with partial composite action

In timber engineering built-up cross-sections are quite common. The parts are often connected by glue, in which case the slip between the parts may not be much of a trouble. But slipping between member parts must be considered when mechanical connectors are used instead of glue or when the quality of the glue-line cannot be relied upon. In Figure 5.19 are shown some different beam structures having built-up cross-sections for which partial composite action may play a substantial role.

Suppose that we have a beam or SSP-element subjected to bending and that it is made of two or more individual parts connected to each other by some mechanical connectors, soft and

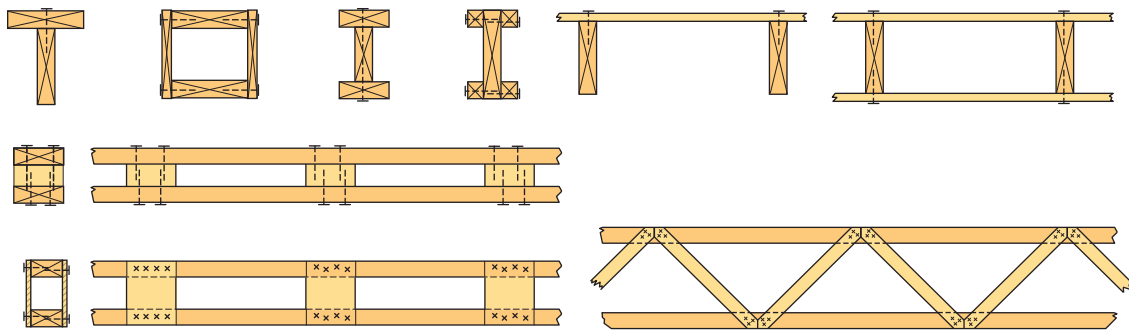


Figure 5.19: Typical composite beam elements for which partial composite action is relevant.

thick glue interfaces or some rather thick interlayer that is only capable of transferring shear stresses but no direct stress (such as the core of a sandwich element). By partial composite action is then meant that the shear deformation between separate parts is non-negligible. This shear deformation is hereafter also referred to as slip deformation or simply slip. The principal behaviour of an element having only two parts connected by one shear transferring interlayer is depicted in Figure 5.20.

The response is characterised by some slip of the intermediate layer, which may be categorised into three different cases. First, we have **full composite action (FCA)** if the shear connection is infinitely stiff. Second, we have **no composite action (NCA)** if the shear connection has no stiffness at all, that is the two layers act independently of each other and no shear forces are transferred along the joint line. Third, we have **partial composite action (PCA)** if the shear connection has a finite stiffness, that is the “true” response that must be in between the first two limiting cases. The first two cases are well covered by traditional beam bending theory and will not be treated any further in this section. Here, we shall focus on the third case characterised by partial composite action.

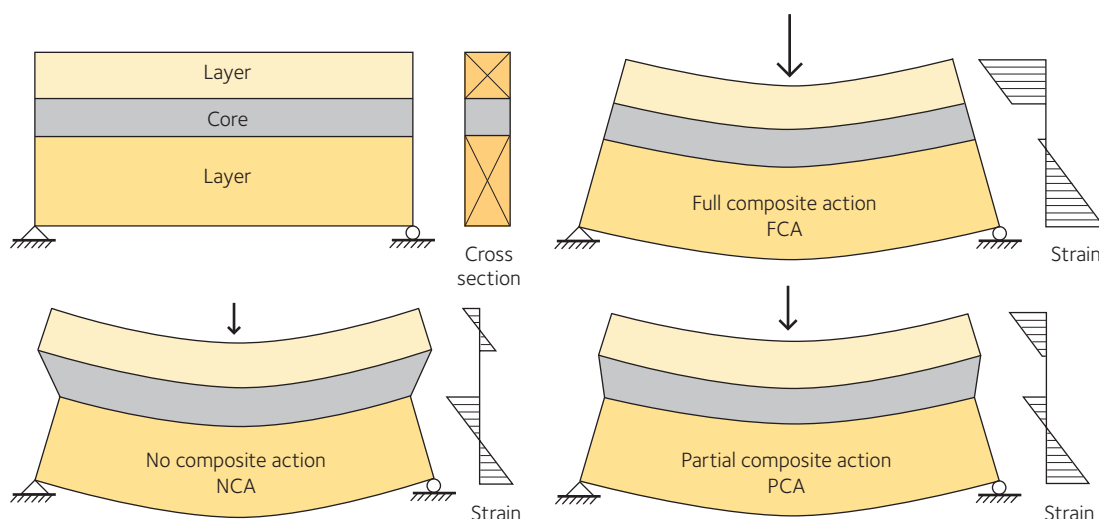


Figure 5.20: Three degrees of composite action, for equal midspan deflections.

The partial composite action is governed by either of the differential Equations 5.50 or 5.51 on page 184. Equation 5.50 was first derived by Stussi (1947), but he had no term to account for the temperature difference between the two layers. Equation 5.51 was derived by Granholm

(1949), also neglecting the influence of temperature differences. Newmark (1951), derived Equation 5.50 independently of the first two authors. After these pioneers many others have followed publishing minor or major modifications to the basic equations given here. Such modifications include the influence of an external axial load, creep, more than two layers, non-linearities or de-bonding. Also, many contributions deal with different solution procedures like symbolic closed form solutions, finite difference methods and finite element methods. Only the basic equations are needed to solve the problem treated in this book. The other mentioned references will be left without any further treatment.

As a curiosity it is worth to mention that researchers working in this field may be divided into two main categories. The first category is those working with composite beams connected by mechanical connectors like nails, screws, studs and also glue. The second group is concerned with all kinds of sandwich beams where the layers are connected by some kind of foam or honeycomb, for example expanded polystyrene (EPS). But the basic equations are the same no matter which group the researcher belongs to. Despite this the two groups almost never reference each other's work. The authors referenced here belong to the first group.

5.2.1 Built-up bending elements

We will start by deriving the governing equations for a two layered composite beam subjected to bending only. We will then go on to show certain simplifications of the analysis, making it possible to obtain approximate solutions with relative ease. Then the equations are modified such that three layered double symmetric sections can be properly analysed. The equations of a single symmetric three layered beam with unequal parts are not treated here, but are given in Oduyemi and Wright (1991).

5.2.1.1 Basic assumptions

Consider, as in Figure 5.21, an infinitesimal segment cut out along the longitudinal axis of any element. The differential Equations 5.50 and 5.51 govern the response of this small piece, when subjected to external loading. These equations are derived using the following assumptions and limitations:

- a) All material remains linear elastic.
- b) Small displacement theory is assumed.
- c) The two layers must have equal deflections, i.e. they must not separate and the model does not consider any tensile or compressive stresses in the thickness direction. Furthermore, both layers are assumed to have equal radius of curvature, which is justified by the fact that the thickness of the element is much less than the radius of curvature.
- d) No shear deformation within each layer is considered, only the shear deformation of the core is accounted for. Also, any influence of straining perpendicular to the longitudinal axis is neglected. This implies that the assumption in normal beam bending theory, that plane cross-sections remain plain, holds for each layer.
- e) The core is only assumed to have shear stiffness and its only purpose is to act as a shear connection between the two layers.
- f) The element may also be subjected to a uniform strain difference between the two layers, for instance caused by temperature differences between the two layers.

- g) In addition to the strain difference the only allowed load is a bending moment caused by some external load acting in the z -direction. External axial loads are not accounted for.
- h) The model can only account for deflections within the xz -plane and the cross-section must, therefore, be symmetric about the z -axis.

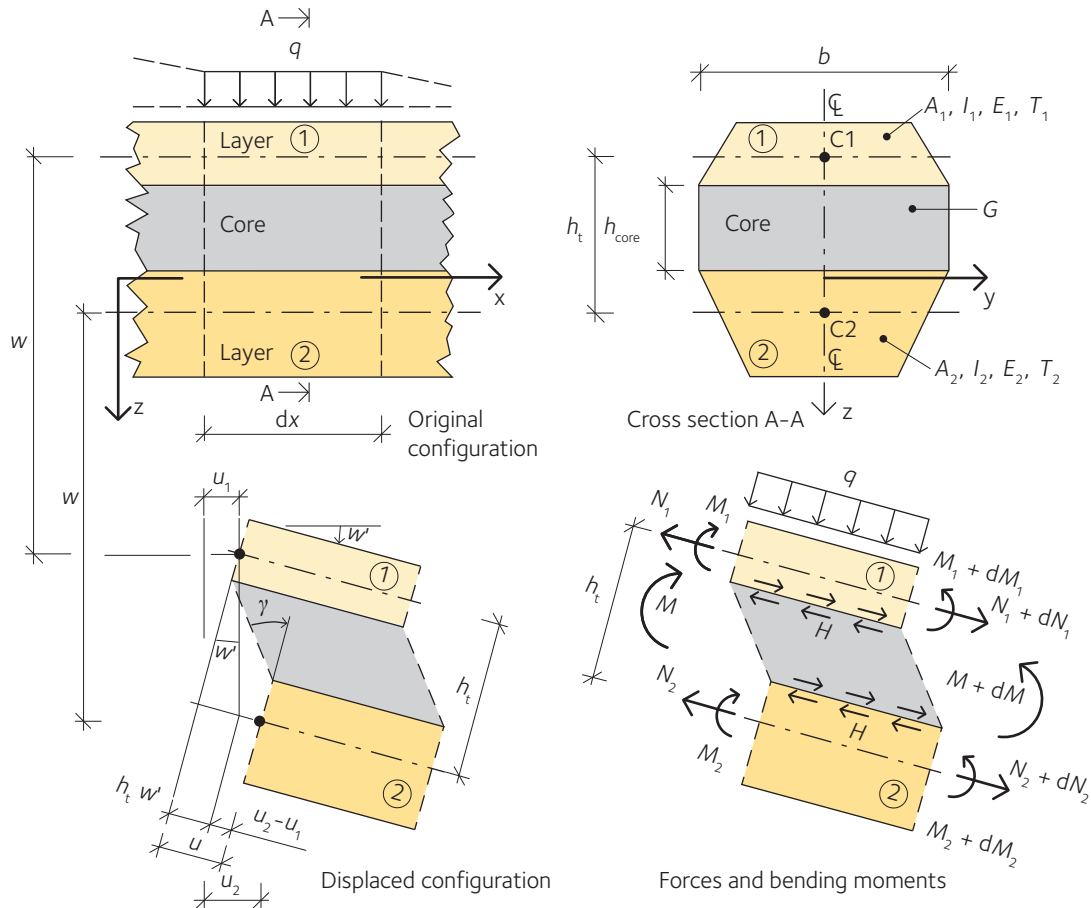


Figure 5.21: The beam cross-section and a small cut-out dx along the beam axis.

In Figure 5.21 the following notations are used (other symbols are explained as they are introduced):

A_1	Cross-sectional area of layer 1
A_2	Cross-sectional area of layer 2
b	Width of the core material
E	Modulus of elasticity, which is assumed to be equal for both layers. If equality does not hold the method of transformed cross-sections can be used
$\gamma = \gamma(x)$	Shear angle of the core
G	Shear modulus of the core material
h_{core}	Core thickness
h_t	Distance between the centres of layer 1 and 2
$H = H(x)$	Shear flow transmitted, by the core, between the two layers

I_1	Second moment of inertia, for the plane cross-sectional area of layer 1
I_2	Second moment of inertia, for the plane cross-sectional area of layer 2
$M_1 = M_1(x)$	Internal bending moment in layer 1
$M_2 = M_2(x)$	Internal bending moment in layer 2
$M = M(x)$	External bending moment affecting the entire cross-section
$N_1 = N_1(x)$	Internal axial force in layer 1
$N_2 = N_2(x)$	Internal axial force in layer 2
$q = q(x)$	Transverse loading
$u = u(x)$	Shear deformation of the core
$u_1 = u_1(x)$	Displacement of the centre of layer 1
$u_2 = u_2(x)$	Displacement of the centre of layer 2
$w = w(x)$	Deflection
x, y, z	Co-ordinate axes

The directions of the arrows in Figure 5.21 indicate positive forces, moments and displacements. The prime symbol, ', indicates differentiation of the above functions with respect to x , two prime symbols indicate that the differentiation is performed twice, etc.

5.2.1.2 Derivation of the governing differential equations for a two layered beam

From Figure 5.21 it is clear that the deformation of the core is:

$$u = u_2 - u_1 + h_t w' \quad \text{Eq. 5.32}$$

and after differentiation:

$$u' = u_2' - u_1' + h_t w'' = \varepsilon_2 - \varepsilon_1 + h_t w'' \quad \text{Eq. 5.33}$$

where:

$$\varepsilon_1 = \frac{du_1}{dx}$$

$$\varepsilon_2 = \frac{du_2}{dx}$$

are the simple engineering strains along layer centres 1 and 2, respectively. We have also assumed both layers to have the same deflection $w(x)$ and approximately the same radius of curvature. The strains may, by using traditional bar theory and positive tensile strains, be expressed as:

$$\varepsilon_1 = \frac{N_1}{EA_1} + \varepsilon_{t1} \quad \text{Eq. 5.34}$$

$$\varepsilon_2 = \frac{N_2}{EA_2} + \varepsilon_{t2} \quad \text{Eq. 5.35}$$

where ε_{t1} and ε_{t2} are uniform strains caused by temperature changes in layers 1 and 2, respectively. By substituting Equations 5.34 and 5.35 into Equation 5.33 we obtain:

$$u' = h_t w'' - \frac{N_1}{EA_1} + \frac{N_2}{EA_2} + \varepsilon_{t2} - \varepsilon_{t1} \quad \text{Eq. 5.36}$$

Now, since both ε_{t1} and ε_{t2} are constant along the beam the difference between them is replaced by one constant:

$$\varepsilon_t = \varepsilon_{t2} - \varepsilon_{t1} \quad \text{Eq. 5.37}$$

Force equilibrium for the cross-section shown in Figure 5.21 requires that:

$$N_2 = -N_1 \quad \text{Eq. 5.38}$$

because there is no external axial load affecting the beam. Substitution of Equation 5.37 and 5.38 into Equation 5.36 yields:

$$u' = h_t w'' - \left(\frac{1}{EA_1} + \frac{1}{EA_2} \right) N_1 + \varepsilon_t \quad \text{Eq. 5.39}$$

Let now the material properties of the core material enter into this derivation. When the shear flow H affects the infinitesimal cut-out in Figure 5.21 it is displaced an angle that may be written as:

$$\gamma = \frac{\text{Shear stress}}{\text{Shear modulus}} = \frac{H}{Gb} \quad \text{Eq. 5.40}$$

and by using Equation 5.40 the resulting displacement is:

$$u = \gamma h_{\text{core}} = \frac{H h_{\text{core}}}{Gb} = \frac{H}{k_{\text{core}}} = \frac{-N_1'}{k_{\text{core}}} \quad \text{Eq. 5.41}$$

where the shear stiffness of the core, k_{core} , has been introduced:

$$k_{\text{core}} = \frac{Gb}{h_{\text{core}}} \quad \text{Eq. 5.42}$$

In Equation 5.41 it is also used that the force equilibrium of layer 1, of the infinitesimal cut-out in Figure 5.21, requires that $H = -N_1'$. Differentiating Equation 5.41 once and equating the result to Equation 5.39 gives:

$$\frac{-N_1''}{k_{\text{core}}} = h_t w'' - \left(\frac{1}{EA_1} + \frac{1}{EA_2} \right) N_1 + \varepsilon_t \quad \text{Eq. 5.43}$$

Equating the external moment M and the internal moments in the cross-section, taken about the centre of layer 2, yields:

$$M = M_1 + M_2 - N_1 h_t \quad \text{Eq. 5.44}$$

The assumption that both layers have equal radius of curvature gives:

$$w'' = -\frac{M_1}{EI_1} = -\frac{M_2}{EI_2} \quad \text{Eq. 5.45}$$

and Equation 5.44 may by using Equation 5.45 be rewritten as:

$$w'' = -\frac{M}{EI_1 + EI_2} - \frac{N_1 h_t}{EI_1 + EI_2} \quad \text{Eq. 5.46}$$

Substituting Equation 5.46 into Equation 5.43 and rearranging gives:

$$N_1'' - \frac{k_{\text{core}}}{E} \left(\frac{1}{A_1} + \frac{1}{A_2} + \frac{h_t^2}{I_1 + I_2} \right) N_1 = \frac{k_{\text{core}}}{E} \frac{h_t}{(I_1 + I_2)} M - k_{\text{core}} \varepsilon_t \quad \text{Eq. 5.47}$$

Introducing the cross-sectional constants:

$$C = \frac{k_{\text{core}}}{E} \frac{h_t}{(I_1 + I_2)} \quad \text{Eq. 5.48}$$

$$\omega^2 = \frac{k_{\text{core}}}{E} \left(\frac{1}{A_1} + \frac{1}{A_2} + \frac{h_t^2}{I_1 + I_2} \right) \quad \text{Eq. 5.49}$$

makes it possible to write Equation 5.47 in its simplest form:

$$N_1'' - \omega^2 N_1 = C M - k_{\text{core}} \varepsilon_t \quad \text{Eq. 5.50}$$

This is a second order linear differential equation with constant coefficients that may be solved easily once the boundary conditions of the problem are known. In Equation 5.50 the unknown function represents the variation of the axial force along layer 1. Sometimes it may, however, be more convenient to express the governing differential equation in terms of the deflection function. This may be achieved by solving for N_1 in Equation 5.46 and substituting the result and its derivatives into Equation 5.43. The result, after introducing two new cross-sectional constants, is:

$$w'''' - \omega^2 w'' = C_{\text{Mb}} (C_{\text{M}} M - M'') + C \varepsilon_t \quad \text{Eq. 5.51}$$

where:

$$C_{\text{Mb}} = \frac{1}{E(I_1 + I_2)} \quad \text{Eq. 5.52}$$

$$C_{\text{M}} = \frac{k_{\text{core}}}{E} \left(\frac{1}{A_1} + \frac{1}{A_2} \right) \quad \text{Eq. 5.53}$$

When explicitly solving a problem it is often slightly simpler to work with Equation 5.50 than Equation 5.51. But sometimes it is convenient to express the boundary conditions in terms of known deflections and sometimes you just want to know the deflections. So if one solves for N_1 in Equation 5.50 it is necessary to have an expression that gives the deflection w once N_1 is known. This link between the deflection function and the axial force function may be derived by solving for N_1 in Equation 5.50 and substituting the result into Equation 5.46. This first step leads to:

$$w'' = \frac{-1}{E(I_1 + I_2)} \left(M + h_t \left(\frac{N_1''}{\omega^2} - \frac{C}{\omega^2} M + \frac{k_{\text{core}} \varepsilon_t}{\omega^2} \right) \right) \quad \text{Eq. 5.54}$$

After rearranging terms and introducing the constant defined by Equation 5.52, we have:

$$w'' = C_{\text{Mb}} \left(\frac{h_t C}{\omega^2} - 1 \right) M - \frac{C}{\omega^2} (N_1'' + k_{\text{core}} \varepsilon_t) \quad \text{Eq. 5.55}$$

Where the factor in front of M may be rewritten as:

$$\begin{aligned} C_{\text{Mb}} \left(\frac{h_t C}{\omega^2} - 1 \right) &= \frac{1}{E(I_1 + I_2)} \left(\frac{\frac{h_t^2}{(I_1 + I_2)}}{\frac{1}{A_1} + \frac{1}{A_2} + \frac{h_t^2}{I_1 + I_2}} - 1 \right) = \\ &= \frac{1}{E(I_1 + I_2)} \left(\frac{h_t^2 A_1 A_2}{(A_1 + A_2)(I_1 + I_2) + h_t^2 A_1 A_2} - 1 \right) = \\ &= \frac{-1}{E} \frac{A_1 + A_2}{(A_1 + A_2)(I_1 + I_2) + h_t^2 A_1 A_2} = \\ &= \frac{-1}{E} \frac{1}{I_1 + I_2 + \frac{h_t^2 A_1 A_2}{A_1 + A_2}} = \frac{-1}{EI_{\text{fca}}} \end{aligned} \quad \text{Eq. 5.56}$$

After a number of algebraic manipulations it is clear that the denominator at the final stage in Equation 5.56 is equal to the bending stiffness for the beam in case of full composite action, where it may be easily verified that:

$$I_{\text{fca}} = I_1 + I_2 + \frac{h_t^2 A_1 A_2}{A_1 + A_2} \quad \text{Eq. 5.57}$$

is the second area moment of inertia for the cross-section when full composite action prevails. Using the result from Equation 5.56 in Equation 5.55 gives:

$$w'' = \frac{-M}{EI_{\text{fca}}} - \frac{C}{k_{\text{core}} \omega^2} (N_1'' + k_{\text{core}} \varepsilon_t) \quad \text{Eq. 5.58}$$

Finally, by integrating Equation 5.58 twice, the link between the deflection function and the axial force function may be expressed as:

$$w = w_{\text{fca}} - \frac{C}{k_{\text{core}} \omega^2} \left(N_1 + k_{\text{core}} \varepsilon_t \frac{x^2}{2} \right) + D_3 x + D_4 \quad \text{Eq. 5.59}$$

where w_{fca} is the deflection in case of full composite action, that is no shear deformation of the core material occurs. Note, also that the term w_{fca} is computed with regard to the moment distribution only and that solutions to such elementary cases often exist in the literature. Coefficients D_3 and D_4 are integration constants that must be determined using appropriate boundary conditions. For many practical cases both coefficients may often be zero. It follows directly from Equation 5.59 that $D_4 = 0$ when $w = w_{\text{fca}} = N_1 = 0$ for the same x -coordinate and that $D_3 = 0$ when $w' = w'_{\text{fca}} = N'_1 = 0$ for some other x -coordinate.

To compute stresses within each layer it is necessary to know M_1 and M_2 . Expressions for these quantities can be obtained by combining Equations 5.44 and 5.45, which yields:

$$M_1 = \frac{I_1}{I_1 + I_2} (M + N_1 h_t) \quad \text{Eq. 5.60}$$

$$M_2 = \frac{I_2}{I_1 + I_2} (M + N_1 h_t) \quad \text{Eq. 5.61}$$

5.2.1.3 Solution procedure

In order to find a function $N_1(x)$ satisfying Equation 5.50 it is necessary to have an expression for the moment distribution $M(x)$ along the beam in question. Here it will be demonstrated how to find a solution for a beam subjected to transverse loading causing a bending moment distribution that can be expressed as a polynomial of order four. Of course higher orders are possible, but are quite rare in practical applications. Note that moment distributions expressed as polynomials are by far the most common type of loading assumed in classic beam bending theory. Besides from this, the beam may have quite general boundary conditions. The moment distribution along the beam may be written as:

$$M(x) = \sum_{i=0}^4 a_i x^i \quad \text{Eq. 5.62}$$

where the a_i 's are known coefficients for statically determinate beams and for statically indeterminate beams one or more of them must be expressed in terms of some indeterminate quantity, for example an unknown support reaction or bending moment at a built in support. Note that the beam may have to be divided into more than one interval along the x -axis, that is the function representing the moment distribution may in general be different for each interval depending on the actual loading and boundary conditions. Here just one interval having a continuous moment distribution is assumed. Substituting Equation 5.62 into 5.50 gives:

$$N_1'' - \omega^2 N_1 = C \left(a_0 - \frac{k_{\text{core}} \varepsilon_t}{C} + a_1 x + a_2 x^2 + a_3 x^3 + a_4 x^4 \right) \quad \text{Eq. 5.63}$$

This second order differential equation is linear and has constant coefficients and may be solved using standard mathematical procedures given in most schoolbooks on elementary differential equations. The total solution is the sum of the homogeneous solution and one particular solution:

$$N_1(x) = N_{1,h}(x) + N_{1,p}(x) \quad \text{Eq. 5.64}$$

The homogeneous solution is:

$$N_{1,h}(x) = D_1 \cosh(\omega x) + D_2 \sinh(\omega x) \quad \text{Eq. 5.65}$$

where the constants D_1 and D_2 must be determined with regard to the boundary conditions of the problem. The particular solution must be a fourth order polynomial because the right part of Equation 5.63 is such a polynomial. By substituting a polynomial trial solution and its second order derivative expressed as:

$$N_{1,p}(x) = \sum_{i=0}^4 b_i x^i \quad \text{and} \quad N_{1,p}''(x) = \sum_{i=2}^4 i(i-1)b_i x^{i-2} \quad \text{Eq. 5.66}$$

into Equation 5.63 and identifying terms a linear equations system in the unknown b_i 's are set up. A symbolic solution to that system expresses the unknown b_i 's of the particular solution in the known a_i 's, which together with Equation 5.65 are substituted back into Equation 5.64 yielding:

$$N_1(x) = D_1 \cosh(\omega x) + D_2 \sinh(\omega x) - \frac{C}{\omega^2} \left[\begin{array}{l} a_0 - \frac{k_{\text{core}} \varepsilon_t}{C} + x a_1 + \left(x^2 + \frac{2}{\omega^2} \right) a_2 + \\ \left(x^3 + \frac{6x}{\omega^2} \right) a_3 + \left(x^4 + \frac{12x^2}{\omega^2} + \frac{24}{\omega^4} \right) a_4 \end{array} \right] \quad \text{Eq. 5.67}$$

Furthermore, it is usually necessary to have an expression for the deflection function, which is obtained by substituting Equation 5.67 into Equation 5.59 resulting in:

$$w(x) = w_{\text{fca}}(x) + D_3 x + D_4 - \frac{C}{k_{\text{core}} \omega^2} \left(\begin{array}{l} D_1 \cosh(\omega x) + D_2 \sinh(\omega x) + k_{\text{core}} \varepsilon_t \left(\frac{1}{\omega^2} + \frac{x^2}{2} \right) \\ - \frac{C}{\omega^2} \left[\begin{array}{l} a_0 + x a_1 + \left(x^2 + \frac{2}{\omega^2} \right) a_2 + \\ \left(x^3 + \frac{6x}{\omega^2} \right) a_3 + \left(x^4 + \frac{12x^2}{\omega^2} + \frac{24}{\omega^4} \right) a_4 \end{array} \right] \end{array} \right) \quad \text{Eq. 5.68}$$

Note that the principal of superposition holds because linear elastic material is assumed, that is our differential equation is linear. Therefore, solutions to elementary cases may be superimposed in exactly the same way as in classic beam bending theory. If you have a library of solutions to some elementary cases they can be combined in numerous ways to find

solutions to new elementary cases, rather than explicitly solving Equations 5.50 and/or 5.51 possibly with the aid of Equations 5.67 and/or 5.68.

One consequence that immediately follows is that it would not have been necessary to carry all terms including ε_t through Equations 5.50 to 5.68. If in Equation 5.50 simply setting $M = 0$ as if no external bending moment is present, a new elementary case only dealing with ε_t is formed. Its solution can then be added to any other case as needed. But now the easiest way to find the solution to this case is to recognise that setting $M = 0$ is equivalent to setting all a_i 's to 0 in Equations 5.67 and 5.68, after which the coefficients D_1, D_2, D_3 and D_4 are determined with respect to actual boundary conditions.

It will now be demonstrated how to find the complete solution to one elementary case of practical interest. The case of a simply supported beam subjected to a concentrated load P acting at mid-span is chosen and it is also assumed that $\varepsilon_t = 0$. The total span is ℓ and the moment distribution has a discontinuous derivative at mid-span, at $\ell/2$. It is therefore necessary to divide the beam into two parts one for $0 \leq x \leq \ell/2$ and one for $\ell/2 < x \leq \ell$. But because of symmetry it suffices to find a solution for one of the intervals, preferably $0 \leq x \leq \ell/2$. The bending moment is expressed as $M(x) = P/2 \cdot x$. By comparing to Equation 5.62 we see that $a_1 = P/2$, while a_0, a_2, a_3 and $a_4 = 0$. The coefficients D_1, D_2, D_3 and D_4 of Equations 5.67 and 5.68 may now be determined using some adequate boundary conditions. The axial force in each layer as well as the deflection must be zero at $x = 0$, which may be expressed as $N_1(0) = 0$ and $w(0) = 0$. These two conditions give without any calculations that $D_1 = D_4 = 0$ in Equations 5.67 and 5.68. At mid-span the axial force N_1 and the deflection w have their maximum and their derivatives must therefore be zero, $N_1'(\ell/2) = 0$ and $w'(\ell/2) = 0$. From Equation 5.59 it follows that $D_3 = 0$. Finally, Equation 5.67 gives:

$$N_1' \left(\frac{\ell}{2} \right) = D_2 \omega \cosh \left(\frac{\omega \ell}{2} \right) - \frac{C}{\omega^2} \frac{P}{2} = 0 \Rightarrow D_2 = \frac{CP}{2\omega^3 \cosh \left(\frac{\omega \ell}{2} \right)} \quad \text{Eq. 5.69}$$

The total solution is obtained by substituting the results of Equation 5.69 back into Equations 5.67 and 5.68, and after some rearranging of terms and developing of the deflection function (wfca for full composite action) we arrive at:

$$N_1(x) = \frac{C}{\omega^3} \frac{P}{2} \left(\frac{\sinh(\omega x)}{\cosh \left(\frac{\omega \ell}{2} \right)} - \omega x \right)$$

$$w(x) = \frac{P\ell}{2} \left(\begin{array}{l} \frac{\ell x}{8EI_{fca}} \left(1 - \frac{4}{3} \left(\frac{x}{\ell} \right)^2 \right) - \\ \frac{C^2}{k_{core} \omega^5 \ell} \left(\frac{\sinh(\omega x)}{\cosh \left(\frac{\omega \ell}{2} \right)} - \omega x \right) \end{array} \right) \quad \text{if } 0 \leq x \leq \frac{\ell}{2} \quad \text{Eq. 5.70}$$

One can of course find solutions to other problems, being loaded differently and having other boundary conditions, by using the same principal approach. A summarising recipe may look like:

- a) Divide if necessary the entire beam into a number of intervals for which the moment distribution is continuous, that is has a continuous first order derivative or in other words can be written as one single function expression. Note also that all cross-sectional constants must not change along each chosen interval.
- b) For each interval identified under point (a), express the moment distribution as a polynomial and identify the a_i 's in Equations 5.62, 5.67 and 5.68. Note also that if the beam is statically indeterminate one or more of the a_i 's have to be expressed in terms of all the statically indeterminate quantities, such as a reaction force or bending moment.
- c) Figure out relevant boundary conditions such that Equations 5.67 and 5.68 plus their first and second order derivatives can be used to find expressions for the coefficients D_1 , D_2 , D_3 and D_4 . Remember that D_3 and D_4 are often zero due to the structure of Equation 5.59. One equation is needed for each D_i and for each statically indeterminate quantity. Boundary conditions securing continuity of Equations 5.67, 5.68 and their derivatives across interval borders are useful if the beam is subdivided into several intervals. Note also that all D_i 's are in general different on each side of an interval border.
- d) Set up all chosen boundary conditions as a system of linear algebraic equations and solve it for the unknown D_i 's and if present the statically indeterminate quantities. The solution can be either symbolic or numeric. A symbolic solution is more versatile and can be reused over and over again for different input data like dimensions and stiffness properties, while a numeric solution is only valid for exactly the same input data as those used for setting up the equation system. Some useful boundary conditions are: $N_1 = 0$ at a free end whether or not the beam is supported or free to deflect at this end; $N'_1 = 0$ at a position where no slip can occur; $w = 0$ at a pinned support; $w' = 0$ at a built in end.

The complete symbolic solutions to a large number of practically useful elementary cases are given in Elsander (1999). Here, the solution to just one more case is given, which is a simply supported beam subjected to uniform loading q . The axial force in part 1 is:

$$N_1(x) = \frac{Cq}{\omega^4} \left(\tanh\left(\frac{\omega\ell}{2}\right) \sinh(\omega x) - \cosh(\omega x) + \frac{\omega^2}{2}(x^2 - \ell x) + 1 \right) \quad \text{Eq. 5.71}$$

It is also possible to use analogies in order to solve the differential Equation 5.50, because other physical phenomena might be controlled by a differential equation of exactly the same mathematical form. For instance, the equation controlling mixed torsion (that is Saint Venant torsion in combination with warping torsion) has an equivalent mathematical form. Also, the equation controlling deflection of an elastic bar subjected to transverse loading in combination with axial tension is equivalent in shape.

As already mentioned, the principle of superposition holds because small displacements and linear elastic material are assumed. Therefore, solutions to elementary cases may be superimposed in exactly the same way as in classic beam bending theory.

By using Equation 5.70 some graphics presenting the basic difference in results between full composite action (FCA), partial composite action (PCA) and no composite action (NCA) have been derived and are presented in Figure 5.22. Parameters used are: $\ell = 2,40$ m, $P = 5,00$ kN, $C = 6,49 \times 10^{-9}$ mm⁻³, $\omega = 9,50 \times 10^{-4}$ mm⁻¹, $E = 11,0$ GPa and $I_{fca} = 3,90 \times 10^7$ mm⁴. These values are representative for a T-section made of two joists of C24-timber having dimensions 45×145 mm (turned edgeways) and 45×120 mm (turned flatways). These joists are nailed together using one row of $120 \times 3,8$ mm smooth grooved nails with a spacing of $s = 60$ mm. Instantaneous conditions have been assumed.

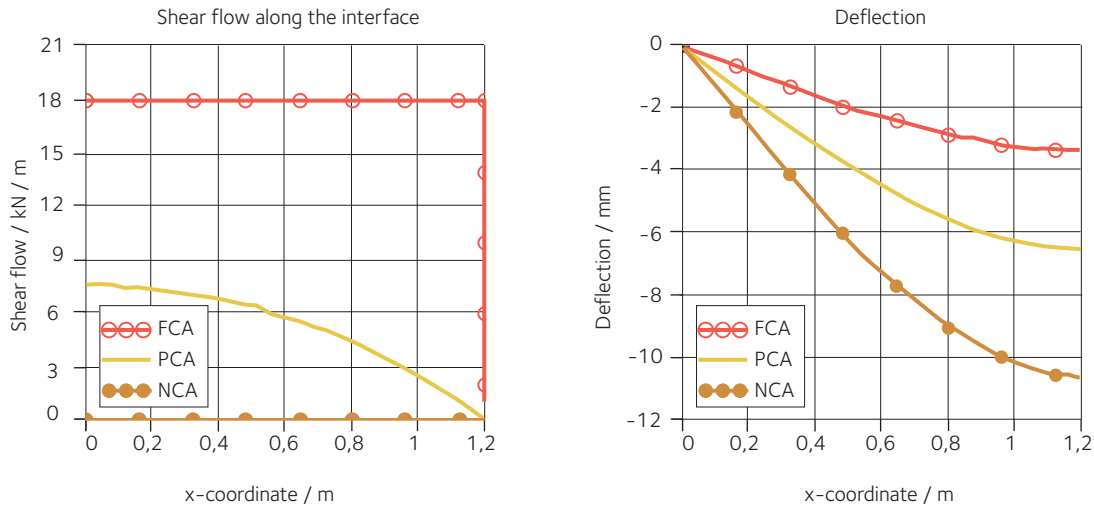


Figure 5.22: Partial composite action in comparison to the upper and lower limits.

5.2.1.4 Relations between the slip modulus of mechanical connectors and shear stiffness of the interlayer core

The governing equations have been derived assuming the presence of an interlayer core material characterised by a shear stiffness as given by Equation 5.42. But all derived equations are also valid for mechanically connected beams like nailed timber beams if the spacing between the discrete connectors is small enough. Here, small enough means that the stiffness of the connectors can be assumed to be smeared out along the beam axis as if the connection is continuous. For mechanically connected beams a different shear stiffness must be used, so that Equation 5.42 must be replaced by the shear stiffness of the mechanical connectors, which may be computed as:

$$k_{\text{core}} = \frac{k n}{s} \tag{Eq. 5.72}$$

where k is the slip modulus of a single connector (for instance K_{ser} or K_u for a timber structure designed according to Eurocode 5), s is the constant spacing between the connectors in the longitudinal direction and n is the number of connectors placed on a line perpendicular to the longitudinal direction. Suppose that two (or more) different types of connections connect the two layers, for example a shear flexible core of thickness h_{core} and mechanical connectors with slip modulus k and spacing s . One must then distinguish between parallel or series arrangement of the different connections. In the parallel case both types of connectors connect the two layers independently of each other, for example mechanical connectors that go right through a core material.

In the series case the shear flow is transferred from one of the layers into the first type of connection then into the next, etc., until the second layer is reached. An example is a shear flexible core that is connected to each layer by mechanical connectors that do not fully penetrate the core material, but have the same properties at each side of the core. For the discussed examples the stiffness in the parallel case may be written as:

$$k_{\text{core}} = \frac{k n}{s} + \frac{G b}{h_{\text{core}}} \quad \text{Eq. 5.73}$$

and for the series case it may be expressed as:

$$k_{\text{core}} = \frac{1}{2 \frac{s}{k n} + \frac{h_{\text{core}}}{G b}} \quad \text{Eq. 5.74}$$

Equations 5.73 and 5.74 are merely examples and must be properly adjusted in each particular case. For more complicated interlayer connections, as in latticed girders and girders with packs or gussets (Vierendeel type), expressions for k_{core} are for instance given in Larsen and Riberholt (2005).

In connection to Equations 5.40 to 5.43 is explained that the shear flow along the interface is the first derivative of the axial force in layer 1, i.e. $H = -N'_1$. The shear force acting on an individual connector is therefore:

$$F(x) = -N'_1(x) \frac{s}{n} \quad \text{or} \quad F(x) = k \cdot u(x) \quad \text{Eq. 5.75}$$

and from Equation 5.41 the slip along the interface is expressed as:

$$u(x) = \frac{-N'_1(x)}{k_{\text{core}}} \quad \text{or} \quad u(x) = -N'_1(x) \frac{s}{k n} = \frac{F(x)}{k} \quad \text{Eq. 5.76}$$

5.2.1.5 Approximate solutions of the governing differential equations

Equations 5.50 or 5.51 may be solved approximately. In many practical applications the approximate solution is good enough, and indeed it forms the basis for a very practical method of finding the flexural buckling resistance of built-up columns, see Eurocode 5 or Larsen and Riberholt (2005).

In short the approximate solution is based on the exact solution to an elementary case in which a simply supported beam is loaded by a sinusoidally distributed load, that is half a sinewave. The load and bending moment is expressed as:

$$q(x) = q_0 \sin\left(\frac{\pi x}{\ell}\right); \quad M'' = -q \quad \text{gives} \quad M(x) = q_0 \left(\frac{\ell}{\pi}\right)^2 \sin\left(\frac{\pi x}{\ell}\right) \quad \text{Eq. 5.77}$$

where q_0 is the maximum amplitude of the distributed load. By substituting the bending moment into Equation 5.50 and setting $\varepsilon_t = 0$ we obtain:

$$N_1'' - \omega^2 N_1 = C q_0 \left(\frac{\ell}{\pi}\right)^2 \sin\left(\frac{\pi x}{\ell}\right) \quad \text{Eq. 5.78}$$

It can now be easily verified that D_1 and D_2 in the homogeneous solution must be zero, meaning that the particular solution is the entire solution. By attempting a sinusoidally shaped particular solution the total solution becomes:

$$N_1(x) = \frac{-q_0 C}{\left(\frac{\pi}{\ell}\right)^2 \left(\left(\frac{\pi}{\ell}\right)^2 + \omega^2\right)} \sin\left(\frac{\pi x}{\ell}\right) \quad \text{Eq. 5.79}$$

By substituting Equation 5.79 into Equation 5.59 noting that $D_3 = D_4 = \varepsilon_t = 0$ and topping off with some algebraic manipulations the deflection function is found to be:

$$w(x) = \frac{q_0 \ell^4}{\pi^4 E I_{fca}} \frac{1 + \frac{1}{C_M} \left(\frac{\pi}{\ell}\right)^2}{1 + \left(\frac{\pi}{\omega \ell}\right)^2} \sin\left(\frac{\pi x}{\ell}\right) = \frac{q_0 \ell^4}{\pi^4 E I_{fca}} \gamma_{dmf} \sin\left(\frac{\pi x}{\ell}\right) \quad \text{Eq. 5.80}$$

During the algebraic rearranging the second area moment in case of full composite action I_{fca} and the constant C_M from Equation 5.53 have been introduced. From Equation 5.80 it is clear that γ_{dmf} represents a displacement magnification factor, which multiplied by the fca-displacement gives the “correct” displacement accounting for interlayer slip. Another way of understanding Equation 5.80 is to see:

$$E I_{ef} = E I_{fca} \frac{1}{\gamma_{dmf}} \quad \text{Eq. 5.81}$$

as an effective bending stiffness of the cross-section, which is constant along the entire beam. Note that this is the only elementary case for which the bending stiffness can be viewed as constant along the beam. For all other cases the hyperbolic terms from the homogeneous solution makes such a simple interpretation impossible. But we can expect that Equations 5.79 to 5.81 give a very good approximate solution in cases where a beam is supported and loaded such that the external bending moment distribution is similar to half a sine-wave. The method for designing built-up beams and columns, characterised by partial composite action, in Eurocode 5 depends entirely on this simplified solution. In Eurocode 5 the effective bending stiffness in Equation 5.81 is expressed differently, but is really the same thing.

5.2.1.6 Three layered double symmetric beams

The characteristics of a three layered beam are shown in Figure 5.23. Note that even the properties and spacing of the fasteners connecting parts 1 and 2 must be the same as for those connecting parts 2 and 3. Also, the strain difference ε_t between the layers are left out, because it is rare to have a symmetric influence from this kind of effect.

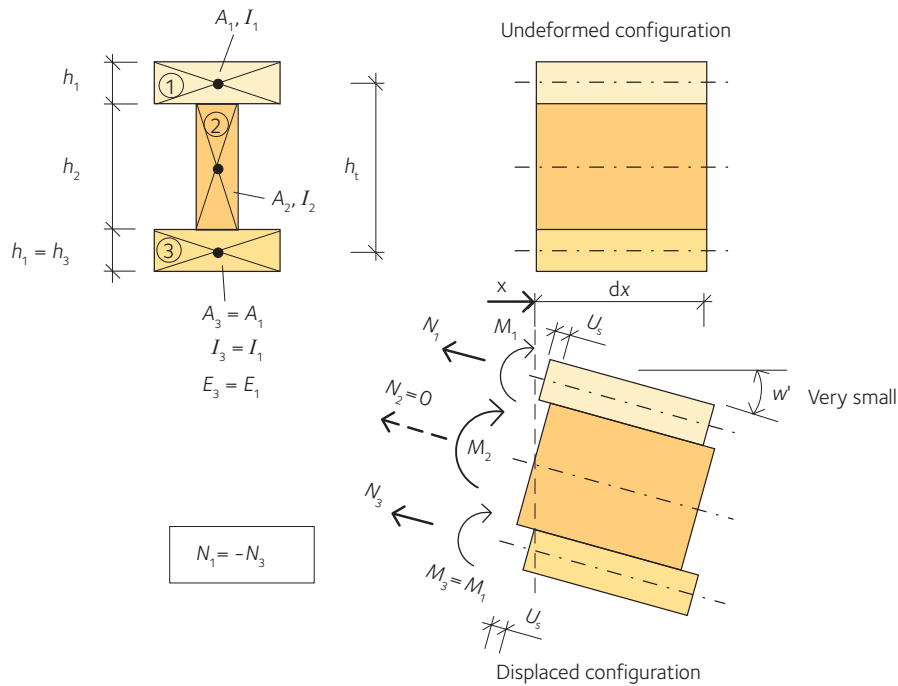


Figure 5.23: A piece of a three layered double symmetric beam.

It is for a double symmetric three layered beam actually possible to use most of the previously defined equations. By going through the derivation phase as explained by Equations 5.32 to 5.51, it can be shown that the differential Equations 5.50 and 5.51 does not change. It is just some of the cross-sectional constants together with a few of the equations that need to be modified. The cross-sectional constants become:

$$\begin{aligned}
 C &= \frac{k_{\text{core}}}{2E} \frac{h_t}{(2I_1 + I_2)} && \text{valid for DS3L} \\
 \omega^2 &= \frac{k_{\text{core}}}{E} \left(\frac{1}{A_1} + \frac{h_t^2}{2(2I_1 + I_2)} \right) && \text{valid for DS3L} \\
 C_{\text{Mb}} &= \frac{1}{E(2I_1 + I_2)} && \text{valid for DS3L} \\
 C_{\text{M}} &= \frac{k_{\text{core}}}{EA_1} && \text{valid for DS3L}
 \end{aligned}
 \tag{Eq. 5.82}$$

where each equation is labelled with DS3L, which means “double symmetric 3-layered”. This labelling is just used to avoid mixing up with the two layered case. The equations that need adjustment are the bending moment in each part (Equations 5.60 and 5.61), which change to:

$$\begin{aligned}
 M_1 = M_3 &= \frac{I_1}{2I_1 + I_2} (M + N_1 h_t) && \text{valid for DS3L} \\
 M_2 &= \frac{I_2}{2I_1 + I_2} (M + N_1 h_t) && \text{valid for DS3L}
 \end{aligned}
 \tag{Eq. 5.83}$$

and the deflection given by Equation 5.59 changes into:

$$w = w_{\text{fca}} - \frac{2C}{k_{\text{core}} \omega^2} N_1 + D_3 x + D_4 \quad \text{valid for DS3L} \quad \text{Eq. 5.84}$$

The more general single symmetric three layered case is more complicated. A detailed derivation will result in a system of differential equations, where each equation is similar to the one given in Equation 5.50. In general one equation is needed for each interface between the parts, that is two equations for a 3 layered beam, three equations for a 4 layered beam, etc. Equations for the 3 layered case can for instance be found in Oduyemi and Wright (1991).

5.2.2 Summary of equations

A summary is made in Table 5.3 of the most relevant equations to be used for 2 and 3 layered beams, including formulas for connector force and interface slip. Note that the 3 layered beam must be double symmetric. Note also that the ε_t -term has been left out.

Table 5.3 Summary of equations for 2 layered and 3 layered double symmetric beams. No ε_t -term is present.

Explanations	Equations common for 2 and 3 layered beams	
Differential equation in N_1	$N_1'' - \omega^2 N_1 = C M$	
Differential equation in w	$w'''' - \omega^2 w'' = C_{\text{Mb}} (C_{\text{M}} M - M'')$	
Connector force	$F(x) = -N_1'(x) \frac{s}{n}$ or $F(x) = k \cdot u(x)$	
Interface slip	$u(x) = \frac{-N_1'(x)}{k_{\text{core}}}$ or $u(x) = -N_1'(x) \frac{s}{kn} = \frac{F(x)}{k}$	
	Two layers	Three layers, double symmetric
Relation between N_1 and w	$w = w_{\text{fca}} - \frac{C}{k_{\text{core}} \omega^2} N_1 + D_3 x + D_4$	$w = w_{\text{fca}} - \frac{2C}{k_{\text{core}} \omega^2} N_1 + D_3 x + D_4$
Constants	$C = \frac{k_{\text{core}}}{E} \frac{h_t}{(I_1 + I_2)}$ $\omega^2 = \frac{k_{\text{core}}}{E} \left(\frac{1}{A_1} + \frac{1}{A_2} + \frac{h_t^2}{I_1 + I_2} \right)$ $C_{\text{Mb}} = \frac{1}{E(I_1 + I_2)}$ $C_{\text{M}} = \frac{k_{\text{core}}}{E} \left(\frac{1}{A_1} + \frac{1}{A_2} \right)$	$C = \frac{k_{\text{core}}}{2E} \frac{h_t}{(2I_1 + I_2)}$ $\omega^2 = \frac{k_{\text{core}}}{E} \left(\frac{1}{A_1} + \frac{h_t^2}{2(2I_1 + I_2)} \right)$ $C_{\text{Mb}} = \frac{1}{E(2I_1 + I_2)}$ $C_{\text{M}} = \frac{k_{\text{core}}}{E A_1}$
Bending moment in parts 1 and 2	$M_1 = \frac{I_1}{I_1 + I_2} (M + N_1 h_t)$ $M_2 = \frac{I_2}{I_1 + I_2} (M + N_1 h_t)$	$M_1 = M_3 = \frac{I_1}{2I_1 + I_2} (M + N_1 h_t)$ $M_2 = \frac{I_2}{2I_1 + I_2} (M + N_1 h_t)$

Equations 5.72 to 5.74 give possible relations between k_{core} and k for mechanically connected parts or layers. Equations 5.70, 5.71 and Equations 5.79 to 5.81 give solutions to 3 different elementary cases of practical interest.

5.2.3 Failure criteria for built-up beams

5.2.3.1 Recommendations regarding the stiffness properties to be used

For ordinary beams designed under the assumption of full composite action we need only to account for the difference in elastic modulus and creep properties of the parts making up the cross-section, as explained in Section 5.1.1.1. It is recommended to perform the analysis for partial composite action in the same way, that is using a transformed cross-section such that all parts can be assumed to have the same modulus of elasticity. It may also be necessary to make relevant checks at “instantaneous” and “final” conditions in order to cope with the creep effects.

There is one additional complication for partial composite beams due to the creep properties of the shear interface. Mechanical connections in timber materials are definitely sensitive to creep. Usually, the connection is more prone to creep than the timber itself, mainly due to relatively high stress levels in the small timber volumes surrounding the connector. In Eurocode 5 it is recommended to use a creep factor k_{def} for the slip modulus of the connector that is twice as large as for the timber itself, that is to use $2 k_{\text{def}}$. Creep usually plays a substantial role even if a thick shear interface layer (core) is used without any mechanical connectors, because such materials are often sensitive to creep.

Here, we will concentrate on mechanical connectors in built-up timber beams. Their slip modulus k is in Eurocode 5 characterised by either K_{ser} or $K_{\text{u}} = 2/3 \times K_{\text{ser}}$. K_{ser} is the gradient of a straight line going through a point on the load-slip relationship corresponding to approximately 40 percent of the connector’s ultimate resistance, while K_{u} corresponds to the 70 percent level.

How will the internal forces and stresses change in the composite beam if the slip modulus increases or decreases? The answers are that a decreasing slip modulus will give:

- A smaller axial force N_1 but the bending moments M_1 and M_2 will increase such that the net outcome is greater stress at the outermost edge of each layer.
- A smaller connector force F , i.e. the shear flow $H = -N'_1$ is decreasing and more of the load is carried by individual bending of the layers. We will have no composite action at all for a slip modulus equal to zero.
- A greater deflection.

An increasing slip modulus will give the opposite effects to those listed above. Luckily a large change in the slip modulus will produce a much smaller change in member forces, member stresses and deflection. Reducing k to half its value or doubling it will change these values in the order of 10 %, under typical conditions. But note that the connector force will typically change by the same order as k .

From the above description it is recommended that K_{u} is used when checking resistances in the ultimate limit state (ULS). This holds even if the weakest part of the beam is the connectors, because then their slip modulus will be even smaller than K_{u} for the most loaded (and failing)

connectors, hence subjected to a smaller load than actually calculated. Therefore, K_u is still a good choice, also noting that many connectors along the beam are not fully used. Of course, if one really wants to be on the safe side, K_{ser} might be used when the connectors are the weakest part of the beam. If the connectors are much stronger than the timber, then a value closer to K_{ser} may be beneficial when checking the timber resistance.

In the ULS it may still be necessary to distinguish between “instantaneous” and “final” conditions due to the creep properties of the timber itself. Here the key recommendation is to check the members most prone to creep at instantaneous conditions, while members less sensitive to creep are checked at final conditions.

In the serviceability limit state (SLS) it is recommended that K_{ser} is used when determining all the cross-sectional constants.

Following the recommendations given in Eurocode 5 as close as possible, the slip modulus and the modulus of elasticity may for the instantaneous and final conditions be taken from Table 5.4.

Table 5.4 Recommended values for the slip and elastic modulus of composite timber beams for which mechanical connectors are used and partial composite action prevails.

	Instantaneous conditions, inst	Final conditions, fin
Serviceability limit state, SLS	$E_{SLS,inst} = E_{0,mean}$ $k_{SLS,inst} = K_{ser}$	$E_{SLS,fin} = \frac{E_{0,mean}}{1 + k_{def}} *$ $k_{SLS,fin} = \frac{K_{ser}}{1 + 2k_{def}} *$
Ultimate limit state, ULS	$E_{ULS,inst} = E_{0,mean}$ $k_{ULS,inst} = K_u$	$E_{ULS,fin} = \frac{E_{0,mean}}{1 + \psi_2 k_{def}}$ $k_{ULS,fin} = \frac{K_u}{1 + 2\psi_2 k_{def}}$

* A note on determination of final deformations. Cross-sectional constants, that is I_{ca} , C , ω , C_{Mb} and C_M of transformed cross-sections, should be based on $E_{SLS,fin}$ and $k_{SLS,fin}$ for the instantaneous condition before the effect of creep is added. But all moduli beyond these fictitious cross-sectional constants should keep their instantaneous values $E_{SLS,inst} = E_{0,mean}$ and $k_{SLS,inst} = K_{ser}$, because they will at a later stage be multiplied by either $(1 + k_{def})$ or $(1 + 2k_{def})$ when going from instantaneous to final deformation as given by Equation 5.86.

Note, that the relations of Table 5.4 should be used to calculate all cross-sectional constants of a transformed cross-section when determining the distribution of internal forces between members of different materials having different creep properties. After that the final SLS-deformation is estimated using Equation 5.86, the method outlined here will slightly over-estimate the final creep deformation in that all the cross-sectional constants are determined using the creep factor for permanent loading. In the ULS ψ_2 shall be determined for the load causing the greatest stress, which effectively is a way of calculating a kind of weighted mean value of the cross-sectional constants that will give reasonable estimations of internal forces and stresses.

Equations 5.59 and 5.84 are made up of two terms: the first is called w_{fca} as it is the deflection in case of full composite action, while the second is called w_{slip} as it is the additional deflection caused by the connector slip. In w_{fca} the modulus of elasticity E is controlling the deflection, while the slip modulus k (k_{core}) is controlling the deflection in w_{slip} . Of course these moduli are also hidden within I_{fca} , C and ω , but are then purely relational and determine the distribution of internal forces between the members. Suppose now that we have permanent loading denoted G and variable loading denoted Q_1 for the leading variable load and Q_i for the other variable loads, i is 2, 3, etc. Following the recommendations in Eurocode 5 to use the characteristic combination in Eurocode 0 for determining SLS-displacements, the instantaneous deflection may formally be expressed as:

$$w_{inst} = \begin{cases} w_{fca,SLS,inst}(G) + w_{slip,SLS,inst}(G) + \\ w_{fca,SLS,inst}(Q_1) + w_{slip,SLS,inst}(Q_1) + \\ \sum_i \left(w_{fca,SLS,inst}(\psi_{0,i} Q_i) + w_{slip,SLS,inst}(\psi_{0,i} Q_i) \right) \end{cases} \quad \text{Eq. 5.85}$$

and the final deflection as:

$$w_{fin} = \begin{cases} w_{fca,SLS,fin}(G)(1 + k_{def}) + w_{slip,SLS,fin}(G)(1 + 2k_{def}) + \\ w_{fca,SLS,fin}(Q_1)(1 + \psi_{2,1} k_{def}) + w_{slip,SLS,fin}(Q_1)(1 + 2\psi_{2,1} k_{def}) + \\ \sum_i \left(w_{fca,SLS,fin}(Q_i)(\psi_{0,i} + \psi_{2,i} k_{def}) + w_{slip,SLS,fin}(Q_i)(\psi_{0,i} + 2\psi_{2,i} k_{def}) \right) \end{cases} \quad \text{Eq. 5.86}$$

where ψ_2 is a reduction factor giving the quasi permanent load value, while ψ_0 gives the combination value of the load.

In Equations 5.85 and 5.86, make sure to use $E_{0,mean}$ in all w_{fca} and K_{ser} in all w_{slip} . Note also that shear deflection caused by shearing of each timber member is neglected, which is justified by the fact that its contribution usually is small compared to w_{slip} .

5.2.3.2 Failure criteria in the ULS

Any failure criterion must be adjusted to meet the requirements of each particular material used as members and connectors. Stresses in each member must in general be checked, remembering that shear lag must be considered in very wide flanges and in wide compression flanges local buckling as well. The effective width can be determined using the approach given in Section 5.1.2.1. The integrity of the shear interface/interfaces must also be checked, that is shear stresses in a continuous core material or connector forces if discrete mechanical fasteners are used. Here, the failure criteria of a beam made of timber members joined by mechanical connectors will be discussed.

If the flanges are made of structural timber, glulam, LVL or alike, then the interaction formulas in Section 3.1.3 (or if the bending part in a member is small the criteria in Equation 5.6 or 5.25) should be checked for each part. If lateral torsional instability is also judged to be a problem then one may proceed as outlined in Section 5.1.1.2, but making proper adjustments due to the influence of partial composite action.

If the connectors are made of nails, staples, screws, dowels or bolts, then the failure modes according to Chapter 4 must be checked. Note, however, that bolts (and maybe also dowels) are quite unsuitable to use as connectors in this kind of composite beam, because the large initial slip caused by the clearance between bolt and hole will give large deflections before any shear force can be transmitted.

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Chapter 6

Horizontal stabilization

Roberto Crocetti

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6 Horizontal stabilization

6.1 General considerations

In design of structural systems, the way lateral stability is achieved is of fundamental importance. For the sake of simplicity let us approach the problem by studying a planar structure. All structures undergo some changes in shape under load. In a stable structure, the deformations induced by the load are typically small. Moreover, in a stable structure, internal forces are generated by the action of the load; these forces tend to restore the structure to its original shape after the load has been removed. In an unstable structure, the deformations induced by a load are typically large and tend to continue to increase as long as the load is applied. By contrast, an unstable structure does not generate internal forces that tend to restore the structure to its original configuration. It is the fundamental responsibility of the structural engineer to assure that a proposed structure does indeed form a stable configuration. Stability is a crucial issue in the design of structures that are assemblies of discrete elements. For example, the post-and-beam structure illustrated in Figure 6.1 a) is apparently stable. However, any horizontal force tends to cause deformations of the type indicated in Figure 6.1 b).

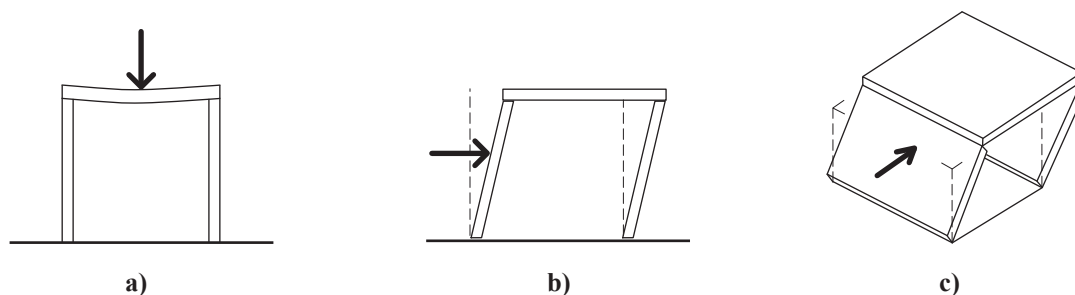


Figure 6.1: a) Column-and-beam assembly, b) instability under horizontal loads, c) instability in a wall-and-plate assembly.

Clearly, the structure has no capacity to resist horizontal loads, nor has it any mechanism that tends to restore it to its initial shape after horizontal load is removed. The large changes in angle that occur between members characterise an unstable structure that is beginning to collapse. Consequently, this particular pattern of members is referred to as a collapse mechanism.

There are just a few fundamental ways of converting a self-standing structure of the general type shown in Figure 6.1 from an unstable to a stable configuration. These methods are illustrated in Figure 6.2.

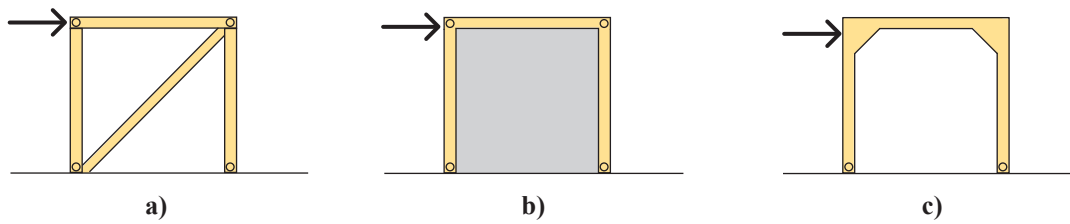


Figure 6.2: a) Diagonal bracing, b) shear wall, c) rigid joints.

The first is to add a diagonal member to the structure, see Figure 6.2 a). In such a way, the structure cannot undergo the “parallelogramming” indicated in Figure 6.1 a) 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, that is rigid planar surface elements that resist changes in shape of the type illustrated, see Figure 6.2 b). A third method used to achieve stability is by stopping the angular changes that occur between members and that are associated with collapse see Figure 6.2 c). In this case the connections between members are such that the angle of intersection between members remains constant under any loading. Such connections are often referred to as rigid connections (or rigid joints).

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 in a structure, for example a structure having 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 redundancy is obviously increased. As an example, consider the following glulam frame, with clamped connection at the base of the columns and pinned connections between beam and columns, see Figure 6.3. The system is stable without any diagonal bracing. However, the introduction of a steel diagonal dramatically increases the horizontal stiffness of the frame, even when relatively small diameters of the steel bar are used.

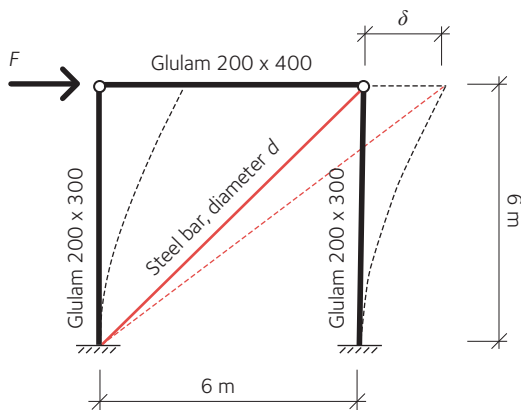


Table 6.1

Diameter	Horizontal stiffness $k = F/\delta$
No diagonal	$k = k_0$
$d = 10 \text{ mm}$	$k \approx 7 \cdot k_0$
$d = 20 \text{ mm}$	$k \approx 25 \cdot k_0$
$d = 30 \text{ mm}$	$k \approx 50 \cdot k_0$

Figure 6.3: Influence of the diagonal bracing on the stiffness of the frame. The column are bent around their strong axes.

In the case of a real building (which by nature is of spatial character) the way it resists lateral forces influences not only the design of vertical elements, but the horizontal spanning system as well.

The basic issues associated with the effect of lateral forces due to wind or earthquakes on a spatial structure are illustrated in Figure 6.4. Lateral forces not only cause structures to deform horizontally. They can also cause twisting or torsional deformations. If no adequate resistance to such mechanisms – for example shear walls, diagonal bracings or rigid frames – are present to resist lateral forces, unacceptable displacements and possibly complete collapse can occur.

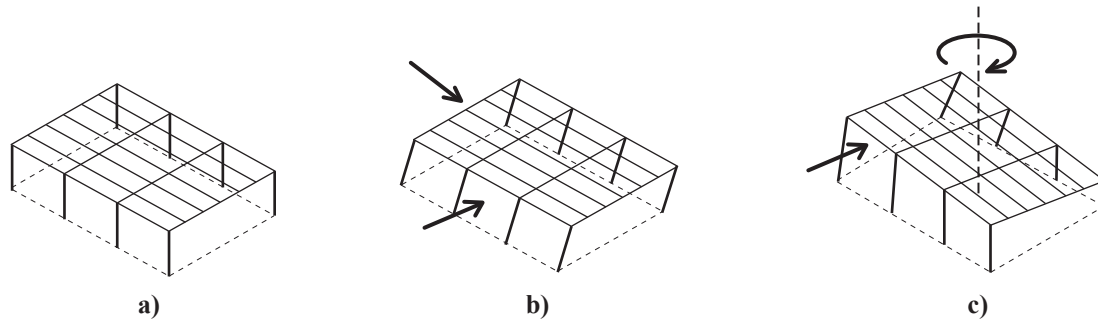


Figure 6.4: a) Basic structure without lateral stability devices, b) major lateral displacements (“racking”) due to horizontally-acting wind or earthquake loadings, c) torsional displacements.

Wind forces on the side of simple buildings of the type shown in Figure 6.5, for example, would be picked up by surface members, which in turn transfer them to secondary frame elements. Earthquake forces have a slightly different character than wind forces, as they are normally concentrated at high-mass areas, such as roofs or floors, but for the purpose of this discussion, we assume that the effects of wind or earthquakes can be represented by a series of forces acting laterally on the primary structure.

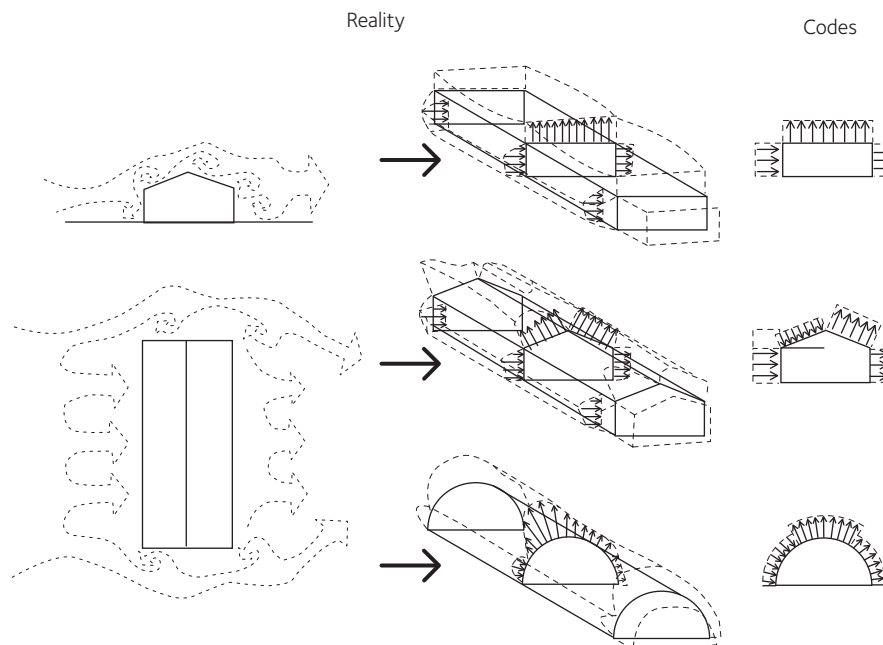


Figure 6.5: Wind pressure and suction, reality versus building codes. In some building codes the model for wind pressure and suction may be more sophisticated with slightly different coefficients.

It should be emphasised that a primary role played by a roof or a roof plane is, besides resisting vertical loads, to transfer lateral forces to side shear walls, cross braces, or frames. Figure 6.6 illustrates two methods for picking up forces acting on transverse wall and transferring them to side shear planes. The figure also illustrates how forces on the longitudinal face are transferred to side reverse shear planes.

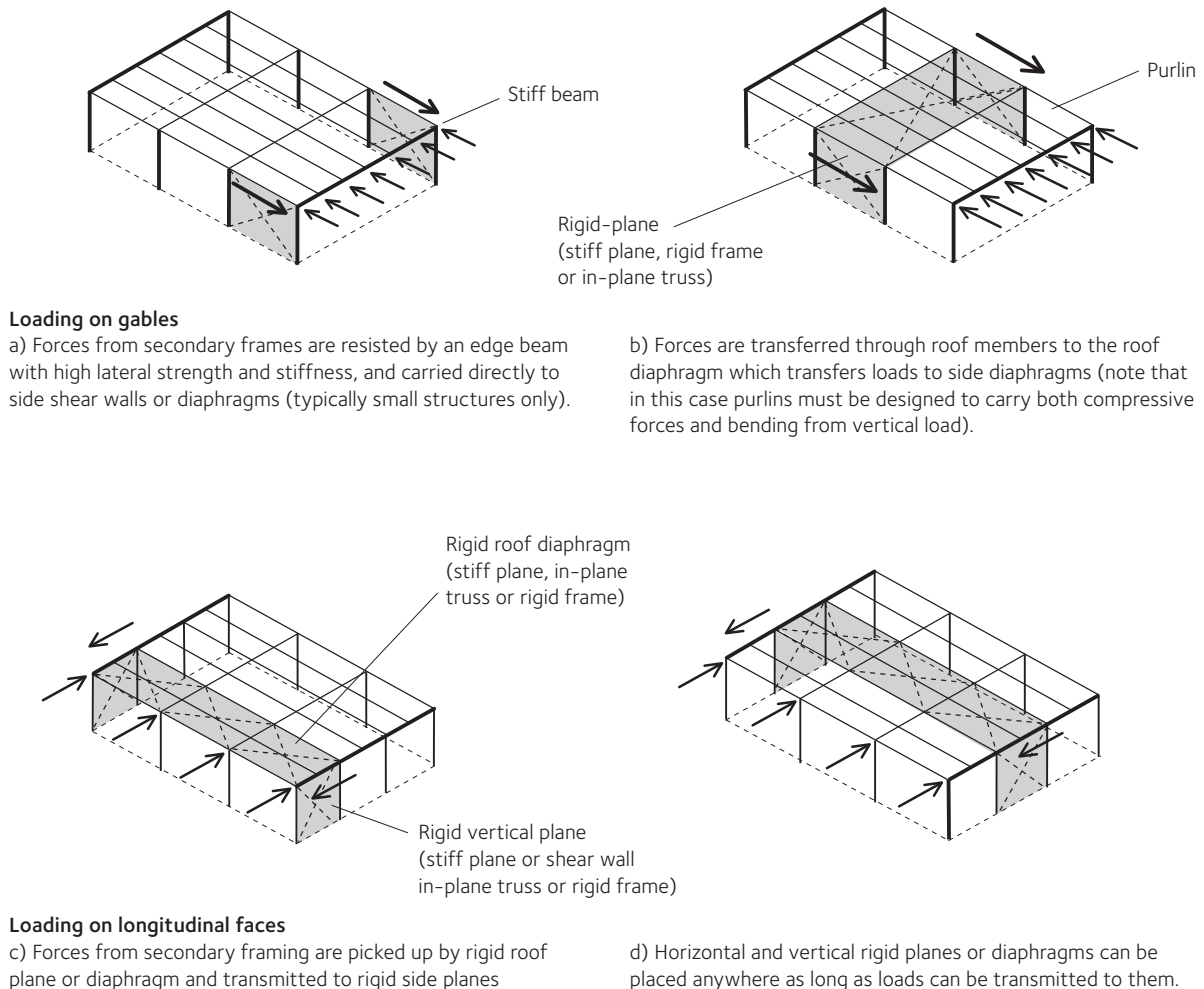


Figure 6.6: Force transfer to rigid wall or roof-planes. The graphic convention of a grey tone with dotted “X” is used herein to depict a rigid plane, regardless of whether it is a wall, truss, or rigid frame.

The use of an edge beam approach illustrated in Figure 6.6 a) demands that the edge beam be sized for both vertical and horizontal loads. This approach is often used for relatively small buildings. As building dimensions increase, a larger part of the roof plane is typically utilised for stiffness. An entire bay, for example, might be cross braced as a way of providing sufficient in-plane stiffness for a large building Figure 6.6 b). In these cases, it is essential that the whole system be organised such that the rigid horizontal plane (at the roof) both receive externally induced horizontal forces and transmit them to side shear planes (at the walls). Unless there are two separate systems for transmitting horizontal forces, for example purlins and struts, the floor or roof beams (purlins) must be designed to carry both bending due to vertical loads and compression due to horizontal loads.

6.2 The importance of bracing

Bracing effectively resists the racking of the roof, especially under lateral loads from a direction other than parallel or perpendicular to the building. In other words, an effective bracing provides the roof with beneficial torsional stiffness in the roof plane. Bracing is also often needed during the construction stage. Whilst for minor timber structures sufficient bracing can be achieved by means of wood panel sheathing, for heavy timber structures bracing consists, generally, of either:

- steel rods capable of acting in tension only
- timber struts, which can take both tension and compression
- profiled steel shear diaphragms.

When steel rods are used, they are usually arranged in crossing pairs, so that if racking puts one of the elements into compression (which it is incapable of resisting), its complement will resist racking by tension.

Bracing may also be necessary to prevent lateral buckling of the entire roof plane and 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, see Figure 6.7.

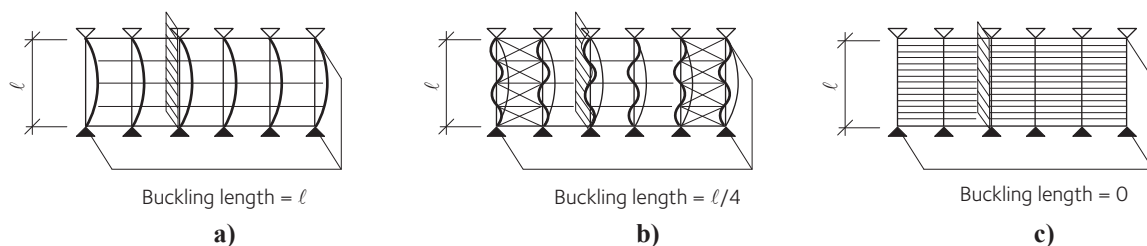


Figure 6.7: Roof structures with main beams braced in different ways:

- a) Unbraced. Buckling length of the main beams $\ell_{ef} = \ell$.**
b) Braced with crossing pairs of steel rods. Buckling length of the main beams $\ell_{ef} = \ell/4$.
c) Braced with profiled metal sheathing. Buckling length of the main beams $\ell_{ef} = 0$.

The specific arrangement of the bracing in the roof plane depends on different aspects, such as:

- the position of the lateral force resisting elements (wall bracing systems)
- the required stiffness of the roof structure
- the magnitude of the force flow.

The wall bracing system which resists lateral forces may be arranged in a symmetrical or asymmetrical manner. For the symmetrical case they will only carry direct forces as caused by translation. For the asymmetrical case, additional forces due to rotation (torsion) are generated, see Figure 6.10 b).

6.3 Fundamental statics

As a rule, for closely spaced, parallel, supporting, vertical structures (that is the wall bracing systems), the roof or floor diaphragm can be considered as rigid, see Figure 6.8. Therefore, lateral load distribution is known, being just a function of the tributary façade area. However, in the case of a statically indeterminate system, the load distribution is not obvious if rigid frames or shear walls are placed rather far apart. In such a case, in fact, the roof or floor diaphragm becomes more flexible, see Figure 6.9, and therefore, lateral load distribution depends on the ratio between the rigidity of the roof (or floor) diaphragm and the rigidity of the supporting structure. Though there is no such thing as infinitely rigid or flexible diaphragm, the concepts are still useful for developing some understanding about lateral force flow. Normally, concrete slabs and some types of steel deck are considered be rigid diaphragms which are assumed to distribute the horizontal forces to the supporting vertical structures in proportion to their relative rigidity R .

For a building with rigid roof diaphragm and four equally stiff shear walls (or equivalent frames) each wall carries the same force, see Figure 6.8 a). In the case b), the interior walls only provide one-half the rigidity of the exterior walls. In this case, the total horizontal action H must be redistributed in proportion to the wall rigidities. The total rigidity is:

$$\sum R = 2 \cdot \left(R + \frac{R}{2}\right) = 3 \cdot R \quad \text{Eq. 6.1}$$

Hence, each of the external walls carries:

$$V_e = H \cdot \frac{R}{3 \cdot R} = \frac{H}{3} \quad \text{Eq. 6.2}$$

while each of the interior walls resists:

$$V_i = H \cdot \frac{0,5 \cdot R}{3 \cdot R} = \frac{H}{6} \quad \text{Eq. 6.3}$$

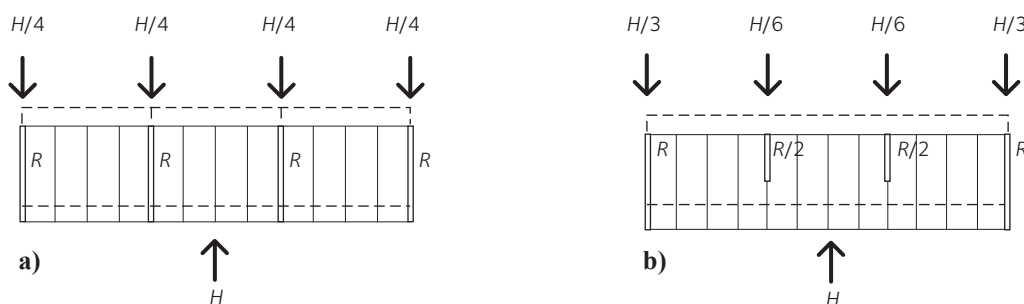


Figure 6.8: Rigid roof diaphragm. a) Four walls with equal rigidity R , b) walls with different rigidities.

Roof with wood based sheathing, or with some type of steel deck, are generally considered to be flexible diaphragms. They are normally so much more flexible in comparison to the lateral supporting system that the supporting structures are considered to be rigid. The behaviour of the flexible diaphragm may be visualised as simply supported, discontinuous beams sitting on unyielding supports. Thus, the loads can approximately be distributed as based on tributary

area. For the case in Figure 6.9 a), note that the smaller interior walls carry double the force as the longer exterior walls, because the tributary areas are twice as large as the ones for the exterior walls; the stiffness of the walls is not considered since they are assumed infinitely rigid.

The behaviour of semi-flexible or semi-rigid diaphragms, on the other hand, is highly indeterminate. They are assumed to distribute the loads like a continuous beam on flexible supports.

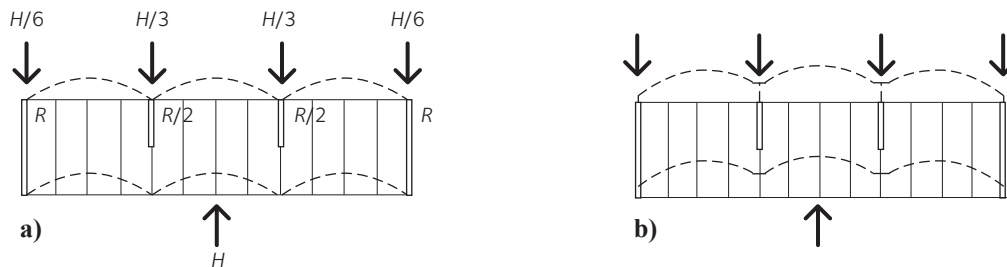


Figure 6.9: a) Flexible roof diaphragm, b) semi-rigid roof diaphragm.

If the lateral force-resisting structures are arranged in an asymmetrical fashion, then in addition to translation (that is direct shear), rotation (that is torsion) is generated. The magnitude of the generated rotation is also influenced by the building proportions, see Figure 6.10.

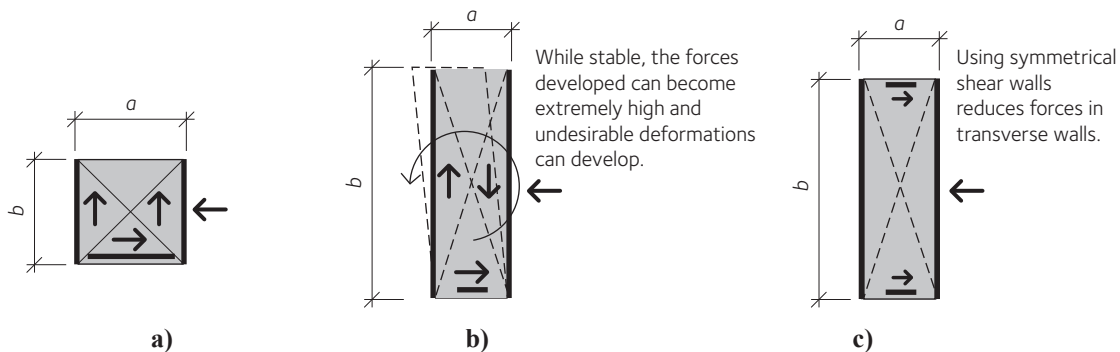


Figure 6.10: Effect of building proportions on asymmetrically braced structure. a) Square $b = a$, b) rectangle $b = 4 \cdot a$, problematic arrangement, c) rectangle $b = 4 \cdot a$, preferred arrangement.

The solution for the distribution of the lateral forces to the asymmetrically arranged shear walls can be approached by distinguishing between determinate and indeterminate systems as based on the following simplified assumptions:

- Consider the roof as a rigid diaphragm. Flexible diaphragms are considered incapable of transmitting rotational forces.
- Neglect the bending stiffness of the walls about their weak axis.
- Consider intersecting walls as separate units. There is no shear transfer at the corners.
- Consider the shear flow along the wall to be constant.
- Neglect the effect of columns in resisting lateral forces.

Hence, according to these assumptions, as based on statics, a minimum of three shear walls or other types of lateral force-resisting planar systems is necessary to stabilize a building against force action from any direction; the arrangement of the walls shall be neither parallel nor concurrent according to the laws of statics. When more than three shear walls are present, the structure is statically indeterminate and the force distributions can be found by means of either approximate methods or computer-based approaches. As an example, let us study the force distributions in the building shown in Figure 6.11.

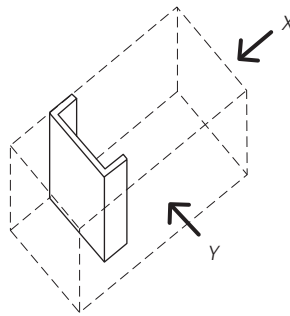


Figure 6.11: Illustration of a building stabilised by a channel-shaped bracing system.

The building has a channel-shaped stabilising system and the horizontal loads act mainly along two orthogonal directions, that is x and y. Let us first consider the equilibrium for loads acting along y-direction, see Figure 6.12.

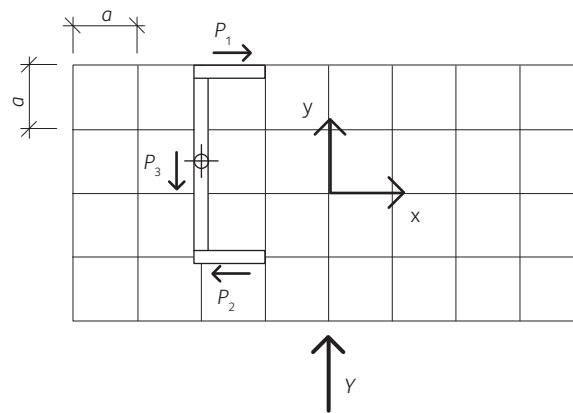


Figure 6.12: Resisting mechanism for horizontal loading perpendicular to the long façade.

The total horizontal force “Y” acting perpendicular to the long façade of the building must be resisted by the forces P_1 , P_2 and P_3 . It is worth to remember that the wall assembly is not considered as a single channel-shaped unit but as three individual, independent elements. Moment equilibrium about the intersection of the unknown forces P_1 and P_3 yields the unknown wall force P_2 :

$$\sum M = 0 \Rightarrow Y \cdot 2 \cdot a - P_2 \cdot 3 \cdot a = 0 \Rightarrow P_2 = \frac{2}{3} \cdot Y \quad \text{Eq. 6.4}$$

Equilibrium in x-direction results in:

$$\sum F_x = 0 \Rightarrow P_1 - P_2 = 0 \Rightarrow P_1 = P_2 = \frac{2}{3} \cdot Y \quad \text{Eq. 6.5}$$

Equilibrium in the y-direction results in:

$$\sum F_y = 0 \Rightarrow P_3 - Y = 0 \Rightarrow P_3 = Y \quad \text{Eq. 6.6}$$

For the case where a lateral force resultants “X” acts perpendicular to the short façade, see Figure 6.13, the force P_3 along the core wall is zero.

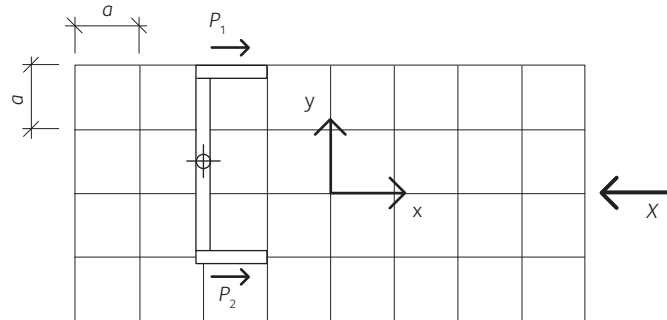


Figure 6.13: Resisting mechanism for horizontal loading perpendicular to the short façade.

In fact:

$$\sum F_y = 0 \Rightarrow P_3 = 0 \quad \text{Eq. 6.7}$$

Rotational equilibrium force about force P1 yields:

$$\sum M = 0 \Rightarrow X \cdot 2 \cdot a - P_2 \cdot 3 \cdot a = 0 \Rightarrow P_2 = \frac{2}{3} \cdot X \quad \text{Eq. 6.8}$$

Equilibrium in x-direction results in:

$$\sum F_x = 0 \Rightarrow P_1 + P_2 = X \Rightarrow P_1 = \frac{1}{3} \cdot X \quad \text{Eq. 6.9}$$

6.4 Strength and stiffness requirements for bracing systems

The 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 (for example wind or earthquake) serve also to increase the buckling strength of the primary members, such as beams and columns.

An adequate bracing system requires both strength and stiffness. A simple brace design formulation such as designing the brace for 1 – 2 % of the member compressive force addresses only the strength criterion. If brace connections are too flexible, they can have a very detrimental effect on the stiffness of the main structure.

6.4.1 Perfectly straight column

Let us study an axially loaded column braced at mid height by means of a bracing element represented in Figure 6.14 by a spring with stiffness C . In order to simplify the problem, we assume that the column is hinged at the mid span. In other words, the bending stiffness of the column is ignored (the more slender the column, the more realistic is the assumption).

Let us now imagine to apply a very small transversal displacement at the column mid-height and let us study the equilibrium of the system.

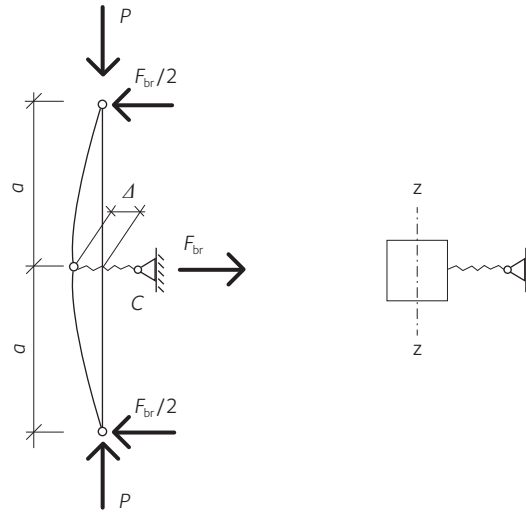


Figure 6.14: Model of a initially straight column braced at mid-height with applied perturbation.

The equilibrium of the system can be studied taking moments about the hinge at mid-height:

$$\sum M = 0 \Rightarrow P \cdot \Delta - \frac{F_{br}}{2} \cdot a = 0 \Rightarrow P \cdot \Delta = \frac{C \cdot \Delta}{2} \cdot a \quad \text{Eq. 6.10}$$

where F_{br} is the bracing force. Note that the term on the left hand side of equation is the external moment M_{unst} , which tends to destabilize the system, while the term on the right hand side is the stabilizing moment M_{st} which occurs due to the presence of the spring at mid-height.

Three types of situations are possible, namely:

$$\text{Unstable} \quad M_{unst} > M_{st} \Rightarrow P > \frac{C \cdot a}{2} \Rightarrow C < \frac{2 \cdot P}{a} \quad \text{Eq. 6.11}$$

$$\text{Stable} \quad M_{unst} < M_{st} \Rightarrow P < \frac{C \cdot a}{2} \Rightarrow C > \frac{2 \cdot P}{a} \quad \text{Eq. 6.12}$$

$$\text{Critical} \quad M_{unst} = M_{st} \Rightarrow P = \frac{C \cdot a}{2} \Rightarrow C = \frac{2 \cdot P}{a} \quad \text{Eq. 6.13}$$

If the equilibrium is stable, it means that when an external perturbation is applied, and when the source of this perturbation is removed the system will return back to its original position, that is the perfectly straight position. On the other hand, if we imagine to gradually increase P , and for each small increment of P we give a small perturbation at the mid-height of the column that produces a displacement Δ , a condition will be reached in which the column – after the given perturbation – will not return to its original position. The load corresponding to this situation is referred to as the critical load. However, the maximum axial load that the column can resist cannot be larger than the buckling load of the column, which in this case is:

$$P \leq P_E = \pi^2 \cdot \frac{E \cdot I}{a^2} \quad \text{Eq. 6.14}$$

P_E is the Euler load for a strut with the length a under compression, E is the modulus of elasticity parallel to grain and I is the second moment of area about the z -axis, according to Figure 6.14.

The corresponding buckling shape, when the stiffness of the spring is large enough for achieving either a stable or a critical equilibrium, is shown in Figure 6.15.

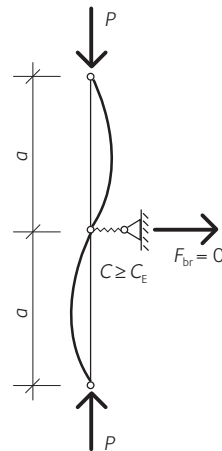


Figure 6.15: Buckling mode for spring stiffness $C \geq C_E$.

The spring stiffness C_E which is necessary in order to achieve the buckling shape shown in Figure 6.15 is therefore:

$$C_E = \frac{2 \cdot P_E}{a} = 2 \cdot \pi^2 \cdot \frac{E \cdot I}{a^3} \quad \text{Eq. 6.15}$$

Figure 6.16 shows the relationship between the load carrying capacity of the column and the stiffness of the bracing element, for an ideal column. When $C < C_E$, there are two equilibrium situations, one stable with no transversal displacements and one critical with a small transversal displacement. This critical situation is achieved when the relationship between P and C is that given by Equation 6.13 (in Figure 6.16 this relationship is described by the line AB). When $C \geq C_E$ there will also be two possible equilibrium situations, again, one stable with no transversal displacement and one critical when $P = P_E$, with the formation of a buckling shape with two half-waves, see Figure 6.15.

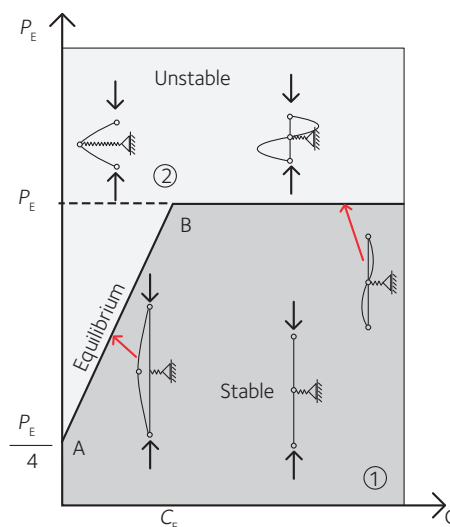


Figure 6.16: Influence of the brace stiffness on the loading carrying capacity of an ideal column. Note that P_E and C_E are defined in Equation 6.14 and Equation 6.15 respectively.

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 . Figure 6.17 shows a column with three intermediate equidistant braces. Increasing the brace stiffness, the buckling mode of the column switches from a single half-sine wave to two, three and finally four half-sine waves, which occurs when C approaches the value $(3,4 \cdot P_E)/a$, Yura (1996).

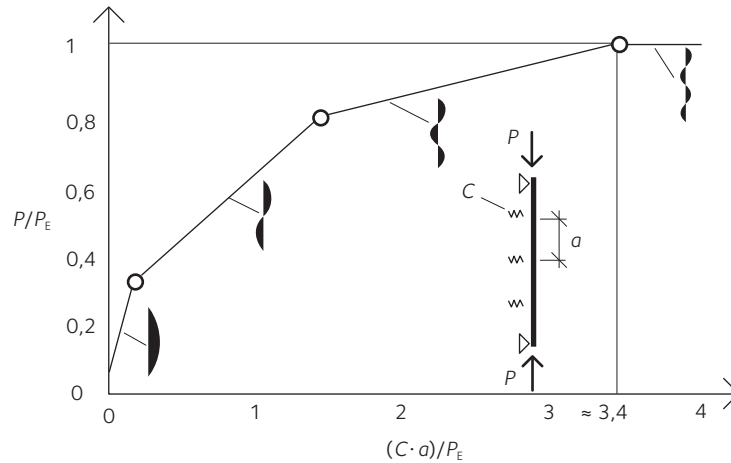


Figure 6.17: Load-stiffness relationship for a column with three intermediate supports. For the case with three intermediate supports, the full bracing stiffness requirement is achieved when $C_E \approx (3,4 \cdot P_E)/a$, where $P_E = \pi^2 EI/a^2$.

If the number of braces is further increased, the brace stiffness necessary to form a series of half-sine waves between all the brace points is slightly larger. For practical cases, when the number of intermediate braces is $n > 4$, the required stiffness should be $C_E \geq (4,0 \cdot P_E)/a$, Yura (1996).

6.4.2 Column with initial imperfections

It is worth observing that for perfectly straight columns with full bracing (that is $C \geq C_E$) there is no force in the brace even at buckling because there is no displacement at the brace point. However, tests have shown that brace forces do develop when a column is subjected to axial loading. The model shown above, with initial straight column, does not give any information about the deformations of the bracing and the corresponding forces which are generated in it. In order to estimate deformation and corresponding bracing force, Winter (1958) proposed a rigid link model with fictitious hinges and with an initial out-of-straightness, Δ_0 , see Figure 6.18.

As a load P is applied there will be additional deflection Δ which can be determined by taking moments about point n :

$$\sum M = 0 \Rightarrow P \cdot (\Delta_0 + \Delta) - \frac{F_{br}}{2} \cdot a = 0 \Rightarrow P \cdot (\Delta_0 + \Delta) = \frac{C \cdot \Delta}{2} \cdot a \quad \text{Eq. 6.16}$$

Observing that $\Delta_T = \Delta_0 + \Delta$, Equation 6.16 can be rewritten in the following manner:

$$P \cdot \Delta_T = \frac{C \cdot (\Delta_T - \Delta_0)}{2} \cdot a \quad \text{Eq. 6.17}$$

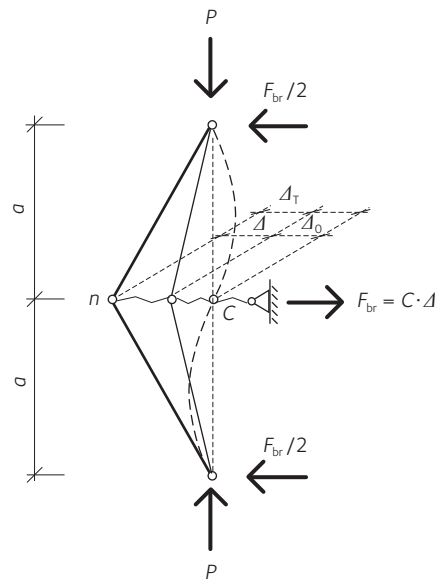


Figure 6.18: Model of column braced at mid-height an initial out-of-straightness.

Let us first study the system when the stiffness of the spring is $C = C_E$, that is stiffness such to make the column to buckle according to Figure 6.15. We impose:

$$C = C_E = \frac{2 \cdot P_E}{a} \quad \text{Eq. 6.18}$$

Substituting Equation 6.18 into Equation 6.17 gives:

$$P \cdot \Delta_T = P_E \cdot (\Delta_T - \Delta_0) \Rightarrow \frac{P}{P_E} = 1 - \frac{1}{\Delta_T / \Delta_0} \quad \text{Eq. 6.19}$$

For $C = C_E$ and $\Delta_0 \neq 0$, the heavy solid line in Figure 6.19 shows the relationship between P and Δ_T / Δ_0 given by Equation 6.19. For $P = 0$, $\Delta_T = \Delta_0$. When P increases and approaches the buckling load $P_E = \pi^2 \cdot E \cdot I / a^2$, the total deflection Δ_T becomes very large (theoretically Δ_T tends to infinity). For example, when the applied load P is about 90 – 95 % of P_E , the total deformation Δ_T becomes as large as 10 – 20 times Δ_0 .

In general this is not acceptable for a real structure. Consider for example a glulam column, with length $h = 6$ m and initial out-of-straightness $\Delta_0 = h/500 = 12$ mm, braced at mid-height. If the chosen brace stiffness should be only $C = C_E$, when a compressive load of the magnitude of $0,95 \cdot P_E$ is applied on the top of the column, the deformation at mid-height could be as large as $20 \times 12 = 240$ mm.

On the other hand, if a brace stiffness twice the value of the ideal stiffness is adopted, that is $C = 2 \cdot C_E$, Equation 6.19 becomes:

$$\frac{P}{P_E} = 2 \cdot \left(1 - \frac{1}{\Delta_T / \Delta_0} \right) \quad \text{valid for } P \leq P_E \quad \text{Eq. 6.20}$$

In this case, the deflections are much smaller than for the case with $C = C_E$, as shown in Figure 6.19. For example, when the load just reaches the buckling load ($P = P_E$), then $\Delta_T = 2 \cdot \Delta_0$, not infinity as in the case with $C = C_E$. For $C = 3 \cdot C_E$ and $P = P_E$, $\Delta_T = 1,5 \cdot \Delta_0$.

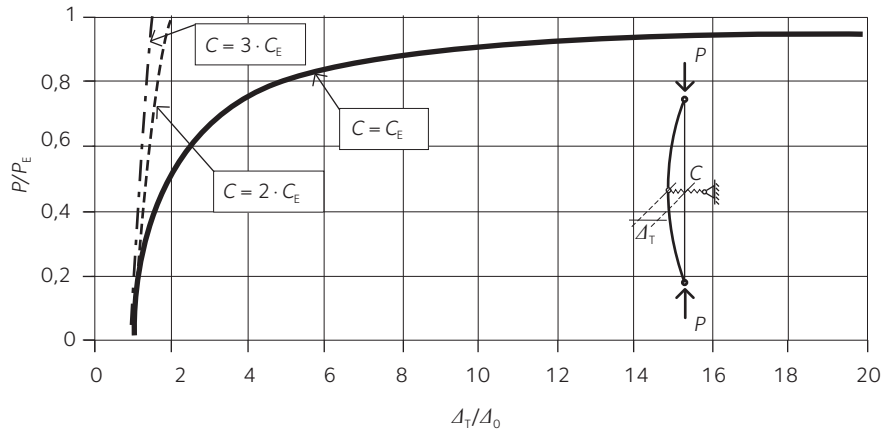


Figure 6.19: Relationship between P and Δ_T for a braced column with initial out-of-straightness.

It is evident that the larger the brace stiffness, the smaller Δ_T . Let us now estimate the forces that occur in the brace when brace stiffness is varying (note that C is derived from Equation 6.17):

$$F_{br} = C \cdot \Delta = \left(\frac{2 \cdot P}{a} \cdot \frac{\Delta_0 + \Delta}{\Delta} \right) \cdot \Delta = \frac{2 \cdot P}{a} \cdot \Delta_T. \quad \text{Eq. 6.21}$$

We study first the system when the stiffness of the spring is $C = C_E$. In that case (see also Equation 6.19):

$$\Delta_T = \frac{\Delta_0}{1 - P/P_E} \Rightarrow F_{br} = \frac{2 \cdot P}{a} \cdot \frac{\Delta_0}{1 - P/P_E} \quad \text{Eq. 6.22}$$

which shows that the brace force is directly related to the magnitude of the initial imperfection. If a member is fairly straight, the brace forces will be small. Conversely, members with large initial out-of-straightness will require stronger braces. Equation 6.22 can be rewritten in the following manner:

$$\frac{P}{P_E} = 1 - \frac{1}{F_{br}/P} \cdot \frac{2 \cdot \Delta_0}{a} \quad \text{Eq. 6.23}$$

A plot of Equation 6.23 for an initial imperfection $\Delta_0 = (2 \cdot a)/500$ is shown in Figure 6.20. If the brace stiffness is just equal to the ideal stiffness, then the brace force becomes very large as the buckling load is approached; this occurs because Δ_T becomes very large, as shown in Figure 6.20. For example, at $P = 0,84 \cdot P_E$, Equation 6.22 gives a brace force of 5 % of P_E . It should be emphasised that Equation 6.22 is only valid for $F_{br}/P > (2 \cdot \Delta_0/a)$, otherwise equilibrium is not possible.

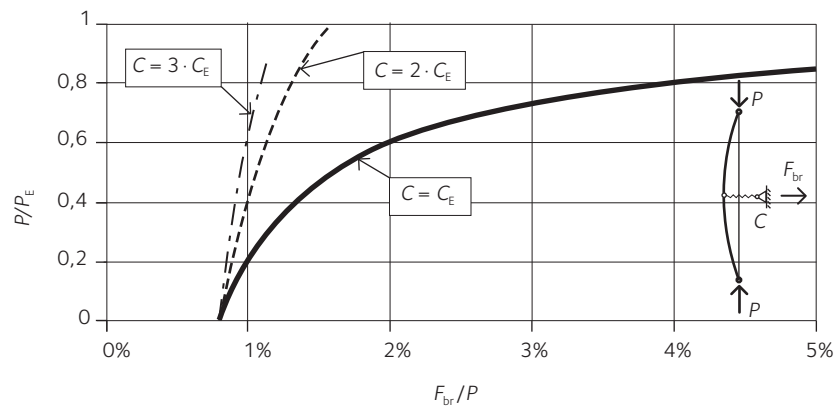


Figure 6.20: Relationship between P and F_{br} for a braced column with initial out-of-straightness equal to $\Delta_0/(2 \cdot a) = 1/500$.

Therefore, it can be concluded that a brace system will in general not be satisfactory if its stiffness is just equal to the theoretical ideal required stiffness C_E , because the brace forces will get too large. If the brace stiffness is ‘over designed’, for example by $C = 2 \cdot C_E$ or $C = 3 \cdot C_E$ as shown in Figure 6.20, then the brace forces will be more reasonable.

6.4.3 Beam, truss or column systems

Where a bracing system is required to provide lateral stability to a series of compression or bending members (for example columns, trusses or beams), this is effectively achieved by providing lateral stiffness using truss or diaphragm action within the plane of the bracing structure, see Figure 6.21.

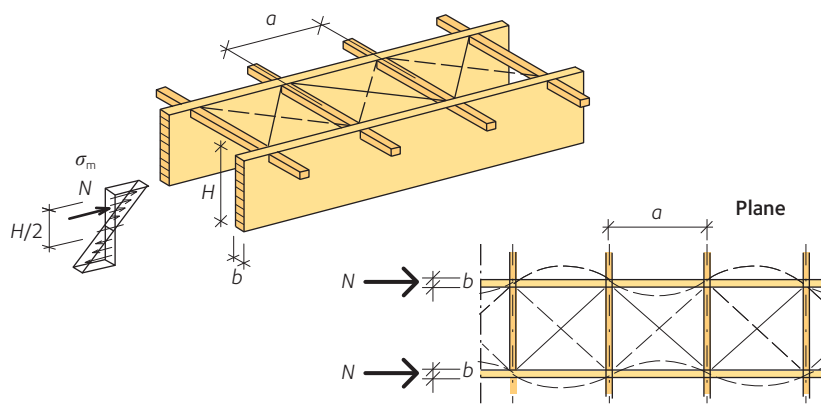


Figure 6.21: Typical bracing of beam system for preventing lateral buckling.

For the general case of a series of similar compression members that require to be braced at positions along their length, it can be assumed that each compression member under load will have an initial out-of-straightness Δ_0 , as shown in Figure 6.22.

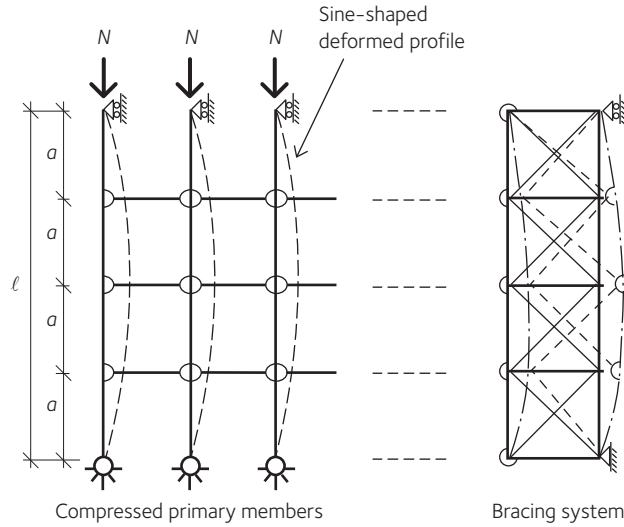


Figure 6.22: Model for the determination of the brace forces: bracing system for a series of compression or bending members.

A single member subjected to an axial force N can alternatively be analysed by replacing the effect of the initial deviation from straightness on the deflection behaviour, by the effect of an equivalent lateral load q_{hl} acting on the member when it is in an initially straight condition – such that the maximum bending moment in each case will be the same, see Figure 6.23:

$$N \cdot \Delta_0 = q_{hl} \cdot \frac{\ell^2}{8} \Rightarrow q_{hl} = \frac{8 \cdot N \cdot \Delta_0}{\ell^2} \quad \text{Eq. 6.24}$$

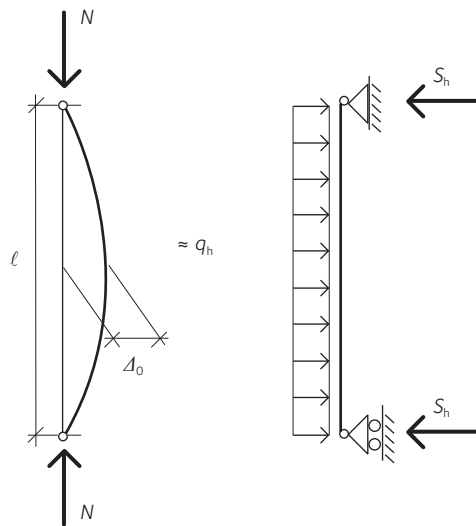


Figure 6.23: Equivalent lateral load acting on the bracing system.

Where n members are to be braced and each member is subjected to a compression force N , assuming that all members contribute to the loading to be taken by the bracing system, the total compression load will be $n \cdot N$:

$$q_h = n \cdot \frac{8 \cdot N \cdot \Delta_0}{\ell^2} \quad \text{Eq. 6.25}$$

For timber members, the initial out-of-straightness defined as Δ_0/ℓ is normally assumed to be 1/500 and 1/300 for glulam and structural timber respectively. Substituting these values into Equation 6.25 yields:

$$q_h \approx n \cdot \frac{N}{60 \cdot \ell} \quad \text{for glulam structures} \quad \text{Eq. 6.26}$$

and:

$$q_h \approx n \cdot \frac{N}{40 \cdot \ell} \quad \text{for structural timber structures} \quad \text{Eq. 6.27}$$

For a member subjected to bending, such as that shown in Figure 6.21, the action of the bending moment can conservatively be assumed to be $M \approx \frac{2}{3} \cdot N \cdot h$, where h is the depth of the beam.

It is important to observe that if the bracing is inadequately stiff, the maximum load carrying capacity of the beam cannot be achieved. This is shown in Figure 6.24. The test results also show that lateral deflections of the bracing beam (and thus the lateral deflection of the timber beam) dramatically increase when the load approaches a certain value, which depends on the lateral bending stiffness of the bracing beam (UPE-type steel beams were used as bracing in the experiment). As an example, when the bending stress in the tested timber beam is 20 MPa, the corresponding bending moment is 7,5 kNm and the vertical load is 8,2 kN/m. The lateral load on the bracing system which is generated by the out-of-plane deflection of the timber beam is approximately:

- 0,70 kN/m (out-of-plane deflection ~ 25 mm) for the less stiff bracing beam (UPE 50)
- 0,35 kN/m (out-of-plane deflection ~ 6 mm) for the stiffer bracing beam (UPE 80).

These results show that the brace force increases with decreasing lateral stiffness of the bracing system.

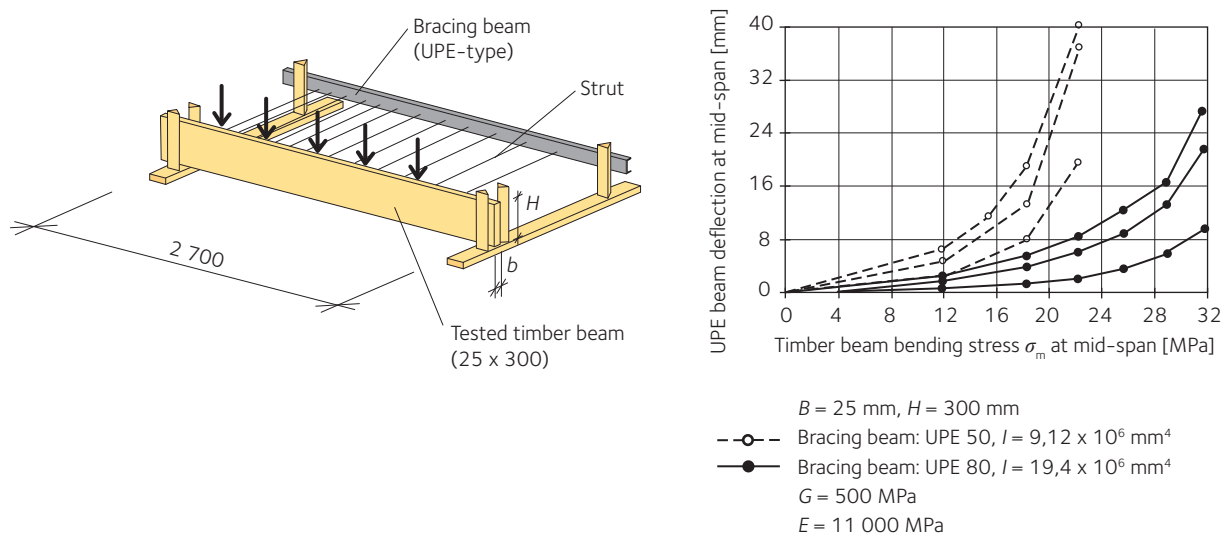


Figure 6.24: Test setup for determining brace force, and test results. The struts are connected to the top edge of the tested timber beam (Holzbau Handbuch, 2000).

Experimental results also show that in case of double pitched beams with upper side inclination of 5°, the brace force is 10 % to 30 % higher than in the case of similar timber beams with parallel upper and lower sides, Holzbau Handbuch (2000).

According to Eurocode 5, the bracing system shall – in addition to other horizontal loads, for example wind or earthquake – be designed for a horizontal, uniformly distributed load:

$$q_h = n \cdot \frac{M}{30h \cdot \ell} \cdot (1 - k_{\text{crit}}) \quad \text{Eq. 6.28}$$

where:

M	maximum moment in the beam
h	depth of beam
ℓ	span of the beam [m]
n	total number of laterally braced beams
k_{crit}	factor that takes into account the risk for lateral buckling in an unbraced beam (see Section 3.2.3.3)

The reduction factor k_ℓ in Eurocode 5 intends to consider that greater care in workmanship can be expected in large span structures. Since it is not representative under normal conditions, it has here been omitted.

The laterally bracing structure marked with (1) in Figure 6.25 must be designed for the load q_h according to Equation 6.28 in addition to the external load from wind q_w shown in the figure.

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

$$F_2 = n_b \frac{q_h \cdot a}{n} \quad \text{Eq. 6.29}$$

where:

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

n_b = number of roof beams braced by the force transferred via the purlin or roof sheeting.

The bracing structure shall be stiff enough to limit the deflection due to the design load q_h to $\ell/700$ and due to the total load, including for example wind load, to $\ell/500$.

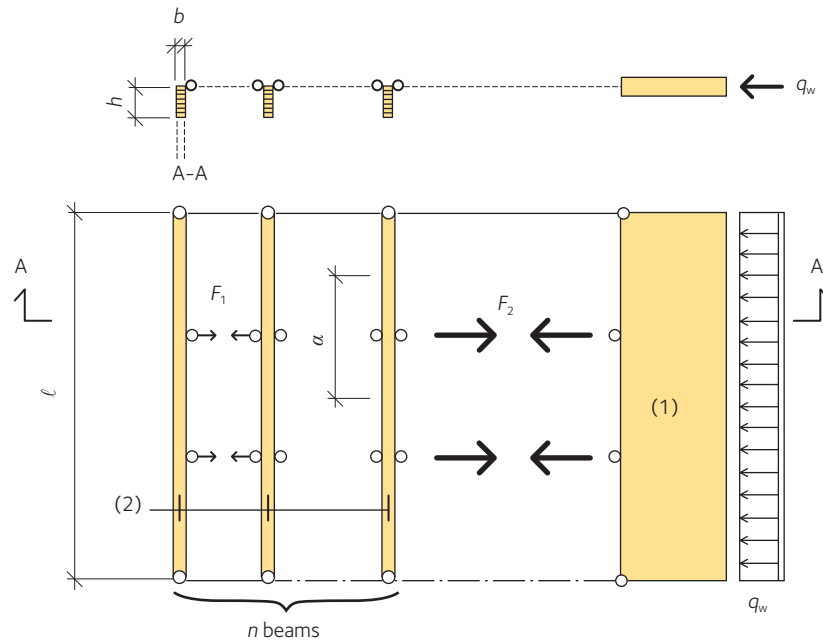


Figure 6.25: Lateral bracing of roof beams. 1) Laterally bracing structure, 2) roof beams.

6.5 Typical bracing systems for heavy timber structures

To be able to take up horizontal loads, heavy timber structures are normally stabilised in accordance with one of the following alternatives:

- The diaphragm action of the roof which transfers the horizontal load to wall diaphragms, normally placed in the gable walls.
- Wind trusses in the roof which transfer the horizontal load from columns to braced columns, normally placed in the gable walls as in Figure 6.26.
- One or both of the columns is rigidly fixed to the foundation as in Figure 6.27 a).
- One or both of the columns is rigidly fixed to the beam as in Figure 6.27 b), forming a three- or two-pin frame respectively.
- The structure is complemented by diagonals as in Figure 6.27 c). For functional reasons this is usually only possible in gables.
- The structure is rigidly connected to a wall diaphragm in its own plane.

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

- The sheets forming the roof can be jointed so that shear forces can be transferred.
- The roof diaphragm is stiffened along its edges by beams designed and fixed to take up axial forces. Edge beams at right angles to the profiling can be of timber, while edge beams parallel with the profiling are most easily made in steel.

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

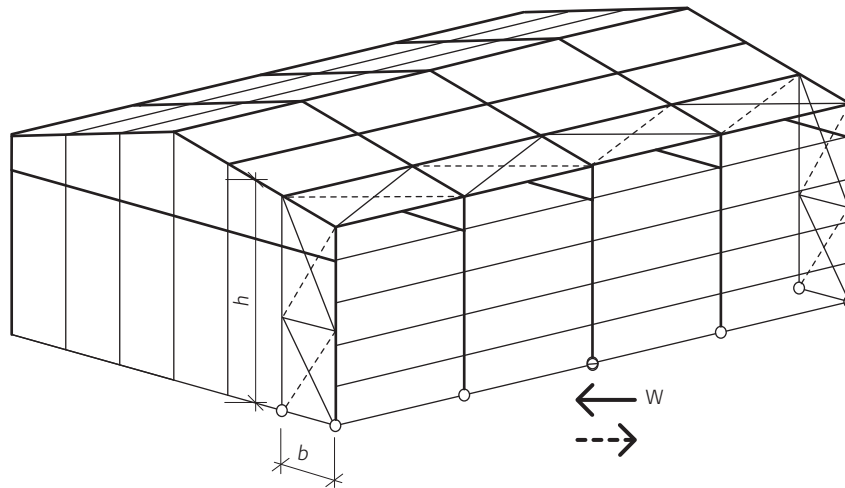


Figure 6.26: Bracing of heavy timber structure using wind trusses and wall bracing.

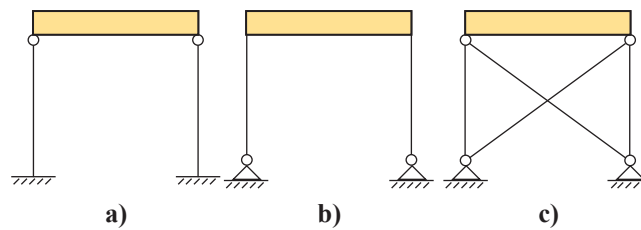


Figure 6.27: Examples of structures which are stable in their own plane.

Frames and arches are stable in their own plane, as are column and beam systems with columns rigidly fixed in the foundations. Systems with hinged columns must, on the other hand, be stabilised by diaphragm action in the roof or by wind trusses in the plane of the roof. The principle of wind bracing with trusses along the side of the building is shown in Figure 6.28. The roof beams (primary beams) are often used as bottom and top chords in the truss, with some of the purlins as posts (or compression struts). The purlins, if they are used as posts for the wind truss, can not be designed as Gerber beams. Only diagonals are added as stiffeners, often crosses of steel rods or sometimes timber diagonals, which are stiff in compression. The joints between the various components forming part of the wind truss must be designed with regard to the forces which arise and to eccentricity. Truss posts and chords are often in different planes.

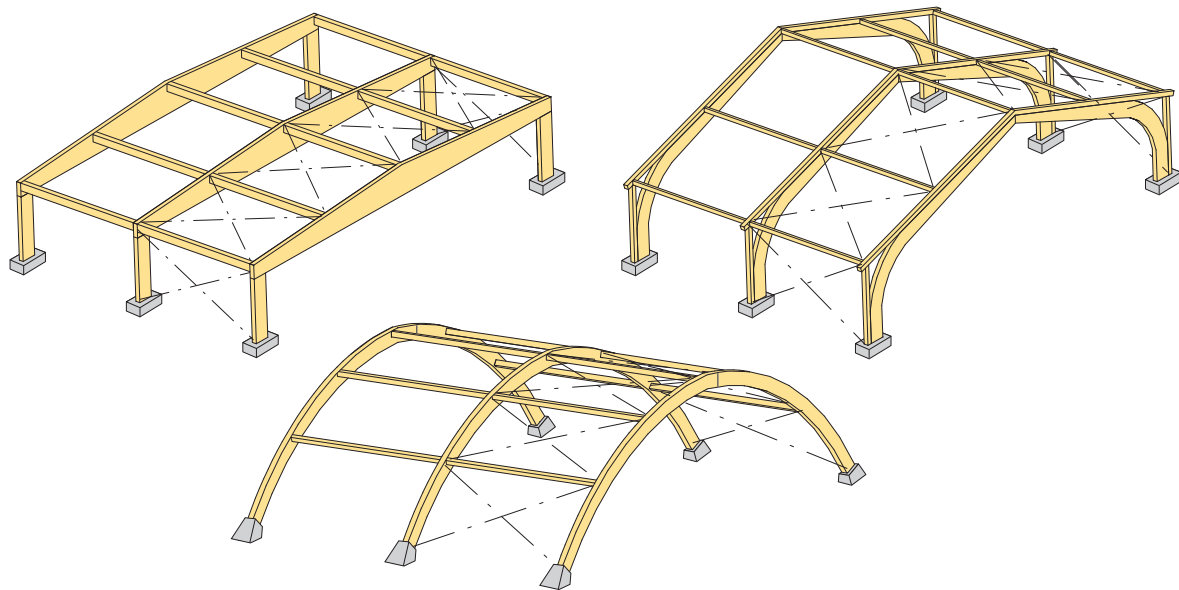


Figure 6.28: Wind bracing of frame and arch structures.

The wind truss is often designed so that it can be used for stabilisation during the erection of the structure. In smaller buildings (say with a longitudinal dimension of about 30 – 40 m), only one wind truss in the roof can be sufficient. The purlins or the roof diaphragm must then be able to transfer both compression and tension forces from the other end. In longer buildings it can be suitable, on the other hand (not least with regard to stability during erection) to arrange two or more wind trusses in different bays within the building. The wind truss can with advantage be placed in the second bay from the end. The design of joints is then not affected by the end wall structure (gable), which usually differs from that in the rest of the building. Other types of bracing arrangements for achieving diaphragm action in the roof are shown in Figure 6.29.

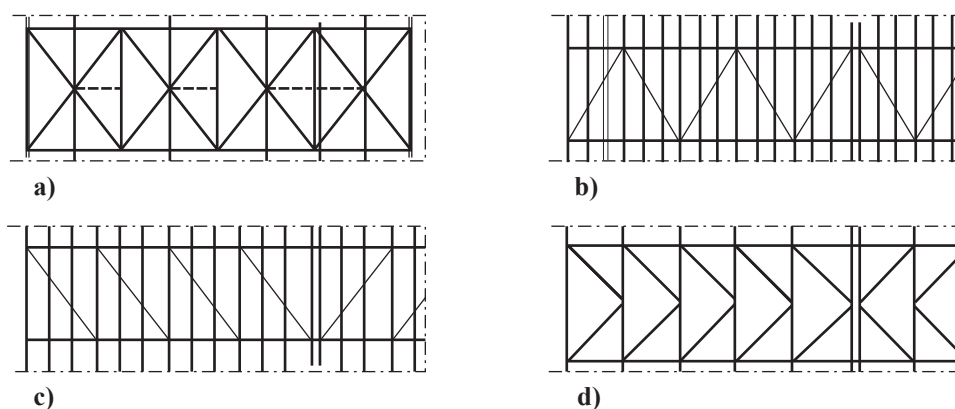


Figure 6.29: Roof bracing arrangements.

Since roof diaphragms or wind trusses are often used as lateral bracing of roof beams, they must – in addition to other horizontal loads – be designed for a horizontal, uniformly distributed load q , derived according to the method shown in Section 6.4.3. The structure in hall-type buildings with large spans (> 15 m) containing spaces where large numbers of people are present at the same time, for example sports or exhibition halls and department stores, shall be designed in such a way that the risk of progressive collapse as a result of an accidental load is acceptably small.

It is important to observe that heavy timber structures must be able to transmit loads from the roof level onto the foundations. Moreover, stability must be ensured against all types of mechanisms that can occur, including lateral buckling. Figure 6.30 illustrates this concept. If a bay of a structure is to be stabilized against lateral loads, the first thing to do is to prevent columns to move laterally which may be achieved, for example, by introducing diagonal members as shown in Figure 6.30 a). However, although the columns are now braced, loads from the roof plane cannot be transmitted to the foundation through diagonal members. Therefore, the beams need – at their support points – a bracing system which permits the transmission of loads from the roof to the underlying structure, for example as shown in Figure 6.30 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 work as compression and tension chords, see Figure 6.30 c).

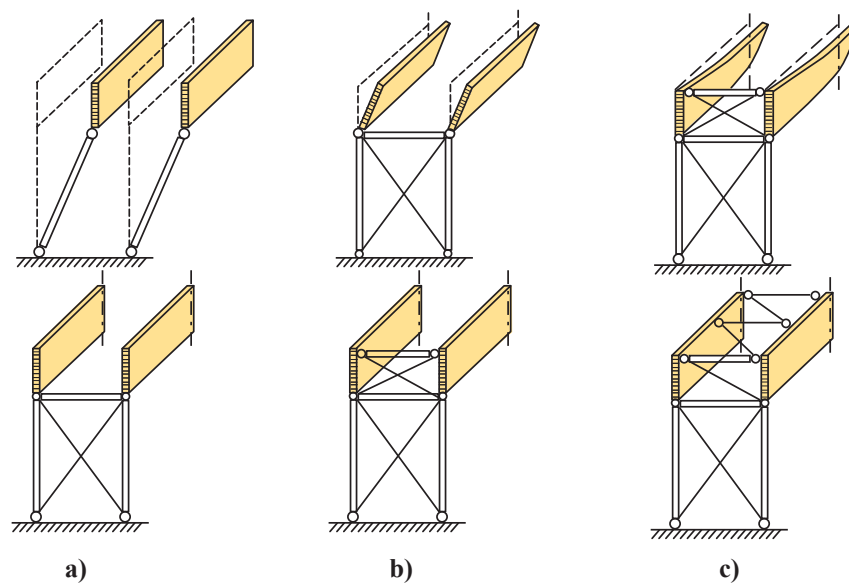


Figure 6.30: Example of a proper lateral bracing of a bay in a larger timber building.

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

Normally, it is sufficient to estimate the horizontal stability of the remainder of the building if one main beam fails. In buildings containing spaces for large numbers of people it must also be shown that the damaged bay can be spanned by an alternative load-bearing system, for example by utilising the line- and membrane effect of purlins and roofing. This can however also be a disadvantage in cases where a primary girder fails due to a design or construction error which is similar for all girders in the building. In that case adjacent girders may also fail in a progressive manner following the failure of one bay. Several such cases have been experienced during recent years; see Sørensen et al. (2011). The risk of progressive failure is then higher if

the bays are structurally connected to each other in the longitudinal direction of the building. In such situations a better strategy is to decouple the system to ensure that failure in one bay will not spread to adjacent bays.

6.6 Special topics

In this section some relevant topics related to bracing systems will be briefly discussed, namely:

- forces that arise due to the slope of the roof
- stability of the upper chord of low-arch or low-truss bridges
- bracing of continuous beams at intermediary supports.

6.6.1 Forces arising due to the slope of the roof

In design it should be observed that the wind truss is seldom plane. At nodes where the members of the truss change direction, for example at the ridge of a double-pitched roof, force components across the roof arise which must be taken care of by the primary structure, see Figure 6.31.

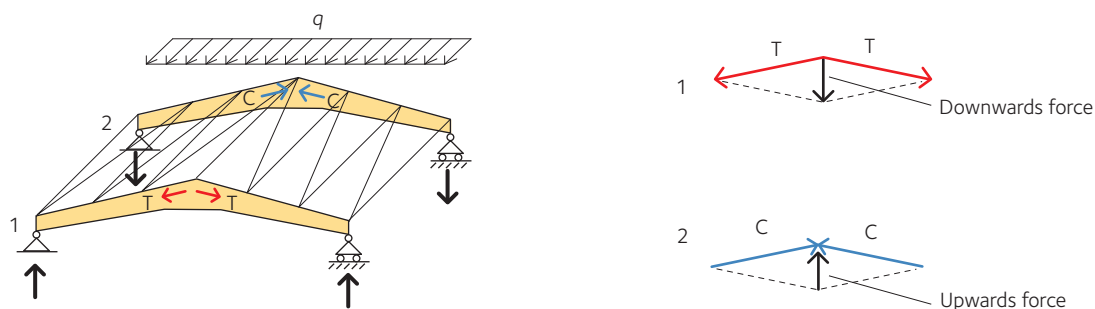


Figure 6.31: Upward and downward forces that generate due to the slope of the roof.

6.6.2 Stability of the upper chord of low-arch or low-truss bridges

In absence of upper chord bracing of a low-arch or low-truss bridge, the lateral buckling of the top chord is only resisted by the elastic reaction of the vertical and/or diagonal members of the truss or arch. In that case it is difficult to verify sufficient bracing of the compressed part of the truss or arch. The stability of the upper chord or of the arch can be increased by increasing the rigidity of the lateral supports, for example as shown in Figure 6.32. The upper chord of a truss or an arch can then be considered as a bar with hinged ends compressed by forces distributed along its length and elastically supported at intermediate points as shown in Figure 6.32. At the supports there are usually frames or bracing members of considerable rigidity, so that the ends of the chord can be considered as immovable in the lateral direction.

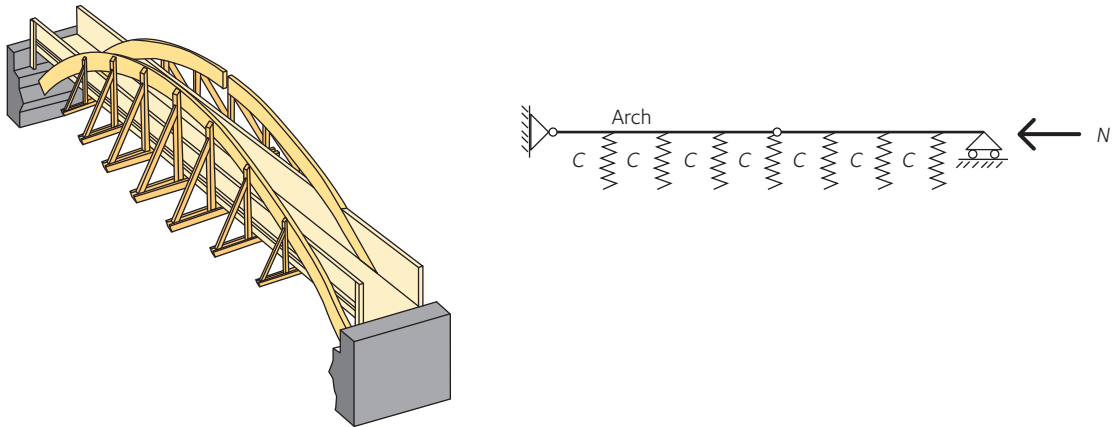


Figure 6.32: The stability of a low-arch bridge can be increased by increasing the stiffness of the lateral supports (that is by increasing the spring stiffness C in the model on the right).

For a constant cross section of the chord and a constant compressive force, the minimum rigidity, at which the supports begin to behave as though they were absolutely rigid is found in Section 6.4.1.

6.6.3 Bracing of continuous beams at intermediary supports

In buildings, for example of hall type, the primary beams are often made continuous, that is with the beam supported by more than two columns. As a rule, the horizontal frame bracing, for example wind truss or roof slab, is at the level of the top of the beam. Therefore, in the zone around the intermediate support, the beam – which is subjected to compression at the underside – must be braced at the underside in order to prevent it from buckling sideways (lateral torsional buckling). The bracing of the underside of the beam can be performed for example as shown in Figure 6.33.

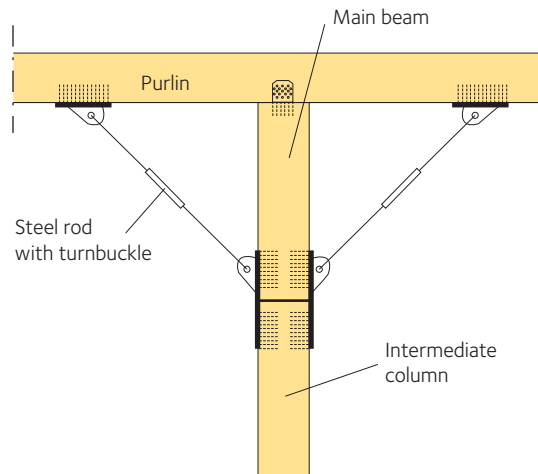


Figure 6.33: Bracing of the underside of a continuous beam at intermediary supports.

The bracing should preferably be such that it can only take tension. This is to avoid possible torsion of the beam due to uneven load distribution which could occur, for example, when strong wind takes place after snowing. In such a case big masses of snow could collect in one span of the beam leaving the adjacent span more or less unloaded. Thus, if the bracing diagonals shown in Figure 6.33 were able to take both compression and tension, the beam would twist on the support with possible risk for collapse of the roof or part of it.

The horizontal force acting on the bracing can be calculated by the following formula, Limträhandbok Del 2 (2016):

$$H = \frac{M}{70 \cdot h} \quad \text{Eq. 6.30}$$

where:

M the moment in the beam at support
 h the depth of the beam.

Similar bracing arrangement as that shown in Figure 6.33 may be needed also for simply supported beams or arches in circumstances where negative bending moment can occur. An example could be long truss beams that support a light-weight roof. In such a case the lower chord of the truss may need to be braced in order to avoid lateral buckling due to wind suction.

6.7 Bracing of timber framed buildings

The behaviour of a roof or a floor under lateral loading depends on its structure. Whilst trussed roof framing and concrete slabs are relatively rigid, skin constructions (for example made of wood based panels) are normally rather flexible. In flexible skin-framed structure the floor or roof sheathing or decking acts as a diaphragm, when properly fastened with shear connectors to the purlins and to the perimeter members so that shear can be transferred. A typical flexible diaphragm is shown in Figure 6.34.

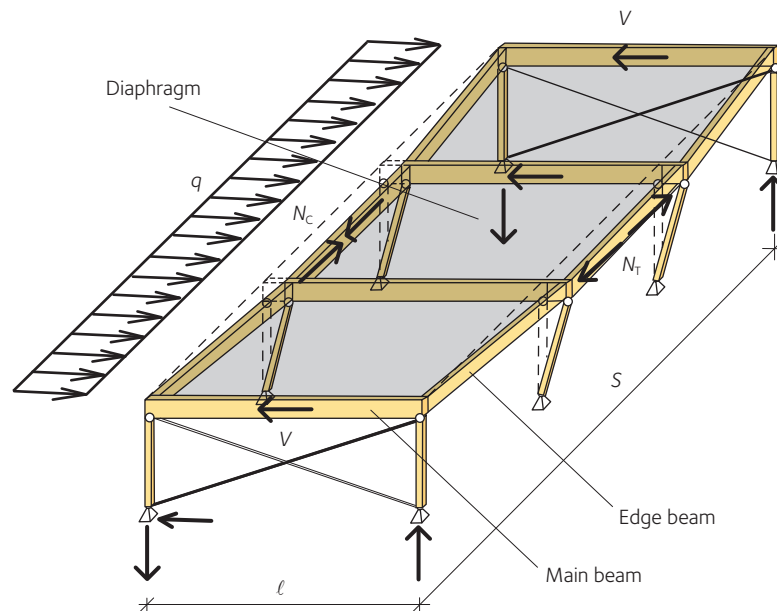


Figure 6.34: Diaphragm action of the roof (flexible diaphragm, for example wood based panel sheathing).

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:

$$v = \frac{V}{b} \quad \text{Eq. 6.34}$$

This means that the force in individual fasteners connecting the sheathing to the frame must be designed for a force $F_v = v \cdot s$, where s is the spacing between the fasteners. Similarly the sheathing 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.

6.7.2 The in-plane resistance of shear walls under horizontal loading

Timber frame shear walls can be sheathed on one or both faces with the sheathing securely fixed to both studs and rails, enabling the wall to act as a rigid diaphragm. The sheathing panels may be made of wooden based materials such as plywood, OSB (oriented strand boards) or hardboard with dimension $b \times h = 1\,200 \times 2\,400 \text{ mm}^2$, see Figure 6.36. The thickness of the panels is normally 12 mm or more. Gypsum boards or various types of cement based boards can also be used as sheathing panels. The frame members (studs and rails) are normally made of strength graded timber. In Sweden a typical cross section adopted for such members is $45 \times 120 \text{ mm}^2$ and the fixing between sheathing and framing is normally made by means of nails or screws.

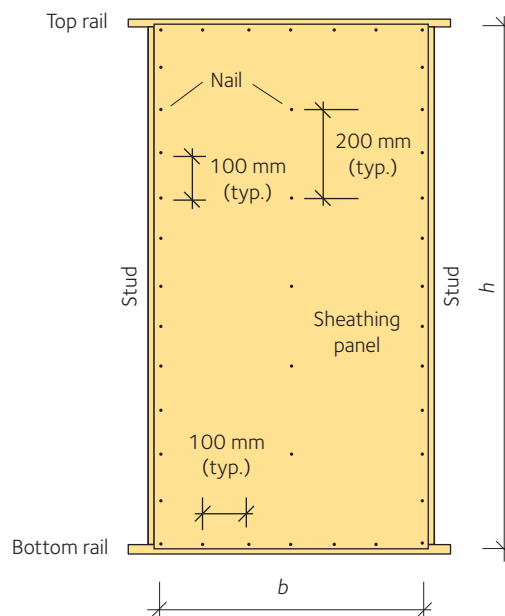


Figure 6.36: A typical shear wall unit built up of a timber frame and a sheet.

The sheathing in shear walls is usually relatively stiff with regard to in-plane deformations and the shear stiffness of the walls depends mainly on the stiffness of the sheathing-to-framing joints. When used in low buildings the stabilizing capacity is normally high compared to the actual horizontal loading.

The sheathing-to-framing joints are normally also decisive for the racking resistance when the lateral external force is applied as shown in Figure 6.37 b). Horizontal sliding of the wall is resisted by anchorages such as anchor bolts that connect the bottom rail to the substrate. Overturning about the bottom corners may also occur (particularly in walls with large height/width ratios) if appropriate holding-down fixings are not provided, as shown in Figure 6.37 c). In most timber-framed buildings, beams and floors are designed as simply supported elements on pin-jointed walls and the lateral strength and stability of the structure is provided by the use of such vertical diaphragms or shear walls. In timber design, the in-plane lateral resistance of a wall diaphragm is referred to as the racking resistance of the wall.

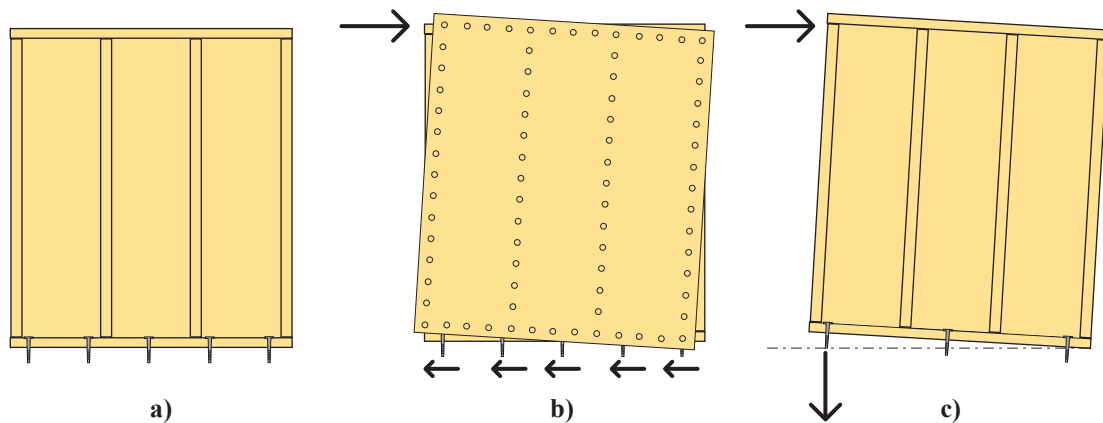


Figure 6.37: Wall diaphragm resisting racking loads: a) Unloaded wall diaphragm, b) racking resistance, c) overturning.

Different models can be used to evaluate the racking resistance of a shear wall, most of them based on the shear capacity of an individual fastener, for example a nail connector. Herein, only one model is described, namely the “plastic model”, which gives a lower bound solution, Källsner et al. (2009). This method can only be applied if the leading stud – that is the stud which is subjected to upwards vertical reaction force – is fully anchored against uplift (normally, holddowns at the end of the wall are used to prevent uplift of the leading stud). In such a case, the shear wall can be regarded as fully anchored and all the sheathing-to-framing connectors can be assumed to reach their maximum shear capacity.

Let us study first a single wall unit. A uniform shear flow is assumed along the edge of a panel, in compliance with equilibrium condition of the wall, see Figure 6.38.

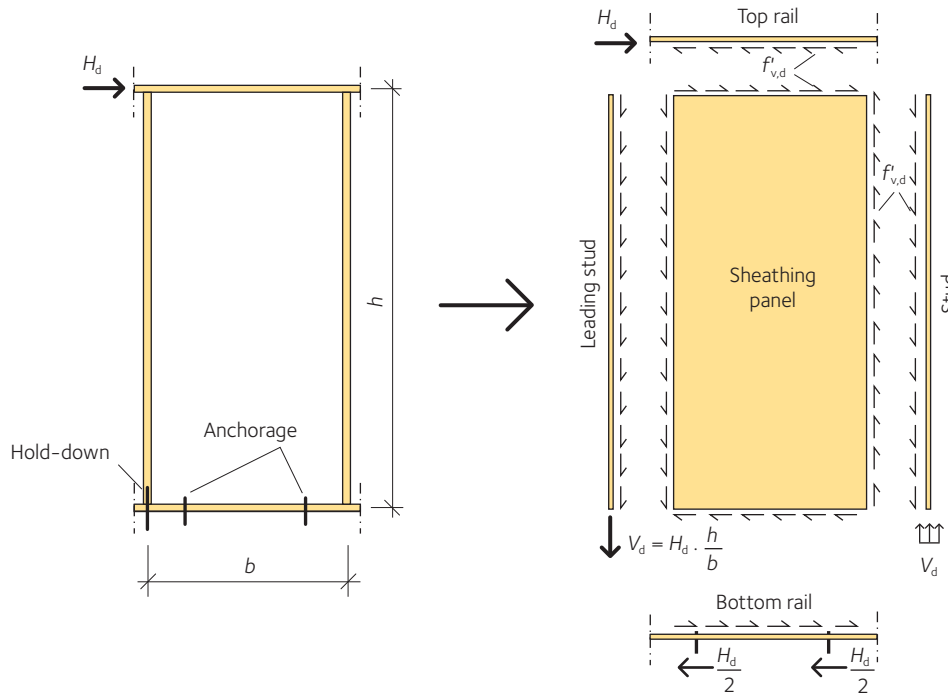


Figure 6.38: Forces in the plastic shear wall model.

The racking capacity of the wall unit H_d can be determined from equilibrium of horizontal forces at the top rail:

$$H_d = f'_{v,d} \cdot b \quad \text{Eq. 6.35}$$

where $f'_{v,d}$ is the shear flow strength (N/mm) at the sheathing-to-frame joint and b is the width of the panel. In reality, the shear between sheathing and frame is not continuous but punctual, since it is transmitted by the connectors – which are placed with a centre-to-centre spacing s (typically $s = 100$ mm at the perimeter of the wall). According to the plastic model all individual connectors carry the full design load $F_{v,d}$. For equal spacing s between the connectors we get $f'_{v,d} = F_{v,d}/s$, so that:

$$H_d = \frac{b}{s} \cdot F_{v,d} \quad \text{Eq. 6.36}$$

The hold-down force of the panel can also be calculated as:

$$V_d = v_d \cdot h = \frac{h}{s} \cdot F_{v,d} \quad \text{Eq. 6.37}$$

where h is the height of the shear wall.

For walls made up of several panels (or wall units), for example according to Figure 6.37, the racking capacity is the sum of the racking capacity of each single wall unit:

$$H_d = \sum_i H_{d,i} \quad \text{Eq. 6.38}$$

where $H_{d,i}$ is the resistance of wall unit i .

In design, when openings are present in the shear wall, the capacity of a wall unit including an opening is normally disregarded. Moreover, the wall must be anchored on each side of the opening. This means that a building from a static point of view will consist of a number of continuous vertical strips of shear walls that act separately.

The plastic model described above rests on the assumption that the connectors between panels and framing behave in an ideal plastic manner. For most wood based panels connected to the framing with nail and screw connectors this is a reasonable assumption. For other panel products such as gypsum boards or cement based panel products the behaviour of the connections may be more brittle and application of models based on plasticity cannot be guaranteed. Models based on elastic behaviour of the connectors are also available; see for example Carling (1992). Utilizing an elastic model means that the racking resistance will be determined by high forces in a few connectors near the corners, that is a more conservative design than that obtained from the plastic model.

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Chapter 7

Design for serviceability

Annika Mårtensson

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7 Design for serviceability (SLS)

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

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

Some of the notations used in this chapter coincide with the ones adopted in Eurocode but the discussion is general and describes the phenomenon and the design methods that can be used in serviceability state analysis.

7.1 Performance requirements

Deflections and movements in a building must be limited to avoid damage and other undesirable effects in service. A building must have sufficient strength to carry the maximum expected loads with an adequate factor of safety. Structures designed in 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 like timber.

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. This is often a problem in the design process today since the knowledge is lacking with regard to both requirements and prediction methods.

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

In principle, serviceability requirements should be set by the client in agreement with the builder and not by codes. The codes give, however, 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 manifest in damage to structural as well as non-structural elements, in detrimental effects on use and in 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, see Chapter 6. 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. Examples are insufficient slope on roofs and problems with opening doors and windows. In special buildings such as gymnasiums or facilities with high requirements of surface planeness for example due to sensitive equipment, deflections may also lead to troubles.

Serviceability limits should also be applied to buildings and their components to avoid non-structural elements being introduced into the load path. If there is insufficient separation between deflecting primary structural elements and non-structural components, the load may be transferred through the non-structural element. Such non-structural elements may include walls, windows, ceilings, floor and roof coverings, facades, lifts etc. The resulting damage may include cracking and buckling of some elements of moderate stiffness and integrity, or tearing and folding of more flexible items such as coverings and linings. Whereas these forms of failure in general do not affect the stability of the structure, they may render the building unserviceable. Limitations in order to prevent damage to non-structural elements should in many cases be determined on basis of variable loads, with respect to the higher values that these may attain during shorter periods.

Excessive deflections 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 deflection to ensure that confidence is maintained. Such limits apply especially to long term deflections and not so much to shorter 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 various more or less important reasons. In practical design it is convenient to define two principally different categories of deformation control:

- to avoid **permanent damage (irreversible effects)**
- to ensure a good **appearance and general utility (reversible effects)**.

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

Problems with regard to serviceability include also vibrations of different origin. Vibrations due to foot-fall and/or machines of different types lead to human discomfort but may also lead to problems with sensitive equipment as well as structural behaviour. The requirements in those cases are often even more complex than with static deflection. The same discussion as with the 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.

7.2 Static deformation

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

Figure 7.1 illustrates in principle the deflection behaviour of a beam loaded with 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, Mårtensson (2003). 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 7.1, occur both for snow load and imposed (live) loads in the most common types of buildings.

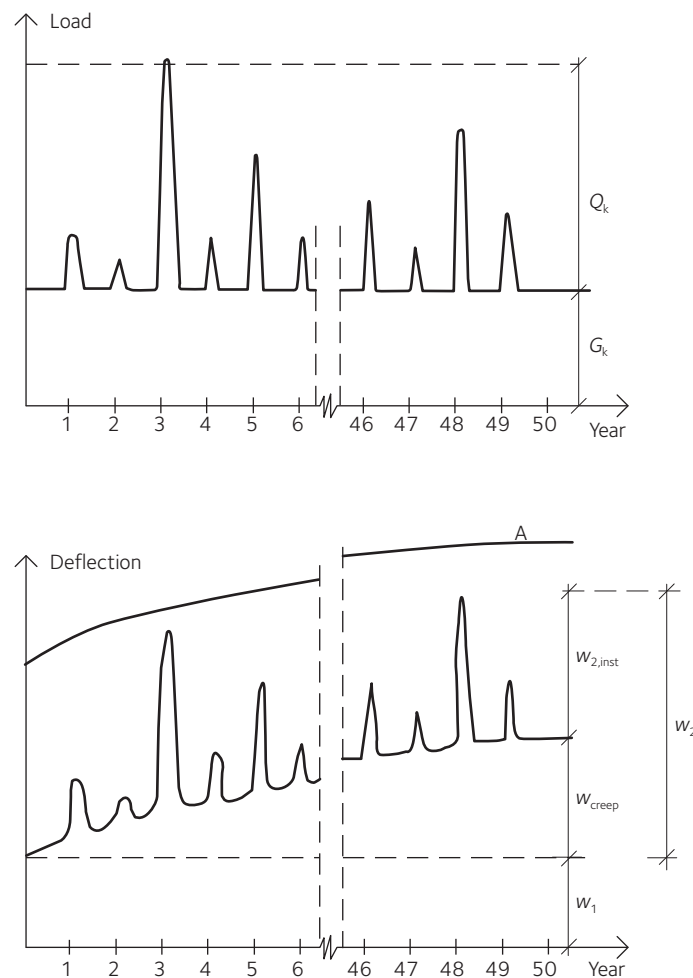


Figure 7.1: Time variation in principle for deflection (lower figure) of a beam with permanent and variable loads according to the upper figure. Curve A shows the creep deflection if the beam is loaded with the characteristic loads $G_k + Q_k$ during the whole period.

7.2.1 Deformation

In Figure 7.2 a schematic figure of the deformation for a timber beam is given with 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 any time during the structure's service life.

One part of the load will be sustained for a longer time period and will give rise to a gradual increase in the deflection, w_{creep} . In order to determine the total deflection after 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 on the modulus of elasticity and in some cases the shear modulus is used. It is most reasonable to use the mean values of these moduli in the serviceability state.

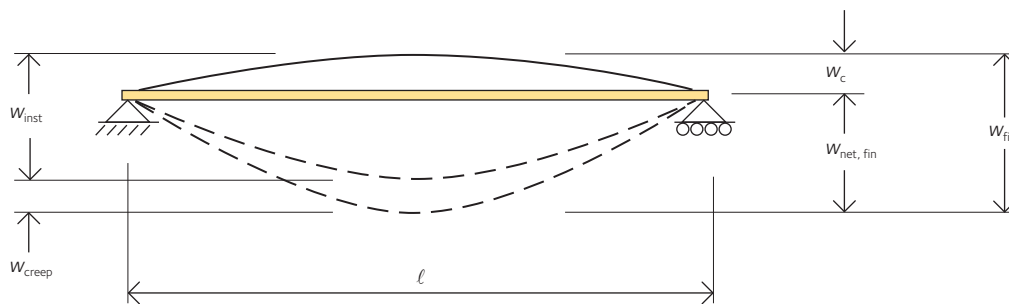


Figure 7.2: Definitions of deflection.

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, Porteous and Kermani (2007). Consider, for example, a simply supported rectangular beam of depth h and design span ℓ carrying a point load at mid-span. The ratio of the instantaneous deflection at mid-span caused by the shear forces, $w_{inst,v}$, to the instantaneous deflection at mid-span caused by the bending moment, $w_{inst,m}$, will be:

$$\frac{w_{inst,m}}{w_{inst,v}} = 1,2 \frac{E}{G} \left(\frac{h}{\ell} \right)^2 \quad \text{Eq. 7.1}$$

With softwood for structural purposes, E/G is approximately 16 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 timber structures.

One special case of deformation is settlement of structures. This is often treated as an ultimate limit state design problem by checking the compressive stresses in contact areas where the timber is compressed perpendicular to grain, see Figure 7.3, Mårtensson (2003). It is, however, in practice often a serviceability problem since the settlements of the joints can be

a problem rather than actual failure of the joint. In a multi-storey wood frame building, high magnitude forces can result in the lower floors and at the foundation level. The compressive forces perpendicular to grain lead to deformations which may become relatively large especially in comparison with deformations parallel to grain.

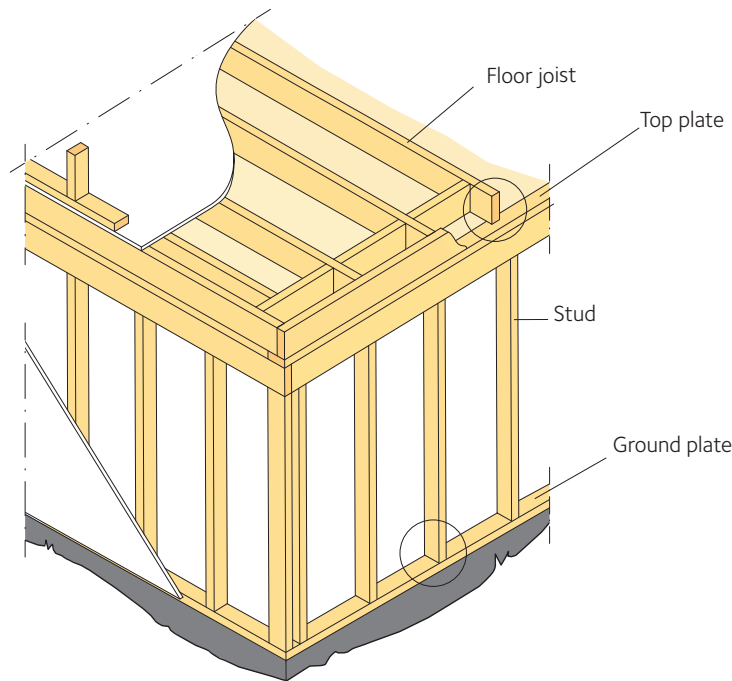


Figure 7.3: Platform frame construction. Circles indicate details with high compressive stress perpendicular to grain.

In order to estimate the magnitude of the compression of a joint it is important to observe that the whole joint design has to be considered. See for instance a joint like the one in Figure 7.4. If the bearing length covers the whole surface of the ground plate, that is $\ell_e = 0$, it would be reasonable to assume that the deformation Δ would be equal to:

$$\Delta = \frac{\sigma_{\perp} d}{E_{\perp}} \quad \text{Eq. 7.2}$$

where d is the ground plate depth, E_{\perp} is the elastic modulus perpendicular to grain and the stress is given by:

$$\sigma_{\perp} = \frac{P}{b\ell} \quad \text{Eq. 7.3}$$

where b is the ground plate width, ℓ is the length of the loaded surface and P is the applied load. Experiments have shown that in the case where the load does not cover the entire surface the deformation is less than that defined by Equations 7.2 and 7.3.

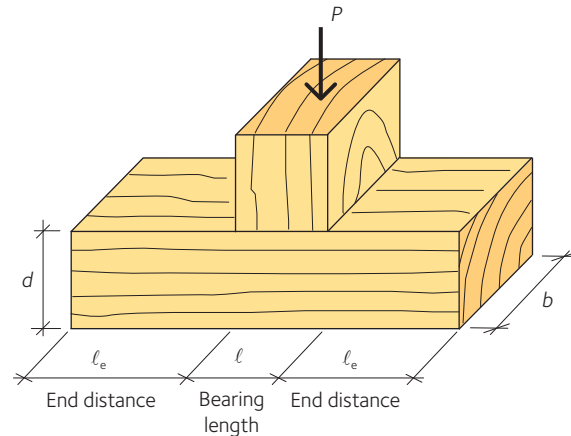


Figure 7.4: Typical layout of a bearing joint.

The shorter the bearing length, the less deformation is obtained for maintained stress level, which can be seen as an increase in stiffness with decreasing bearing length. Another factor that affects the behaviour is the influence of different ground plate depths, d . Directly under the surface of the loading plate (or stud) the stress will be equal to the value given by Equation 7.3. If it is assumed that the ground plate is resting on the whole bottom surface, the stress would be less at that boundary than at the top boundary, since the loaded area is larger there. This would mean that the deformation over the ground plate depth is not as large as a calculation based on Equations 7.2 and 7.3 would indicate. This can be considered in estimations of the deformation by introducing an effective compressed area $A_{\text{eff}} = \mu b l$ instead of $b l$ in Equation 7.3, with $\mu \geq 1,0$ for all cases where $l_e \neq 0$. There exists different formulations for μ , but in all cases they depend on l_e and on d . Tentatively a value on μ of 1,5 – 2 can be used for cases where the stud is not close to a plate end. In the latter case a lower value on μ should be used, Mårtensson (2003).

7.2.2 Long term and climate effects

Creep behaviour in timber and wood-related products, see for example Figure 7.5, is a function of several factors, and there exists complex models that can be used to calculate deformations. The higher the moisture content the larger is the creep, that is the increase in deflection with time.

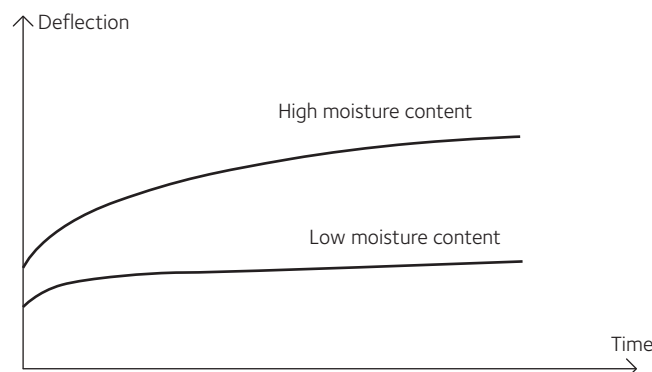


Figure 7.5: Schematic description of creep effects in a timber beam for different moisture contents.

In practice there is a problem also with varying humidity which also influences the behaviour, see Figure 7.6. 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 during the lifetime of the structure.

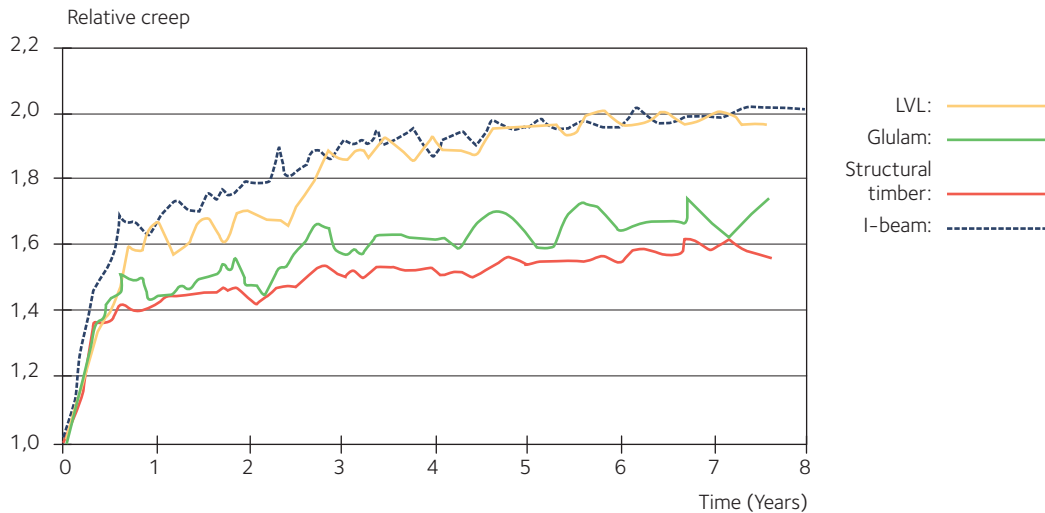


Figure 7.6: Relative creep in outdoors sheltered environment. (Relative creep = beam deflection at time t /initial deflection) Stress level 2 MPa. Ranta-Maunus, Korttesmaa (2000).

In addition to the effects of pure creep under constant load and creep under varying humidity, often called mechano-sorptive creep, there is the effect of varying load levels during the structures lifetime. 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 in terms of deformations and time effects as a beam subjected to bending, 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 normally negligible. The creep effects are also more severe than for a beam subjected to bending. Figure 7.7 shows examples of deformation in a joint with the layout shown in Figure 7.4.

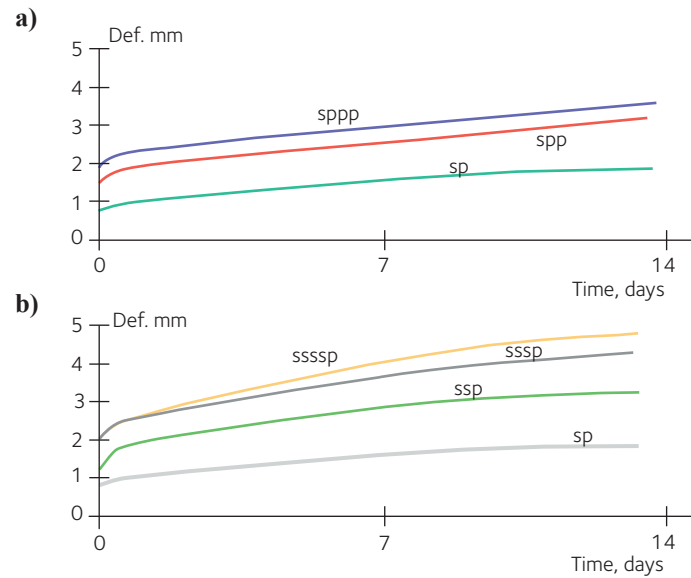


Figure 7.7: Deformation of a joint where timber is subjected to constant compression perpendicular to grain. sp = one stud and one plate, spp = one stud and two plates, sppp = one stud and three plates. Same stress level 2,6 MPa in all cases. The joint had been conditioned to 60 % RH before loading while it was exposed to RH = 30 % during the whole loading period, Mårtensson (2003).

The figure shows that there is a significant increase in deformation during the two week test period. The pure shrinkage (60 % RH to 30 % RH) was about 1/3 of the additional deformation increase after the initial deformation, that is the major part of the increase in deformation was due to creep. Figure 7.7 also shows that there is a non-linear increase in deformation with increasing depth d , as mentioned in Section 7.2.1.

The complex models of the behaviour of wood subjected to varying load as well as varying climate are not feasible for design in practice and instead more simplistic models can be used. In the following a simplified method to consider creep effects is described:

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

$$w_{creep} = k_{def} w_{inst} \quad \text{Eq. 7.4}$$

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

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

$$w_{fin} = w_{inst} + w_{creep} = w_{inst} (1 + k_{def}) \quad \text{Eq. 7.5}$$

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

- For permanent actions, G , where the load is constant in time:

$$w_{\text{fin},G} = w_{\text{inst},G} + w_{\text{creep},G} = w_{\text{inst},G}(1 + k_{\text{def}}) \quad \text{Eq. 7.6}$$

- For variable actions, Q_i :

$$w_{\text{fin},Q_i} = w_{\text{inst},Q_i} + w_{\text{creep},Q_i} = w_{\text{inst},Q_i}(1 + \psi_{2i}k_{\text{def}}) \quad \text{Eq. 7.7}$$

The value of k_{def} is decided based on climate conditions and type of material, see also Section 2.4.7. A number of experimental studies have been made in order to estimate values for this factor and examples of values can be found in handbooks and codes. Different service classes are usually defined in order to classify the environmental conditions that the structural elements are subjected to. The value of k_{def} will increase with increasing humidity in the surrounding climate. It is then important to remember that not only the maximum or average values of the relative humidity are of interest but also variations in humidity. If it is reasonable to assume that large temporal variations in humidity will occur, it could be an idea to use a more severe service class than if only the maximum humidity value is used.

Values of k_{def} are based on results from creep tests under constant load, and the uncertainties in the figures are relatively large. Since creep effects also depend on the time period that the load will be sustained, the factor ψ_2 is introduced in Equation 7.7 to describe this effect ($\psi_2 = 1$ for constant load in time). For a time variable load $Q(t)$ with characteristic value Q_k the reduced load $\psi_2 \cdot Q_k$ can be seen as the time average of the load $Q(t)$. In Eurocode, this is referred to as the quasi-permanent value, and values for the factor ψ_2 are specified for different types of variable loads.

In the case where a structure consists of elements/materials with different creep behaviour special considerations have to be taken. In these situations, the creep behaviour will affect the stiffness and stress distribution and a linear relationship between the instantaneous and the creep deflection described earlier will not apply even for low stress levels. This is described in Chapter 5.

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:

$$E_{\text{mean,fin}} = \frac{E_{\text{mean}}}{1 + k_{\text{def}}} \quad \text{Eq. 7.8}$$

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

7.2.3 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 defined in order to define relevant design situations. The load combinations for serviceability limit states defined in SS-EN 1990 are given below for the general case with several variable loads $Q_{k,i}$.

Characteristic combination:

$$\sum_{j \geq 1} G_{k,j} + Q_{k,1} + \sum_{i > 1} \psi_{0,i} Q_{k,i} \quad \text{Eq. 7.9}$$

where $\psi_{0,i} Q_{k,i}$ is the combination value of the variable load.

This combination gives a high value on the load and should normally be used in cases where excessive deformations may lead to irreversible effects or permanent damage.

Frequent combination:

$$\sum_{j \geq 1} G_{k,j} + \psi_{1,1} Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i} \quad \text{Eq. 7.10}$$

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, that is they occur with a certain frequency but the effects will be reduced when the load decreases again.

Quasi-permanent combination:

$$\sum_{j \geq 1} G_{k,j} + \sum_{i \geq 1} \psi_{2,i} Q_{k,i} \quad \text{Eq. 7.11}$$

The value $\psi_{2,i} Q_{k,i}$ 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 7.5 – 7.7 and 7.9 – 7.11 are adopted the final deformation w_{fin} under permanent and variable loading will be as follows, when the serviceability design is made on the basis of the characteristic load combination, Equation 7.9:

For permanent actions, G :

$$w_{\text{fin},G} = w_{\text{inst},G} + w_{\text{creep},G} = w_{\text{inst},G} (1 + k_{\text{def}}) \quad \text{Eq. 7.12}$$

For the leading variable action, Q_1 :

$$w_{\text{fin},Q_1} = w_{\text{inst},Q_1} + w_{\text{creep},Q_1} = w_{\text{inst},Q_1} (1 + \psi_{2,1} k_{\text{def}}) \quad \text{Eq. 7.13}$$

For accompanying variable actions, Q_i :

$$w_{\text{fin},Q_i} = w_{\text{inst},Q_i} + w_{\text{creep},Q_i} = w_{\text{inst},Q_i} (\psi_{0,i} + \psi_{2,i} k_{\text{def}}) \quad \text{Eq. 7.14}$$

The final result for permanent load and n variable actions will be

$$w_{\text{fin}} = w_{\text{fin},G} + w_{\text{fin},Q_1} + \sum_{i=2}^n w_{\text{fin},Q_i} \quad \text{Eq. 7.15}$$

It is important to notice 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 then 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 7.10 or the quasi-permanent load combination Equation 7.11. The formulas 7.13 and 7.14 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 :

$$w_{\text{fin},Q_1} = w_{\text{inst},Q_1} + w_{\text{creep},Q_1} = w_{\text{inst},Q_1} (\psi_{1,1} + \psi_{2,1} k_{\text{def}}) \quad \text{Eq. 7.16}$$

For accompanying variable actions, Q_i :

$$w_{\text{fin},Q_i} = w_{\text{inst},Q_i} + w_{\text{creep},Q_i} = w_{\text{inst},Q_i} (\psi_{2,i} + \psi_{2,i} k_{\text{def}}) \quad \text{Eq. 7.17}$$

7.2.4 Limitation of deformations

Any deflection limit which is set can be specified due to functional reasons or purely for visual reasons. For example, it has been found by experience that beam deflections not exceeding $\ell/300$ are usually acceptable. This is often used as a value when designing in the serviceability state. It is, however, important to consider why the control is done and on what type of structure and for which loads. If for instance the beam is cambered to off-set dead load deflection, the deflection limit applies to movement under imposed loading only. For beams spanning a short distance (say less than 2 m) it seems rather unnecessary to be too pedantic about observing this kind of deflection limit, Ozelton and Baird (2006), Porteous and Kermani (2007).

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

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

- the span
- the type of structure and the usage
- the risk 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 the 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 that 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 characteristic load combination in Equation 7.9 and Formulas 7.12 – 7.15. 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 for damages 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:

$$w_{fin,qp} = w_{inst,G} + w_{creep,G} + \sum_{i=1}^n w_{creep,Q_i} = \quad \text{Eq. 7.18}$$

$$w_{inst,G} (1 + k_{def}) + \sum_{i=1}^n \psi_{2,i} k_{def} w_{inst,Q_i}$$

- In some cases it can be more relevant to control the long term deflections only, that is the extra deflection that occurs with time, omitting the instantaneous parts in the control. This can be the case when the beam is cambered so that the instantaneous effect of the permanent load is taken care of. And when the control is made with respect to appearance and utility and not damage.
- For cases where it is of interest to control deformations connected to bearing stress 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, this means that 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 reasonable to use higher values on the factor k_{def} than 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.

7.2.5 Methods to avoid deformation problems

In practice it is impossible to avoid deformations when constructing with traditional building materials, as for example timber. But with better prediction tools it ought to be possible to avoid deformations of such a magnitude that they may 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 it is important to reduce the humidity variations. This means that one has to be observant on the climate conditions in the building

where the timber is to be used. One especially important aspect of the climate is if the timber structure is situated in a structure where the climate is different on different sides of the structure, as in the case with a climate separation structure. Different climate on the different sides may increase the problems with deformations.

For joints subjected to compression it is important to avoid or reduce compression perpendicular to grain since this is the most sensitive direction both with regard to load and to effects of moisture variations. In Figure 7.8 two examples of joints are presented where only a minor part of the timber is subjected to compression perpendicular to grain. There may also be problems if different type of materials is used in the building. If, for instance, a beam is situated on a concrete wall in one end and on a timber structure in the other, this often leads to inclination of the beam. In this case one may introduce a wooden piece between the concrete and the beam so that deformations of the same magnitude will occur in both ends.

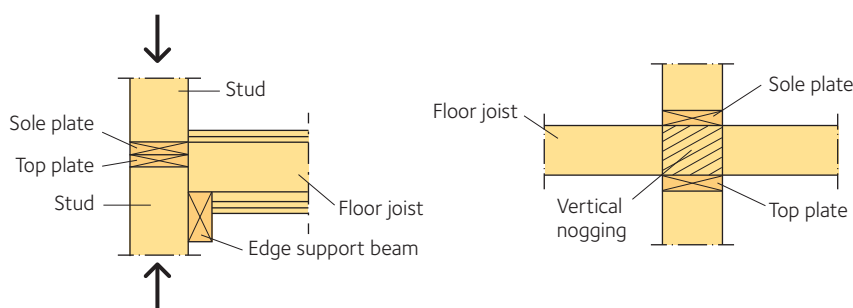


Figure 7.8: Examples of joints that can be used in order to reduce the problems with compression perpendicular to grain.

Another method to avoid the negative effect of moisture variations in timber structures and thereby decreasing the deformations is to use some sort of coating that makes it more difficult for the moisture to move into the timber. This is not practical for most cases since it is laborious and uneconomical.

It is also of major importance that the engineer really focuses on the question: what is the acceptable serviceability level? In many cases the designer uses very simple rules of thumbs for the limit values, which in some cases are adequate but in some cases are too crude.

7.3 Vibration

Serviceability requires consideration of the comfort of the user, and into this comes dynamic effects such as vibration when persons walk 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 providing lateral distribution of applied dynamic load and damping of the generated vibration.

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

- acceleration causes forces on the body that are felt by the balance organs
- visual cues (for example movement of objects resting on or hanging from the structure relative to the observer).
- audio cues (for example creaking or rattling created by motion of the structure).

Surveys for domestic buildings indicate that audio cues are very important to perceptions of how well floors perform, Smith (2003).

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 activities of others that cause 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 used (Smith, 2003):

- **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 foot-fall on a floor are characterised by perception of floor vibration induced by other persons than the one that is disturbed.

Springiness is a problem usually only 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 having 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. It is mainly the latter one that theoretical models can describe.

It is well established that humans are most sensitive to vibrations in the frequency range 4 to 8 Hz, which is the range of vibration for human internal organs, and thus researchers agree that structural frequencies in that range are undesirable. Human sensitivity to low-frequency accelerations is highly variable and depends also on situation – are they working, sitting down, standing up, resting etc.

7.3.1 Dynamic loads

7.3.1.1 Human induced 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)
- gymnasias and sports halls
- dance and concert halls.

There are 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 (for example walking, running, jumping), the number of people, whether activities of different people are coordinated, and the characteristics of the floor surface. Annoying vibration of timber floors is commonly associated with walking and running forces. Vertical forces due to an individual foot impact have been measured and characterised, Figure 7.9. As can be seen, there are two peaks in a force-time history with the first corresponding to heel strike contact and the second to toe-lift off contact. Flexibility of both footwear and floor coverings influences forces. The peak force is much higher, but the duration much shorter, for running than for walking. The response of short or medium span floors is a mixture of low-frequency forced vibration and high-frequency resonant vibration.

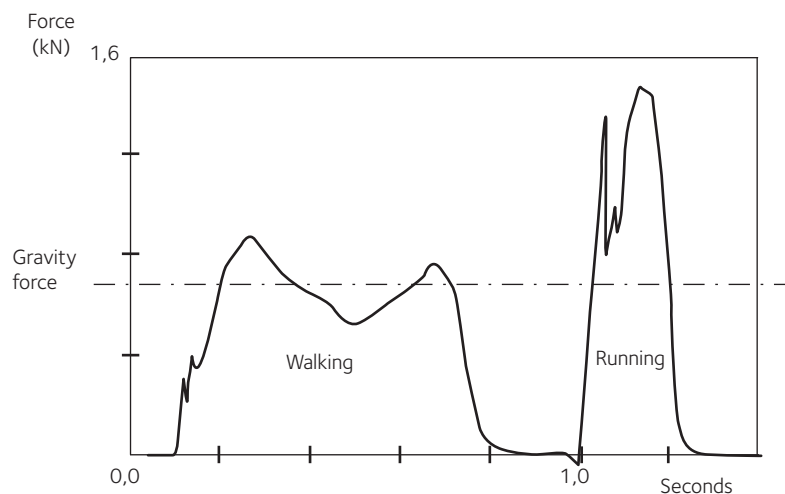


Figure 7.9: Footfall forces. (Smith 2003).

7.3.1.2 Mechanical equipment

Powered mechanical equipment usually exerts reciprocating or harmonic loads on floors (for example industrial machinery, washing machines, spin dryers). Depending upon the nature of the equipment and the orientation of the axis about which moving parts spin, induced motion can be horizontal, vertical or a mixture of the two. Ideally, mechanical equipment should be isolated from floors used by humans or premises where sensitive equipment is housed. If that cannot be done, attention should be directed to avoiding resonance. Except for some reciprocating machinery, resonance is usually automatically avoided because the excitation frequency is much higher than frequencies of the most energetic structural modes. Audible effects (noise) associated with operation of mechanical equipment heightens human perception of and sensitivity to motion, and should therefore be minimised.

7.3.2 Response to dynamic loads

From a structural standpoint, a timber 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. The situation for design against annoying floor vibrations is a far more complex issue. Performance is then influenced by stiffness, mass, damping and the so-called two-way action of the floor structure. In many cases, the static stiffness properties of timber floor systems are adequate to ensure satisfactory vibration performance.

In some instances however, floor systems designed merely 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 timber 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 timber floors.

There exist a number of different design methods with regard to vibration problems and a number of studies have been made to improve the methods, Hu et al. (2001); Brunskog et al. (2006). Significant progress has been made towards achieving a better understanding of the vibration parameters influencing human perception of vibrations, development of design approaches for control of floor vibration, modeling floor responses to static and dynamic loads, and understanding the influence of components and construction details on vibration performance.

7.3.2.1 Factors affecting human response to floor vibrations

Numerous efforts have been made to identify factors affecting human response to vibrations. Both laboratory studies and large-scale studies on floors in situ have been performed. In both cases subjective assessment by persons together with performance tests are the evaluation factors. In the context of residential construction, floor vibration serviceability relates to human response to vibration effects excited by human activities. The major cause of the excitation is the motion of the occupants on the floors. Essentially, this type of excitation consists of a train of impacts, each causing a transient vibration response characterized by its decay with time. Depending on the intervals between successive impacts and damping of the vibration, adjacent transient vibration responses may interact with each other. Ohlsson (1984) suggested that this interaction can be significant and should be accounted for in any design method. This differed from other researchers who considered response from a single footfall force. Research has shown that continuous excitation caused by walking action is critical for floor systems with fundamental natural frequencies lower than 9 Hz. This is due to resonance effects caused by walking motions in assembly occupancies. However, fundamental natural frequencies of timber floors are rarely below 9 Hz, so resonance rarely occurs in residential occupancies. Analysis of transient response to a single footstep force may therefore be adequate in evaluating vibration properties of timber floors.

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

7.3.2.2 Frequency components

Floors are usually divided into low-frequency floors and high-frequency floors. Low frequency floors have a fundamental frequency below about 8 Hz, and high-frequency floors have a fundamental frequency above about 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 of 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.

For a floor system excited by a footstep force, the transient vibration response is of two types, see Figure 7.10. The initial forced vibration due to the heel strike force component of the footstep lasts for typically 30 – 100 ms. This is followed by free vibration that decays with time. The free-vibration response contains the frequency components coinciding with the natural frequencies of the system. The frequency range of vibration modes excited depends on the duration of the footstep force. The shorter the duration, the broader the frequency range. Only the first few modes can be excited effectively in a typical timber floor by the footstep force from a normal walking person.

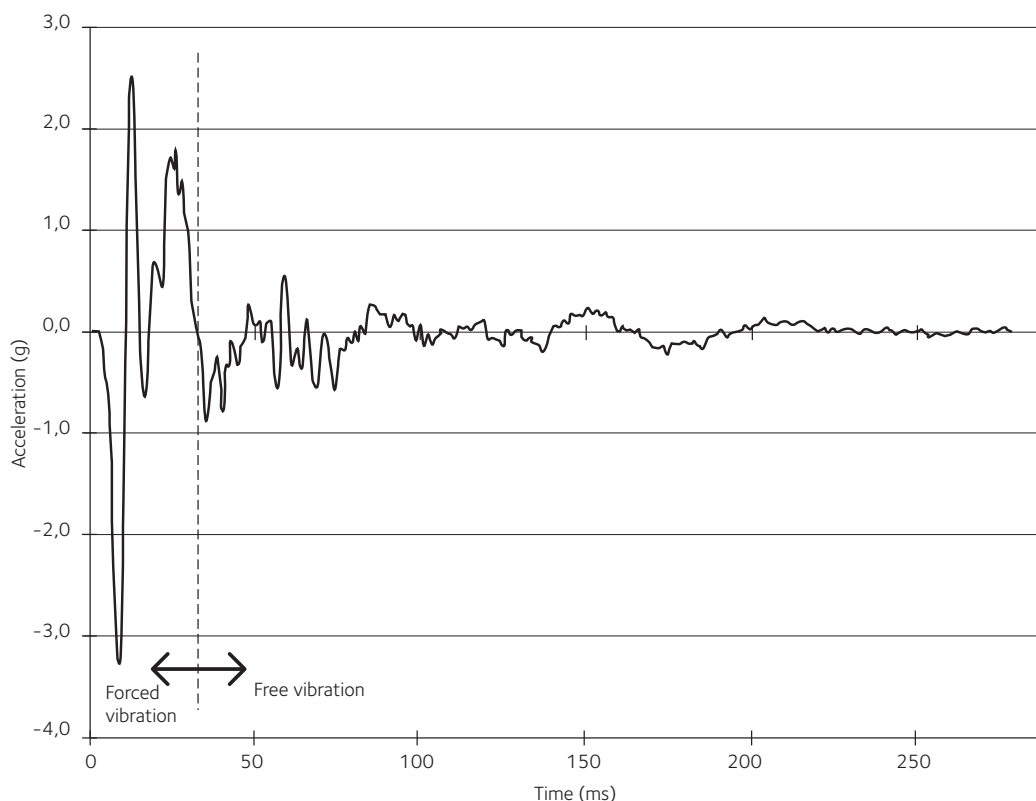


Figure 7.10: A time history response of timber floor caused by a footfall impact, depicting the forced and free vibration components; duration of the forced vibration section matches that of the impact duration.

Most researchers agree that, for a given level of response and damping, the lower the frequency, the longer the vibration will persist above the threshold of perception, that is the low frequencies are of major concern, Hu et al. (2001). However, Ohlsson (1984) has reported that, in addition to the fundamental natural frequency, the spacing between the first few adjacent natural frequencies can also contribute to human discomfort.

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

the cross-joist direction stiffness EI_y and EI_x . The spacing increases with increasing EI_y . EI_x is mainly governed by joist stiffness. Many construction details, such as floor decking thickness and material, attachment of floor decking to joists, joist spacing, and ceiling boards contribute to the effective EI_y .

7.3.2.3 Vibration magnitude

The transient response of floors to an impact can be represented by a time history of displacement, velocity or acceleration. Researchers have used quantities such as peak value of the response, or root-mean-square (rms) value of the entire response history, as a measure of human response to vibration. The peak value is extracted from the initial part of the response (forced vibration) due to an impact. However, the peak velocity used by Ohlsson (1991) is from a pure free vibration response excited by unit impulse. The rms value is determined from the entire response, including the initial forced and free vibration parts of the transient. Although both peak and rms values are related to the characteristics of the impact and the floor properties, the rms value includes the effect of the duration of the transient.

7.3.2.4 Damping

Damping is a property which influences vibration amplitudes under forced vibration and the rate of decay of vibration amplitudes under free vibration. Damping relates to the dissipation of energy, or to the conversion of the mechanical energy associated with a vibration to a form that is unavailable to the vibration. Increased damping results in rapid decay of a free vibration.

Many floor researchers have attempted to use damping to quantify the decay of the transient vibration induced by footstep impact, and hence correlate human response to damping. Despite these attempts, the problem of how to make use of damping information is largely unresolved because of the limited knowledge on quantification and measurements of damping in floor structures having a multiplicity of vibration modes. Damping values measured by researchers are global values arising from various sources. Material damping and friction between components are thought to be the major sources of damping found in structural systems.

In timber floors, material damping depends on the materials used for construction. The construction details such as the manner that components are attached, and the boundary conditions at supports, contribute to the frictional damping. The material damping is usually small. The major contributor to damping appears to be due to friction, a factor that has been found in for example cases where the presence of partitions on floors has provided exceedingly high damping.

End conditions of bending members can greatly affect damping. It has been shown that for timber beams with ideally simply supported ends, the damping ratio associated with the fundamental mode is about 1 percent, which corresponds approximately to the material damping. 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, timber floors often do not exhibit a significant amount of inertial damping. Exceptions can occur with large systems, especially if they have thick concrete topping or support large amounts of imposed mass.

7.3.3 Design approaches to limit vibrations in timber 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. Some examples are presented here.

7.3.3.1 Limiting point load deflection

It has been shown that one possible method to predict human response to floor vibration is to determine the static deflection under a point load. A common method to do 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 to model the effect of a step action. Here one method to do this is presented: 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:

$$a = \frac{P\ell^3}{48EI} \quad \text{Eq. 7.19}$$

where in this case $P = 1$ kN. If the most simple approach is used Equation 7.17 gives the value of the deflection with the flexural rigidity EI 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 in order to calculate the value of the deflection:

$$a = \kappa \frac{P\ell^3}{48EI} \quad \text{Eq. 7.20}$$

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

$$\kappa = \begin{cases} -4,7\beta^2 + 2,9\beta + 0,4 & \text{when } 0 \leq \beta < 0,3 \\ 0,8 + 0,2\beta & \text{when } 0,3 \leq \beta \leq 1,0 \end{cases} \quad \text{Eq. 7.21}$$

with:

$$\beta = \frac{(EI)_\ell}{(EI)_B} \left(\frac{s}{\ell} \right)^4 \quad \text{Eq. 7.22}$$

where $(EI)_\ell$ is the flexural rigidity of floor in the stiffer direction, that is along-joist (Nm²/m) and $(EI)_B$ is the flexural rigidity of floor in the direction perpendicular to the stiffer direction, that is across-joist (Nm²/m), s is the spacing between the joists and ℓ is the span of the joists, Ohlsson (1984).

Note that other methods exist to perform this calculation and that various limit values for a are also discussed.

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

The use of static response parameters such as deflection, while providing some control, 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 Ohlsson (1991). A laboratory study was conducted involving the use of dynamic testing equipment and limited subjective evaluation along with theoretical reasoning. Ohlsson concluded that 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 “unit impulse of 1Ns” $< 100\ell^{\zeta-1}$ (m/s)/Ns where f_1 is the fundamental natural frequency and ζ is the damping ratio for f_1 .

The first criterion is similar to that presented in 7.3.3.1 and 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. As discussed in previous sections, the impact excites higher frequency components in a timber floor (8 – 40 Hz) and its response is governed by stiffness, mass and damping of the floor. Believing that contributions to the total motion from vibration modes with natural frequencies higher than 40 Hz are insignificant, Ohlsson (1991) proposed the following equation for calculating the peak velocity due to unit impulse for a rectangular floor system simply supported on all four sides, valid for $f < 40$ Hz:

$$v = \frac{4(0,4 + 0,6n_{40})}{mB\ell + 200} \quad \text{Eq. 7.23}$$

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

$$n_{40} = \left[\left[\left(\frac{40}{f_1} \right)^2 - 1 \right] \left(\frac{B}{\ell} \right)^4 \left(\frac{(EI)_{\ell}}{(EI)_B} \right) \right]^{0,25} \quad \text{Eq. 7.24}$$

where B is the width of floor (m), ℓ the span (m); m the mass per unit area (kg/m^2), $(EI)_{\ell}$ is the flexural rigidity of floor in the stiffer direction, that is along-joist (Nm^2/m) and $(EI)_B$ the flexural rigidity of floor in the direction perpendicular to the stiffer direction, that is across-joist (Nm^2/m).

Ohlsson (1991) also provides an equation for calculating the fundamental natural frequency of a floor:

$$f_1 = \frac{\pi}{2\ell^2} \sqrt{\frac{(EI)_{\ell}}{m}} \quad \text{Eq. 7.25}$$

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

In the second criteria above the damping ratio ζ has to be determined which is a difficult matter. Ohlsson (1991) states that the value on ζ can be 1 percent but also states that higher values can be relevant.

7.3.3.3 Classification of floor vibrations

The methods described previously work well in some cases, but are less adequate in other cases. The design methods give usually only a limit value for floors and it is often unclear for the designers what the limit value actually means. How much better is the floor actually if the limit value is lowered with 50 percent? In order to get a better description of the actual behaviour of a floor structure, Talja and Toratti (2006) have introduced a method for classification of different floors based on the response to vibrational excitations. They performed subjective tests and measurements on timber-, steel- and concrete floors both in laboratory as well as in buildings just after construction. Their research has resulted in a method for the assessment of vibration serviceability of floors.

For high-frequency floors ($f_1 > 8$ Hz) they have found that point load deflection is the best indicator when designing with respect to floor vibration. For low-frequency floors ($f_1 \leq 8$ Hz) the recommended parameter to use is acceleration, because of its good correlation with subjective ratings.

Talja and Toratti (2006) propose a classification of floors into five classes, see Table 7.1. This classification presumes human walking to be the designing load case and it is valid for floors in residential and office buildings. It is based on the sensory perception of a sitting person and the human perception of vibrating objects.

Table 7.1 Vibration classes in office and residential buildings (Talja and Toratti 2006).

A	Special class for vibrations inside one apartment. Normal class for vibrations transferred from another apartment. The vibration is usually imperceptible.
B	Higher class for vibrations inside one apartment. Lower class for vibrations transferred from another apartment. The vibration may be perceptible but usually it is not annoying (inside one apartment).
C	Normal class for vibrations inside one apartment. The vibration is often perceptible and some people may feel it annoying (inside one apartment).
D	Lower class for vibrations inside one apartment. For example attics and holiday cottages. The vibration is perceptible and most people feel it annoying (inside one apartment).
E	Class without restrictions.

This method has the advantage that it gives a tool where the designer can be more aware of the actual floor performance and on the target performance to be achieved. Talja and Toratti (2006) also gives tentative limit values on deflections and frequencies for the different classes, see Table 7.2.

Table 7.2 Tentative acceptance limits for vibration classes (Talja and Toratti 2006).

Class	a_{rms} [m/s ²] Peak vertical velocity	δ [mm] Global displacement of load bearing member due to 1 kN point load
A	≤ 0,03	≤ 0,12
B	≤ 0,05	≤ 0,25
C	≤ 0,075	≤ 0,5
D	≤ 0,12	≤ 1,0
E	> 0,12	> 1,0

Other approaches to modelling and prediction of the dynamic response can be found in literature but for all of them it can be said that there are uncertainties and that it is often difficult to find any one 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 obtained by testing it.

7.3.4 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. The 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 that are used. Since there has been efforts made to decrease the amount of material and increase span lengths due to other aims in the design process, it is not surprising that reports of vibration problems in structures have increased lately. It is normally easier to increase the strength to weight ratio than to increase the stiffness to mass ratio of construction materials.

Other means of avoiding “problem” floors are obviously available. A potentially effective approach is to increase the damping (for example add artificial damping or 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 (see Figure 7.11) and to be observant of detailing and connections between different parts of the systems are also important factors.

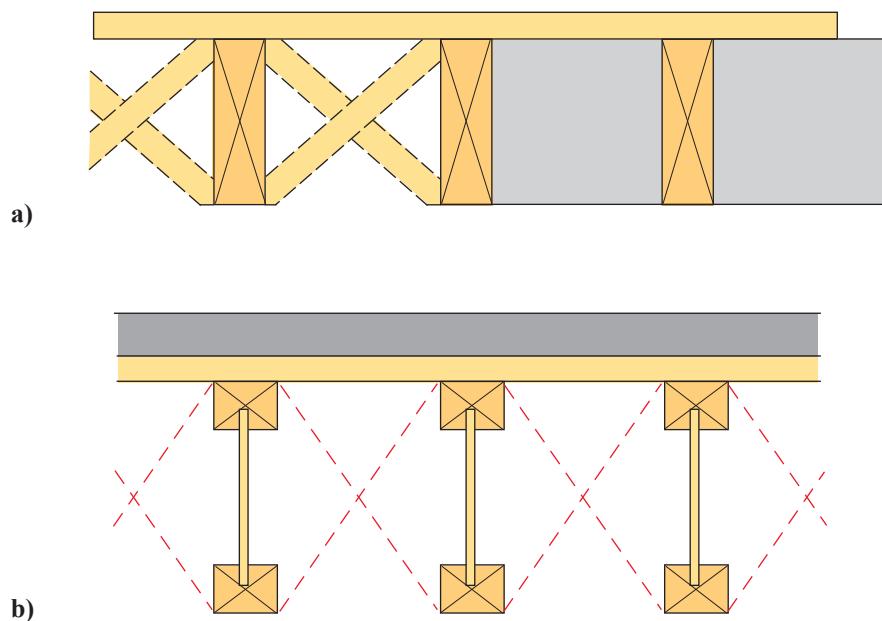


Figure 7.10: a) Plywood sub-flooring on timber joists. To the left with wood cross-bracing between joists and to the right with blocking between joists. b) Plywood sub-flooring with concrete topping on wood I-joists (metal cross-bracing between joists), Smith (2003).

A special type of floors is floating floors and raised floors, Talja and Toratti (2006). These types of superstructures are increasingly used because of impact sound insulation requirements and because of the flexibility of mounting the installations. The vibration or movements of objects, as clinking of glassware or leaf movement of plants, are typical for these type of floors and it has been shown that such effects are highly dependant on the flexural stiffness of the top surface board. Local deflections are caused by soft floating floors and this needs to be limited to avoid such disturbances. On the positive side, floating floors with sufficient bending stiffness of the top layer may effectively distribute point loads to various floor joists and thus increasing the floor vibration performance.

Local deflections are most often difficult to predict based on engineering calculations and it is recommended that these 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 observant to transmission of vibrations between different rooms via the floor joists. With continuous beams, vibration can be transferred from one apartment (or room) to another. Even if no problems are experienced in the room where the vibrations emanates the neighbors can experience nuisance from them. And it has been shown that vibrations coming from a neighboring apartment are more irritating than if the vibration source is in the same apartment.

7.4 References

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Chapter 8

Timber building systems for housing

Helena Lidelöw

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8 Timber building systems for housing

Timber buildings systems can be used to construct a multitude of buildings. Single-family homes have a relatively simple structure, while greater challenges meet the material when facing multi-storey or long-span buildings. Multi-storey timber construction in Sweden started around 1995 with the traditional on-site construction utilizing the 2" × 4" (45 × 95 mm) platform system. Relatively soon, prefabricated systems emerged. In the Nordic countries, prefabricated systems dominate, while systems with a larger degree of craftsmanship are used in central Europe. Timber building systems are common in the Nordic countries and the German speaking countries in central Europe; Germany, Switzerland and Austria and to some extent Italy. Also, timber building systems exist in the United Kingdom, while timber is less used in countries around the Mediterranean. Globally, USA use timber extensively for housing, the most common technology being a simple 2" × 4" frame stabilised by sheathing. New Zealand and Australia also apply timber structures in housing. LVL is a common framing material both in New Zealand, Australia and USA. Finally, Japan has a long tradition using timber in structures. Oftentimes, the great ductility in a timber frame ensures good racking resistance under earthquake loading.

Geometrically, building systems belong to one of the following categories:

- **Panel systems:** are based on planar building elements, for example walls or floors. These systems utilise either light frames or solid wood elements as their basic technology. The panels can be full size or partitioned in widths of 1 200 mm for ease of transportation. Floor elements are not suitable for long-span structures, the limitation in span length is about 8 – 10 m. Roof elements however, can have longer spans.
- **Modular systems** (volumetric modules): utilize the light frame system mainly, but examples exist where solid wood elements apply. The main principle is that the entire volumetric box consisting of walls, floor and ceiling, as well as inner claddings and all services are assembled in a factory and delivered to the building site for erection. Modular systems are not suitable for long-span structures, limitations on span is about 4 m due to transportation limitations.
- **Post and beam systems:** are sometimes named big frame structures and are very common for industrial and commercial buildings. These systems are also used for buildings or parts of buildings, where the load-carrying structure is visible. The post and beam system is a system based on a structural grid of beams and columns, typically with pinned connections. The structure is instead stabilised through diagonal bracing or shear walls. Post and beam systems are suitable for long-span structures and are used for arenas.

Building systems can also be classified according to their connection with the industry sector as:

- **Open systems:** the specifications and building elements can be combined and purchased by anyone. Suppliers manufacture their elements based on an open standard of dimensions, which all suppliers adhere to.
- **Closed systems:** the specifications and building elements are used by the supplier only who makes his profit from the efficiency in building with the system. Closed systems are suitable for integration of different functions in a company, for example design and production.

A supply chain is a group of companies, which produce a product by letting it pass through their production processes in sequence. The building system is transferred to the customer or the next party in the supply chain at the decoupling point. The earlier the decoupling point is placed in the supply chain, the more open the building system. In Sweden there are currently no real open building systems for modular systems or cross-laminated timber (X-Lam).

There is, however, an agreement on dimensions prescribed in SS-EN 8560 and most actors in the building element industry follow the standard and produce elements in sizes of 900 mm, 1 200 mm or 2 400 mm (corresponding to 9M, 12M and 24M). The dimensional standard also simplifies both design and prefabrication, since main dimensions are already negotiated and only special cases need to be addressed.

SS-EN 8560 is not used for arenas and other long-span structures. In Sweden *Limträhandbok Del 1–3* (2016) exists, presenting solutions and details for designing glulam structures. With the help of the handbook, a glulam structure can be realised regardless of supplier input and glulam beam-and-post systems may therefore be considered open systems. In Finland and Denmark the development of open building systems has led to a component supplier market where several suppliers compete with similar products. The *Open Timber Construction System* (2003) was published in Finland to provide prerequisites for an open market with timber elements and it describes detailed solutions for element lay-up and connections for any supplier to use, produce, and sell. A strong governmental body is needed to provide basis for open building systems, if the market is left to regulate itself, closed building systems often result.

Technical design of the frame is always the core part of the contract with the supplier of a building system, which also can include the erection of the structure. Thus, the delivery includes much more than a building material and the supplier has to create an organisation for this, in-house or through partnerships. Integrating large parts of the supply chain in one company is beneficial since the possibility to optimise processes becomes larger if several processes are controlled. The risk with integration is financial since control often leads to investments in automation, which in turn builds up a factory that needs to be fed with a continuous flow of products to produce. The benefits seem to be larger since the majority of building systems are closed or proprietary systems. This is true for most of the building systems in Sweden and the majority of timber building systems in Germany, Switzerland, and Austria. Patent rights for new solutions is important in closed building systems.

8.1 Structural systems overview

8.1.1 Plane elements

8.1.1.1 Light frame system

The light frame building system using joists and studs placed at even distances with the voids filled with insulation and covered by sheathing to stabilise the element was born in USA in the 1830's. The rapid expansion in USA was created by a large demand for housing and combined with machine sawing and industrially produced nails, the light timber frame became

a success that has spread all around the world and is still used extensively. The platform framing technique is still the most common in USA when constructing multi-family houses, see Figure 8.1.

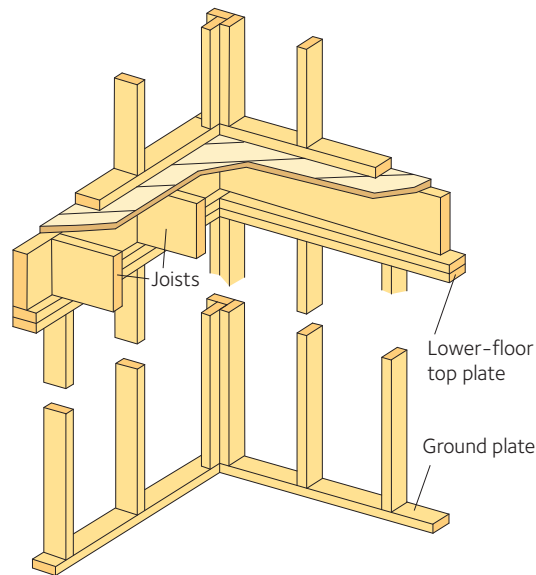


Figure 8.1: Platform framing, McGraw-Hill (2011).

In USA, light frames are commonly produced on site, but in the Nordic countries they are prefabricated in a factory. The light frame technology is the backbone of single-family housing production in the Nordic countries, where the approach to order a stock house, have it produced and erected on site by the same company (a closed building system) has been the norm ever since the 1950's, see Figure 8.2.



Figure 8.2: The load-bearing structure of a detached single-family house.

With the introduction of glulam and LVL, the timber joists in a light frame can be replaced with a stronger material, which enables longer spans. Being so light, the frame is prone to vibrate and deflect, which limits floor spans to 8 – 10 m. Another common problem with light timber frame is their low mass, which can lead to poor acoustic performance, see further Section 8.2. The height limit using light frame technology is currently 6 – 7 storeys. The compression strength perpendicular to the grain for the ground plate limits higher buildings, unless the structure is strengthened by special methods.

8.1.1.2 Solid wood systems

Solid wood systems are usually based on planar cross-laminated (X-Lam – cross-laminated timber) elements, see Figure 8.3, see also Section 2.7.1.2. The cross-lamination is established through gluing planed and strength graded timber together, much in the same manner as when producing glulam. It is possible to use small dimension timber in the production of solid wood panels, which makes them attractive for sawmills. The prefabricated elements are delivered to the building site, where the assembly can be done by the manufacturer themselves or another contractor, see Figure 8.3.

Solid wood elements, such as cross-laminated timber, are most common in Germany, Austria and Switzerland where they are also used in single-family houses to obtain specific architectural expressions. In countries with warmer climates, the wood element can be applied without extra thermal insulation. This is not likely in the Nordic countries, where the X-Lam frame often is completed with a light frame with insulation in the voids.

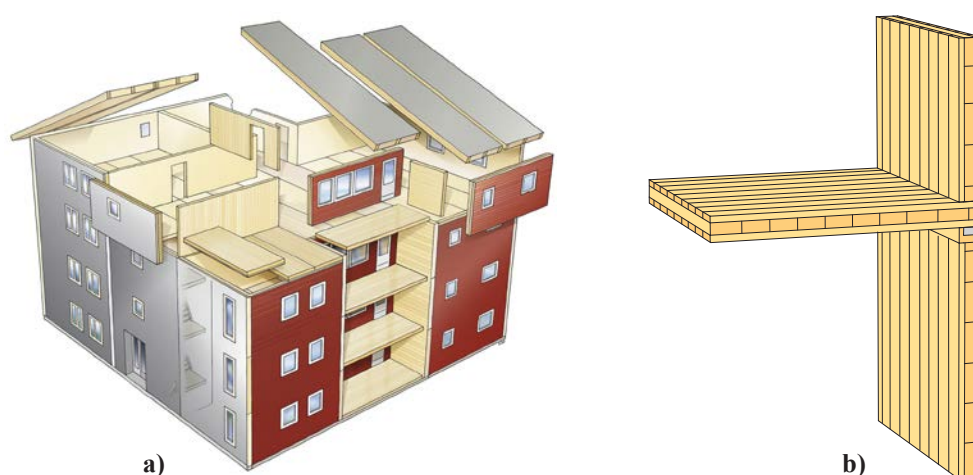


Figure 8.3: a) Structural system with cross-laminated elements. b) X-Lam, *Massivträhdboken* (2006).

The solid wood elements can also be established with planks joined mechanically with aluminium nails instead of gluing. Moreover, the planks can have slots routed on one surface, so the finished wall elements have air enclosed inside them, which improves their insulating properties and no additional thermal insulation is required.

The production of dwellings built in Sweden with X-Lam systems is approximately 150 units per year, with variations depending on the production mix with other types of buildings. The maximum height performed in Sweden is at present around 8 storeys, but development towards both 10 and 20 storeys is underway.

8.1.1.3 Engineered Wood Products

Engineered Wood Products (EWP) are mainly used as a substitute for sawn lumber in light frame timber buildings. There are many different types of EWP, see Section 2.7, but the main types used in building systems are I-joists with web of wood based panel, trusses, and LVL with parallel or cross-wise glued veneer layers.

An example from Sweden is Masonite Flexible Building System with I-joists built up by an OSB web and flanges of structural timber, see Figure 8.4 a) where joints are strengthened with thick (20 – 40 mm) wood based panel. Also I-joists built up by a hardboard web and flanges of LVL are present on the Swedish market. The intention is to use the elements for industrialised timber construction with a high degree of prefabrication, but at the same time this type of building system is relatively open and offers the possibility to be applied along with other types of construction methods, such as building on-site, panel elements or beam and post structures.

Trusses of structural timber are effective as floor joists, see Figure 8.4 b). The voids created by the truss are suitable for running services. The truss might be too high for many applications, and the many joints may infer large flexibility in the truss, leading to excessive deflections for larger spans. The design must be performed carefully to balance the benefits and drawbacks of a truss joist.

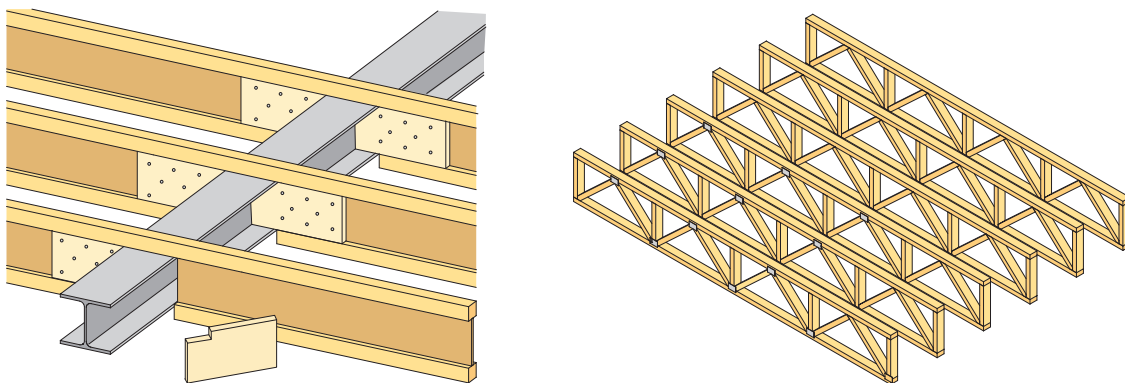


Figure 8.4: a) I-joists with web of wood based panel. b) Floor truss joists.

To address the problem with vibrations and poor acoustic performance in light-weight floors, they can be completed with a concrete topping, creating a composite floor element, see Figure 8.5. Also concrete topping without interaction or fillings with for instance gravel exist.

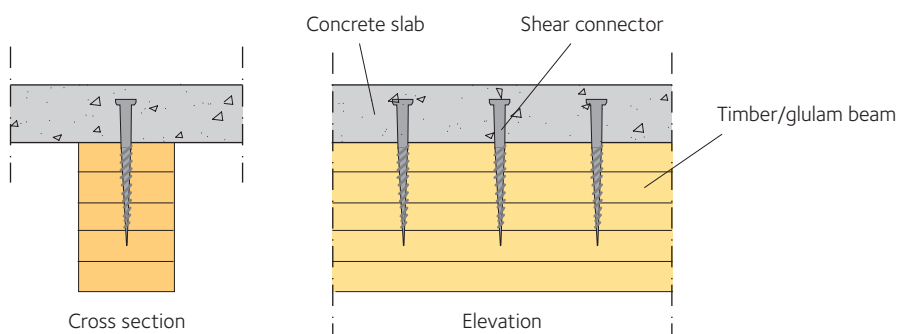


Figure 8.5: Composite timber-concrete floor, Lukaszewska (2009).

Composite timber-concrete floors are successfully used in Austria and Switzerland with several variations, see for example Natterer et al. (1996), and several designs and types of fasteners have been tested to reach desired performance, see Section 8.3.2.

8.1.2 Modular systems

8.1.2.1 Light frame modular systems

An extension of the light frame building system is combining panel elements into volumetric modules at a factory off-site, see Figure 8.6. Walls, floors and ceilings are produced using a light frame structure, using the same methods as when construction a single-family house. The floors and ceilings are hung onto the walls constructing a volume. Apart from the structural system, also claddings, services, and furnishing (for example kitchen furniture and equipment) are mounted. Thereafter, the modules are covered to be weather protected and transported to the building-site for assembly.

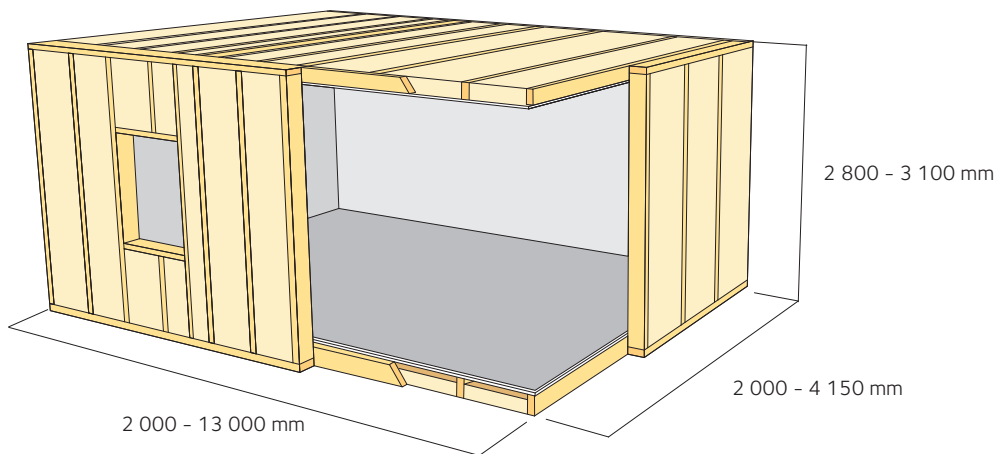


Figure 8.6: Volumetric element or module made with a light frame.

To assemble the modules on top of each other, a helping device is used in the form of a little dub (male-female connection). Between the modules, polyurethane tape or Xylodyne is used for acoustic purposes. Since this is a very expensive product, it is applied only on top of the studs, not continuously along the rim. The floors hang from the walls in modular systems since the outer walls of the module are used as vertical load-carrying components.

Utilising this kind of system gives the opportunity to control the whole building process from architectural and structural design to completion. The architectural design of the buildings built with modules needs to be carried out with knowledge of the building system and the limitations and possibilities in the production system. Modular systems are suitable for student housing, hotels and housing for the elderly, but is also adaptable to condominiums or tenancies. Each module can comprise one or a few rooms, or even the whole apartment in the case of student apartments. The size of the modules is determined by manufacturing, handling and transportation limitations and is regularly maximum 4,15 m in width, up to 13 m in length and 3,10 m in height. The limit in height is the same as for light frame systems, 6 – 7 storeys.

8.1.2.2 X-Lam modular systems

This type of system is based on modules of X-Lam (or semi-solid with air cavities). The modules are entirely built and equipped in the factory and transported to the building site, see Figure 8.7. Using X-Lam instead of light frame systems opens up the possibility for stiffer stabilising walls.



Figure 8.7: Assembly of modular element, Lindbäcks (2013).

8.1.3 Post and beam systems

Post and beam structures have always been very common for timber structures. Through larger cross-sections of members and consequently larger span lengths, the post and beam systems experienced a renaissance with the development of engineered wood products such as glued laminated timber and laminated veneer lumber.

8.1.3.1 Short-span structures

The post and beam structures can have various structural design and many types of buildings fall into this category, see for example Figure 8.8 a), based on a post and beam timber structure with composite timber-concrete floors, also used in the first timber parking house in Sweden, see Figure 8.8 b).



a)



b)

Figure 8.8: a) Kv Uppfinnaren in Växjö. b) Parking house Ekorren in Skellefteå, Martinsons (2010).

A post and beam system for multi-storey timber buildings named “Trä8” in timber was developed in Sweden by the glulam manufacturer Moelven Töreboda, see Figure 8.9 a), and is built up by glulam and Kerto (LVL). The system is based on rectangular division in plane, with a maximum span length of 8 metres, Moelven Töreboda. This gives at present a possibility to use the system for both residential and non-residential structures such as offices. The company has an intention to increase the span length in the future, but this limitation is related to the floor element height, which can be substantial for timber structures.



Figure 8.9: a) The post and beam system “Trä8”, Askims torg (Moelven Töreboda). Photo: Sören Håkanlind.

8.1.3.2 Long-span structures

For long-span structures, glulam presents a very economical alternative. The main competitor is steel structures. The maximum span length is often set to be 80 – 100 m, but much longer built examples exist. The most economical structures built are however ice hockey arenas with a span of 36 m. A recent trend in Sweden has been to also build arenas for bandy, which need to have a much longer span, roughly 70 m. Examples of structures for long-span structures are shown in Figure 8.10.

Such long-span structures are not used for housing, which means that most of the functional requirements in place for housing are not viable for this type of structures. Acoustics is seldom a problem; moisture safety and thermal insulation are currently poorly addressed. Load-carrying

capacity and fire are the most prominent requirements to address for the structure. To reach an economical structure, optimisation of the load-carrying capacity is therefore a must, which includes efficient design of joints, see Chapter 4. Compared to short-span structures intended for housing and offices, long-span structures are relatively easy to optimise.

Apart from the main load-carrying structure (the primary system) a secondary structure of beams might be needed between the primary beams. The secondary structure is made of beams out of glulam, structural timber or LVL and the purpose of it is to support the sheathing covering the building, see Figure 8.11. Optimisation of the building system must embrace both the primary and the secondary structure. Since the main functional requirement is load-carrying capacity, the main indicator for efficiency is the volume of glulam spent. Roughly 30 percent of the glulam volume is spent on the secondary structure.



b)



c)

**Figure 8.9: b) Hockey hall, Ekvallen (Martinsons). Photo: Gösta Wendelius.
c) Bandy hall, Nässjö (Moelven Töreboda). Photo: Sören Håkanlind.**

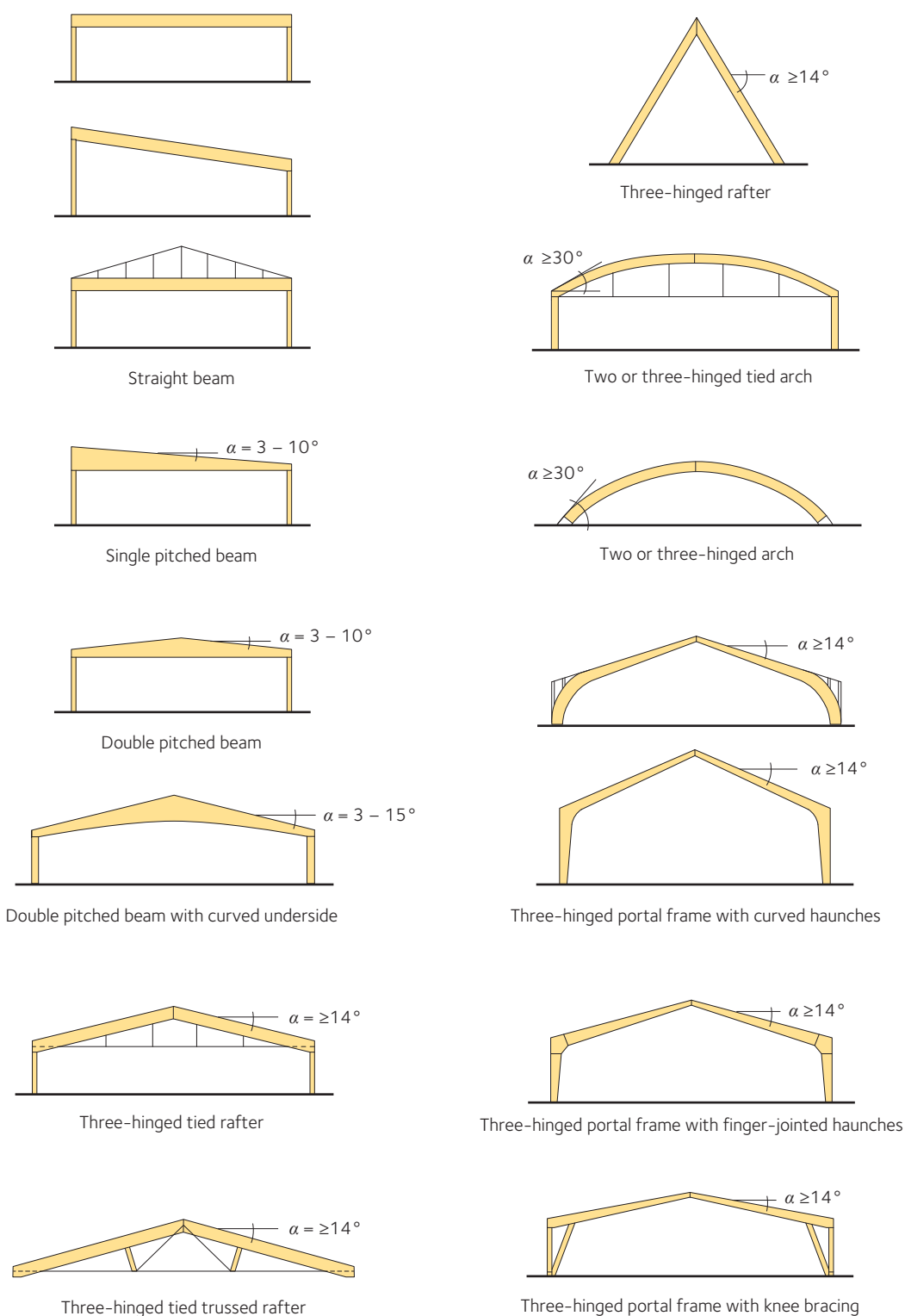


Figure 8.10: Structural systems in glulam, Limträhandbok (2016).

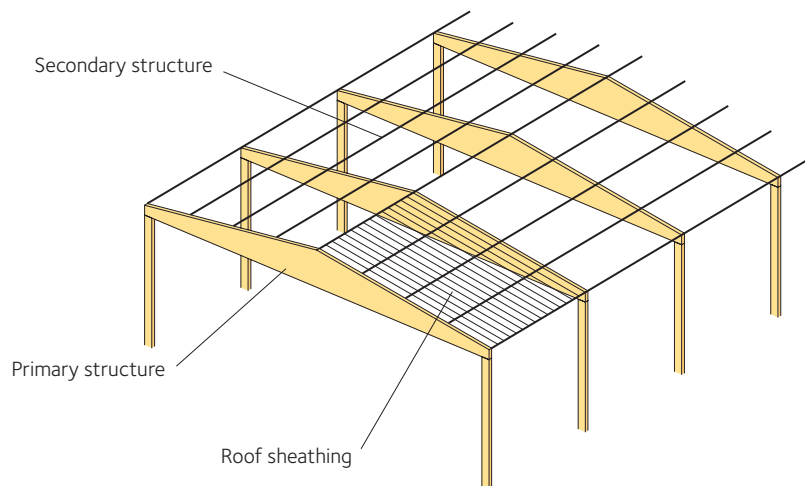


Figure 8.11: Primary, secondary structure, and roof sheathing.

8.2 Functional requirements and system solutions

All structural solutions for building structures need to fulfil the functional requirements for its users. For timber structures the most difficult and economically decisive ones are:

- fire
- acoustics
- horizontal stability
- critical moisture level.

8.2.1 Fire

The European codes for fire safety in timber buildings have recently been revised and new handbooks published, *Fire Safety in Timber Buildings* (2010). Buildings should limit the risk related to fire, limit the spread of fire and fire gases inside the building, and allow the inhabitants to escape from the building or to be evacuated or saved. The safety of firemen is also taken into consideration.

The building fire documentation that is mandatory for all buildings in Sweden should include: fire safety classes of used materials and elements, division into fire cells, evacuation plan, ventilation during fire, and a description of used technical solutions, including any additional installation like sprinklers or fire detection systems. It is recommended to integrate fire safety design in the very early stages of building design.

Structural elements, materials, claddings and surface finishes, depending on their function are assigned one of the following properties, SS-EN 1995-1-2:

- R – load-bearing capacity
- RE – load-bearing capacity and integrity
- REI – load-bearing capacity, integrity and insulation
- E – integrity
- EI – integrity and insulation.

Requirements related to these attributes are specified in terms of time in minutes, under which a specific function shall be preserved from the start of a fully developed fire. As an example

for buildings up to four storeys high, the fire safety class needs to be REI 60, while for higher buildings the requirement is REI 90, where the number indicates the time in minutes. It should be noted that the resistance time is related to an internationally standardised fire with specified temperature as a function of time. The actual fire resistance time in an actual fire in a building may be different from that observed in relation to the standardised fire.

8.2.1.1 Timber

The wood material itself burns and is destroyed in a fire. However, it burns in a controlled manner and quite slowly. A protective layer of coal is created on the surface when a wooden piece is subjected to fire. If the wooden piece is thick, the core of the wood will remain intact for a good deal of time. The charring rate is of the order 0,7 mm/min. In the case of large cross sections (for example glulam), the slow charring rate makes it possible to evacuate the building before collapse. The glue used for glulam can sustain fire well. Design for fire when looking at large cross sections (as those used in long-span structures) is made by first calculating a design load using the load combination for fire in SS-EN 1990. Then, the reduction of the cross section is calculated corresponding to the required fire resistance time (for example R 60), using the design charring rate specified in SS-EN 1995-1-2. If the structure can sustain the load with the remaining cross section, the fire resistance is sufficient.

When considering light-weight structures, the cross section of wood is much smaller (typically 45 mm on the short side). This dimension is too small to sustain a fire during for example 60 minutes and therefore the structure needs to be completed with some type of cladding to increase the fire resistance.

8.2.1.2 Sheathing

In light frame timber walls, the fire resistance depends in decreasing importance upon the following parameters, Fire Safety in Timber Buildings (2010):

- the sheathing firstly exposed to fire
- the sheathing secondly exposed to fire
- cavity insulation protecting the sides of the timber member
- type of insulation in cavity, stone wool is generally better than glass wool.

A design model is presented in Fire Safety in Timber Buildings (2010), where the idea is to calculate the remaining cross section after exposure to fire and make sure that this remaining cross section can sustain the imposed load. The resistance time for the sheathing is deducted first and the fire exposure is assumed to start once all boards have “failed”. The method allows for calculation of the fire resistance of newly developed structures, without the need for performing fire tests.

The most common measure to fulfil the fire safety requirements is to apply sheathing of gypsum boards. Some Nordic manufacturers produce special fire resisting gypsum boards that have a fire resistance of approximately 40 minutes. Combined with one layer of common gypsum board they create a fire safety assuring class R 60. It is important that the edge joints of boards in both layers do not coincide. Boards of wood-based materials usually have a low fire resistance, for example plywood and OSB have a fire resistance shorter than 10 minutes. Note that the boards often play several roles, not only do they function as fire protection, they also stabilise the building and provide acoustic insulation.

8.2.1.3 Sprinklers

Sprinklers are a type of active fire protection. They are used to a varying extent in different countries; some prescribe them in buildings higher than for example 4 storeys while some do not. There are basically two types of sprinkler systems: industrial and residential. Residential sprinklers are an important novelty since most fires take place in residential buildings.

The residential sprinklers are quite easy to assemble in the building and using them gives a basis for risk assessment that may lead to a technical exchange in fire safety design, see Östman et al. (2002). Some examples of successful applications of technical exchange when using sprinkler systems are:

- combustible facade in more than 2 storeys may be used
- lower requirements on protection against fire spread through windows in the same building
- reduced demand on the outer facade layer in residential buildings
- increased distance to evacuation path is allowed.

One of the well-known weaknesses of timber structures is their sensitivity to moisture (valid also for insulation materials of mineral wool). The sprinkler system could be a cause of damage in case of a false fire alarm and subsequent activation of the sprinkler system. In case of fire, residential sprinklers bring comparatively the same amount of water inside the building as in the case of a manual extinguishment. The industrial sprinklers on the other hand, supply much larger amounts of water and the water damage is more severe. In case of an actual fire it is difficult to verify what damage, caused by water or by fire is worse. It is claimed that application of residential sprinklers saves lives; however, still they are optional for 3 – 4 storey buildings in the Northern European countries. Residential sprinklers are connected to the ordinary cold water installations as they operate on much smaller amounts of water than industrial sprinklers. The heads are installed in rooms where the fire statistically often starts.

8.2.1.4 Material modifications

Engineered wood products offer the possibility of material modifications. By adding different chemicals or using specific adhesives increased fire resistance can be achieved through impregnation. This option is not so commonly used due to high cost and uncertainties of the long-term effect of the treatment.

8.2.2 Acoustics

Acoustic performance for sound insulation and noise control is regulated in SS 25267 and SS 25268. These standards classify building parts in four different classes:

- Class A: A very good acoustic environment
- Class B: A significantly better acoustic environment than class C
- Class C: Gives satisfying acoustic environment for the majority of the inhabitants and represents the minimum requirements of the European code
- Class D: Is used when class C cannot be achieved, for instance for classifying older buildings.

Class B is the desired minimum in many cases (especially for residential buildings) and producers of newly developed systems need to meet the requirements for achieving sound insulation class B. Timber structures are treated as light due to their relatively low mass in comparison to concrete and steel structures. An overview of the state of the art regarding

acoustics in timber frame buildings is given in Forssén et al. (2008) and a handbook has been published, Ljunggren (2011). The solid wood technique (for example X-Lam) is definitely heavier than light frame systems, but the acoustic performance of these systems is still fairly poor. For solid wood floors and light-weight structures, the major problems are the flanking transmission and the impact sound insulation at low frequencies (50 – 125 Hz). Currently, well performing solutions however exist to solve both problems. Flanking transmission is solved by using special vibration insulation between building parts. Examples are Sylomere, Xylomer, Xylodyn or Stepisol, which are product brands for a damping material with specified stiffness. Impact sound insulation is dependent on the mass and the stiffness of the floor. Wooden floors must be completed with heavier sheathing materials such as gypsum board in several layers to increase mass and increase energy absorption. A very common solution is also to create two separate floor sub-systems, that is a load-carrying floor slab with a suspended ceiling underneath. These two units are separate and thus sound insulation is improved, see for example Figures 8.20 and 8.22. It is important that the fastening of the suspended ceiling is very flexible, to limit transfer of vibrations from the floor to the ceiling. It is also convenient to place ducts in the space between the floor and the ceiling.

Building systems with volumetric modules have the double layer solutions of floor and ceiling integrated in the system itself. It is seldom possible to make acoustic design calculations for floor solutions in advance, new solutions need to be verified through laboratory tests and sometimes also through measurements on site. An international project, Silent Timber Build, is however in progress (2016), with the aim to present calculation tools that will be able to better predict the acoustic performance.

In the case of solid plane floor elements it is common to apply a floating floor above the load bearing structure to suppress the flanking transmission. The surface weight of this floating floor is enhanced by a layer of heavy material, for example sand or concrete. It has been shown that the airborne sound transmission as well as the impact sound can be sufficiently decreased using this solution. Another possibility is to combine a solid wood floor with a suspended ceiling and walls made from frames clad with plasterboards, as plasterboard structures are in general much less prone to flanking transmission than solid wood, such as a X-Lam structure. Further recommendations regarding reduction of the flanking transmission can be found from Martinsons and in *Massivträhandboken* (2006). Solutions present on the Swedish market that meet the requirements for the acoustic class B are between 300 mm (with floating floor of concrete) up to 600 mm thick for span lengths up to about 8 m. For larger span lengths, up to about 12 m the thickness of the floor structure can reach 700 mm.

8.2.3 Horizontal stability

Since timber structures are categorized as light structures, one of the key structural issues is the stabilisation against horizontal loads. In the Nordic region, wind load is the main cause for horizontal loads, while several areas in the world need to address earthquakes as the main source of horizontal loading, see Tavoussi et al. (2008). Movements and deformations caused by these loads can be highly inconvenient or even dangerous for the inhabitants. The problem becomes more severe with an increasing number of floors. Conventionally, in low-rise buildings, the following two main approaches are utilised:

- diagonal bracing
- shear wall action.

Residential low-rise buildings are often stabilised through shear wall action, where the sheathing (wood based panels or gypsum board) is nailed or screwed to the timber frame in the walls to assure sufficient resistance against horizontal loads, see Gyproc. Often, this method is the most effective and economically advantageous, since it assures relatively ductile performance and no expensive materials or connectors are used. In commercial buildings, where the number of windows are generally less and the aesthetic issues can be of less importance, diagonal bracing can be utilised, either being made of timber members or steel chords. In some cases, albeit unusual, stiffening in one direction can be realised through moment-resisting connections, see also Chapter 6.

When dealing with taller structures it is necessary to assure a good inhabitant comfort also in the higher storeys against possible horizontal deformations in the structural system. For these structures the above methods may not be sufficient. One possible solution is to connect the building, along its full height from foundation to roof with a steel rod anchored to the ground.

The shear forces in tall buildings are transferred from the roof, walls and floor diaphragms into the foundation via a system of connections. The transfer of loads can require special solutions; for example, cross-laminated timber panels (X-Lam) can be used instead of the more common to timber frame nailed or screwed sheathing boards, to limit excessive horizontal deformation. The difficulty is that solutions favourable for the stiffness and strength of the building may be in conflict with the acoustic performance, since increased stiffness is negative for the sound insulation. Design of stabilising systems using boards as sheathing is described in Chapter 6 and EN 1995-1-1. Important factors to ensure good horizontal stabilisation are, see Figure 8.12:

- The sheathing board should be fastened to the timber frame along all edges. The sheathing board cannot be allowed to end before reaching the top or bottom plate of the frame.
- The number of and distance between fasteners is decisive for the horizontal resistance
- Wall sections with openings cannot be part of the stabilising system, unless plastic calculation is applied, see Girhammar and Källsner (2010).
- The leading and trailing stud must be sufficiently connected to the foundation (which could consist of a floor, a wall or the foundation) to avoid uplift.
- It is advisable to spread the stabilisation forces to as many walls as possible to avoid heavy up-lift forces. A method for calculation the load on individual walls in a floor plan is presented in Gyproc.

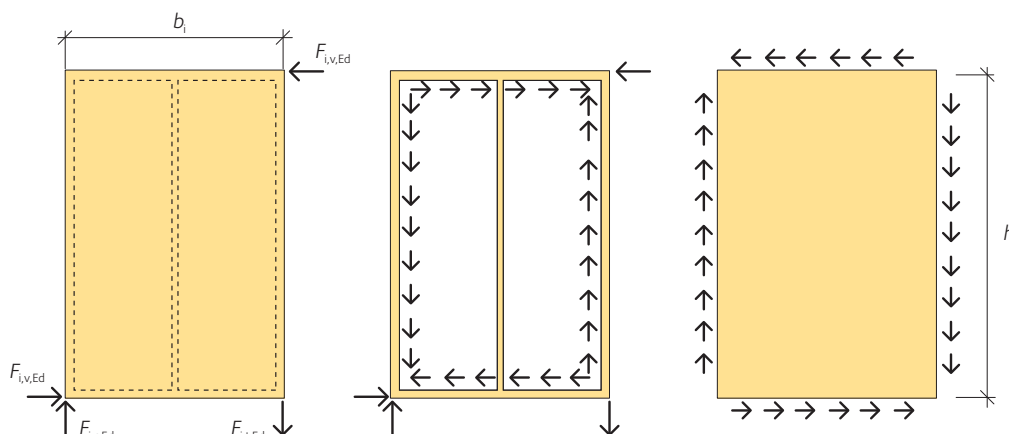


Figure 8.12:
Force flow in
stabilising wall
section.

Another alternative can be to use the staircase or elevator shafts, often constructed in concrete, as the stabilising mechanism for the whole building. This solution is common for medium-rise buildings in many European countries. In Sweden this solution is often avoided, since it is argued that the presence of a concrete shaft inside a timber building may make it visually, economically and also environmentally unattractive. Also, such a solution introduces problems due to different settlements of timber and concrete, which implies a need to adjust the elevator stops over the years in service. It is though a feasible solution structurally, which has been shown by Nordlund (2010).

In systems based on modules, the basic shear wall action needs to be utilised, as modules are mainly manufactured as light frame systems built up by sheathing boards on a timber frame. The main problem for this system is that the joints between the modules should be robust enough and at the same time accessible during assembly, to provide satisfactory transfer of shear forces. In addition, a reliable anchorage to the foundation must be provided to prevent the uplift of light structures, see Girhammar and Källsner (2010) and Vessby (2008).

8.2.4 Critical moisture level

One of the important issues related to using wood as a structural material is its sensitivity to moisture and, generally speaking, weather exposure, especially during the assembly phase. The Swedish code requires that the critical moisture level (relative moisture 75 %) shall not be exceeded during transportation, erection and usage. It is important to emphasise that not only timber is sensitive to moisture, but also other materials used in construction such as gypsum boards or insulation materials. In the light of these requirements, the timber construction companies often use some sort of weather protection during assembly, especially in cases when assembly is not completed within one day, see for example Figure 8.13.

For smaller houses the weather protection issue leads to an increased degree of prefabrication. Buildings (for example single family) can be entirely manufactured off-site and then transported to the destination and the assembly can be completed in $\frac{1}{2}$ – 1 full working day. In this case, additional conditions for a successful moisture protection of the structure are a dry concrete plate and protection of the unassembled elements.



Figure 8.13: Roof used as weather protection during erection of the structure.

However, this method does not apply for non-residential buildings or multi-family houses. For these buildings the assembly time as well as the exposure to weather is longer, so the principal protection against weather should be an integral part of the building system design and assembly process. For multi-storey buildings a very high degree of prefabrication, for example volumetric module elements, can be a solution to avoid moisture influence. Apart from using a construction method assuring assembly of the roof in an early stage, other methods are utilized.

The most advanced method of protecting construction works from weather influence is to cover the building site with suitable tents (light bearing structures with fixed tarp), see Figure 8.14.



Figure 8.14: a) Movable tent over the whole building. Älvsbacka Strand, Skellefteå, Martinsons Byggsystem, Lindbäcks Bygg AB and White Arkitekter AB (Photo: Anders Björnfot). b) Movable tent from the inside with integrated travelling crane (Photo: Anders Gustafsson).

There are tents covering the whole intended structure from the start, with openable roofs or tents protecting only the uppermost storey where the construction work is ongoing, successively moving up as the work progresses. A large advantage of this kind of weather protection is that it enables using a travelling crane inside it, see Figure 8.14 b). However, this method also has some limitations, for example ventilation problems during warmer periods, logistic complications and higher cost in comparison to other methods, Larsson and Söderlind (2006). Also, special foundations must be made for the columns carrying the tent upwards. Examples of the most common weather protection systems can be found in Axelson et al. (2004).

The general evaluation of applying weather protection systems is positive, since they provide a considerably improved working environment, which in turn leads to shortened production time, higher level of safety at work and improved final quality, see Martinsons (2010) and Larsson and Söderlind (2006). The main criticism regarding these systems is their high cost, as well as there are different opinions about their real necessity to fulfill the moisture requirements. As an example in case of the project Inre Hamnen in Sundsvall in Sweden, see Boverket (2010), the cost of the weather protection was approximately 4,6 percent of the production cost in the project. This seems to be the argument for their limited use. However, development to decrease the cost of weather protection should proceed since they are especially advantageous for the construction process during the autumn-winter season, see Larsson and Söderlind (2006).

8.3 Detailed design

The presentation below gives some examples of detail solutions, mainly for multi-storey buildings, but several are also applicable for small buildings (for example semidetached houses). Others can be found via direct contact with suppliers. The fundamentals regarding details and technology for building systems with solid wood elements can be found in *Massivträhandboken* (2006). Generally, there are just a few patented solutions within timber construction, but at the same time the technical information is usually well protected by the companies.

8.3.1 Wall elements

Wall elements in wood are generally of two basic types; light-frame elements or frame elements with solid wood panels. Light-frame elements are built up with vertical solid studs or so called lightweight studs (I-joists), with regular spacing 450 or 600 mm, see Figure 8.15 a). To ensure good thermal insulation in external walls a cross-wise layer 45 mm thick often is added. This can be done either on the inside or on the outside of the vertical studs. The external cladding can be made of wood, steel sheets or cement based materials, but should be ventilated to enable leaking or condensed water to dry out. In the example shown in Figure 8.15 a) 22 mm vertical boards with overlapping vertical battens are used, see also horizontal section in Figure 8.18. The vapour barrier is preferably placed between the inner cross-wise layer, if present, and the load-bearing stud layer to ensure that the barrier is not penetrated by fasteners to the sheathing boards and when installing fasteners to hang for example cupboards and decorations on the wall. Two layers of plasterboard or other dense materials may be needed to provide good sound insulation and fire protection.

A wall element built from a solid wood panel can be almost anything from a single plate to more complicated units consisting of several timber panels. Usually panels with thickness 72 – 95 mm have sufficient load bearing capacity for multi-storey buildings, see Figure 8.15 b). X-Lam-panels are commonly used and can be joined either by screwing or gluing. The solid wood element provides excellent stiffness to the structure and possibility to transfer large horizontal wind forces. However, in external walls, thermal insulation is generally not sufficient and the solid wood wall needs to be complemented with a light-frame stud layer on the outside. The example in Figure 8.15 b) also has an insulation layer on the inside. The vapour barrier is applied on the inner side of the inner insulation layer as in Figure 8.15 a).

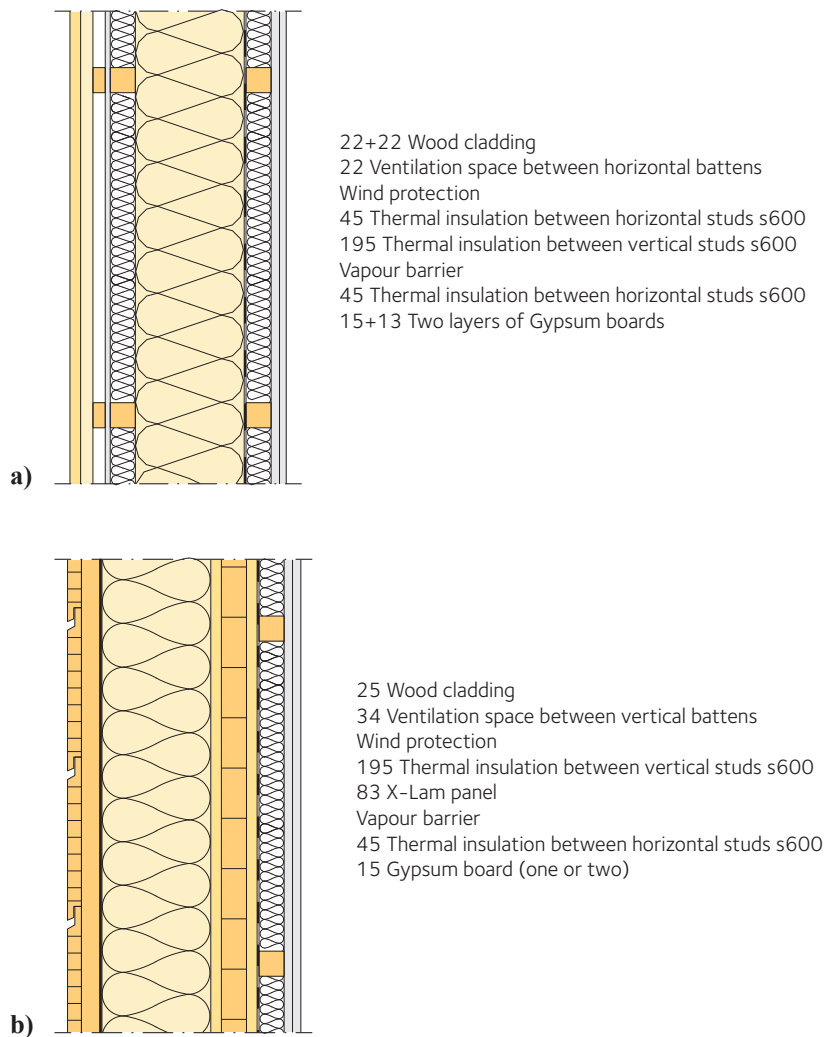
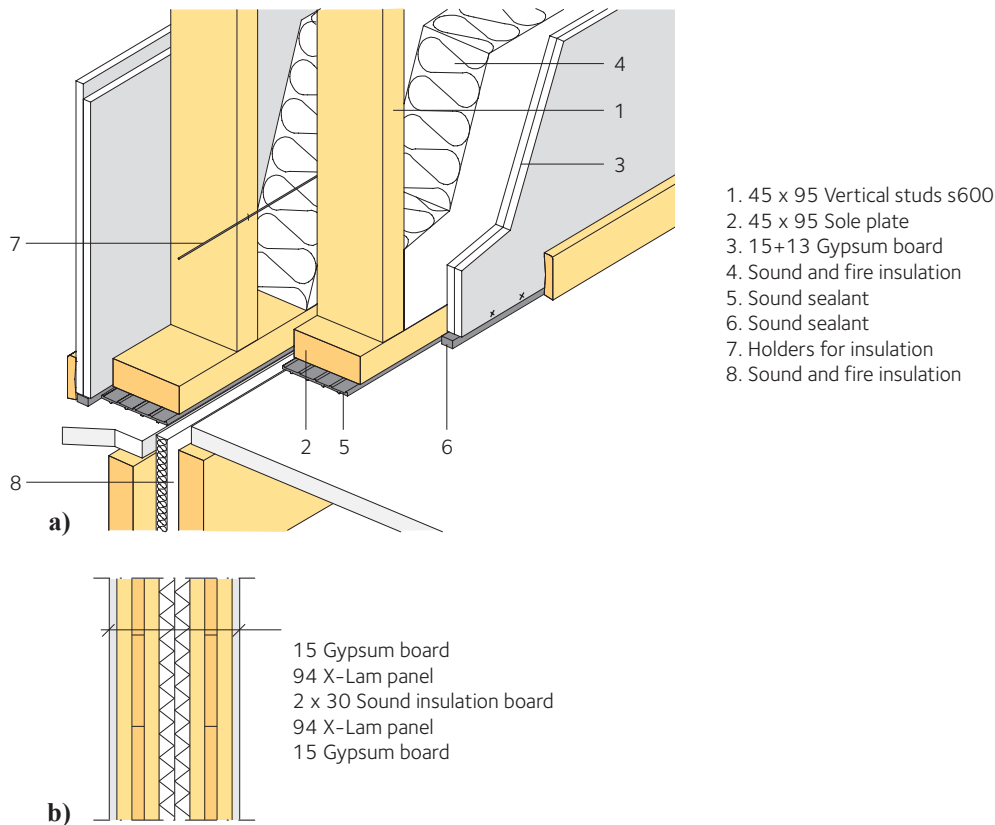


Figure 8.15: Example of a) external wall with light-frame structure, b) external wall with solid wood frame.

Two examples of apartment separating wood-frame walls are presented in Figure 8.16. The example in Figure 8.16 a) is typical for timber frame buildings with a double wall with air space between the two sides to provide good sound reduction. Double gypsum linings are used to meet requirements of fire resistance. The wall studs are braced against buckling in the weak direction by steel ties to ensure stability in fire after the gypsum boards are lost. The example in Figure 8.16 b) with double X-Lam panels with sound insulation boards at the centre provides good sound reduction for airborne sound (61 dB).



**Figure 8.16: a) Apartment separating wall in light timber frame.
b) Apartment separating wall with cross-laminated timber, thickness approximately 280 mm, (KLH).**

Apartment separating walls in timber tend to become quite thick, which reduces the area possible to sublet for the client. Thick walls can decrease the return on investments, what also arises for modular buildings, where the wall structure is double in the building system.

Outer corners in walls also needs to be connected, see Figures 8.17 and 8.18:

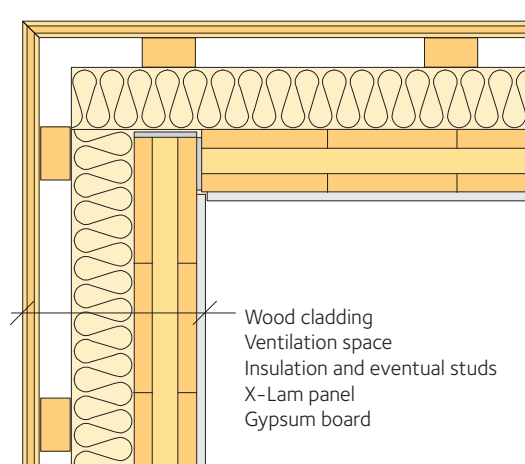


Figure 8.17: Horizontal section showing corner connection of external wall with cross-laminated timber.

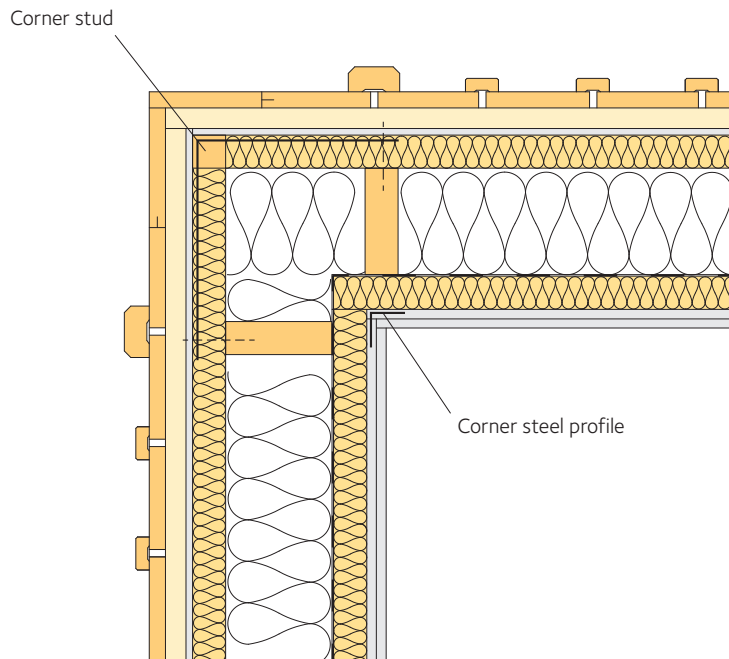


Figure 8.18: Horizontal section of external wall connection in a light frame system. The wall cross section is the same as in Figure 8.15 a), completed with corner details.

External walls are thicker than internal walls to provide good insulation. The thickness of external walls increases with increasing demands on energy efficiency. External walls are subject to several functional requirements; thermal insulation, moisture barrier, sound insulation from sounds in the outer environment, horizontal, and vertical load-carrying capacity. External walls of solid wood panels need to be completed with insulation material, thus implying that a regular light-weight wall must be built outside the actual load-carrying structure, see Figure 8.17. Today there are also systems where the facade is hung up in the primary structure without extra studs, what also reduces thermal bridges. Light frame walls are on the other hand not quite as strong as solid wood walls. The connection between light frame walls in the outer corner can be realised as in Figure 8.18.

8.3.2 Floor elements

One of the toughest challenges to handle in the design of timber buildings is to find a good solution for floor elements. The different types of load-bearing timber elements which can be used in a floor are described in Section 8.1. The challenge is to limit the total depth of the floor for given requirements of span length. From a structural point of view this is governed by serviceability requirements concerning vibrations and deflection, see Chapter 7.

The design is also strongly dependent on the requirements concerning fire resistance, sound performance and integration with services. For floors in single-family houses or within the same apartment, simple floor designs can be found only considering structural serviceability requirements. The real challenge is for floors separating apartments, where the floor must be designed to meet strict fire resistance and sound insulation requirements, see Section 8.2.

An example of floor design used in a timber frame building system is shown in Figure 8.19. This type of solution is typical for timber frame systems used for residential construction in North America and UK, although this design is adapted for higher sound performance. The main truss joists are quite deep and was chosen to allow installation of services horizontally.

The ceiling consists of a double gypsum layer suspended in metal sheet profiles (resilient channels) specially designed for reduction of sound transfer through the floor, see Figure 8.19 b).

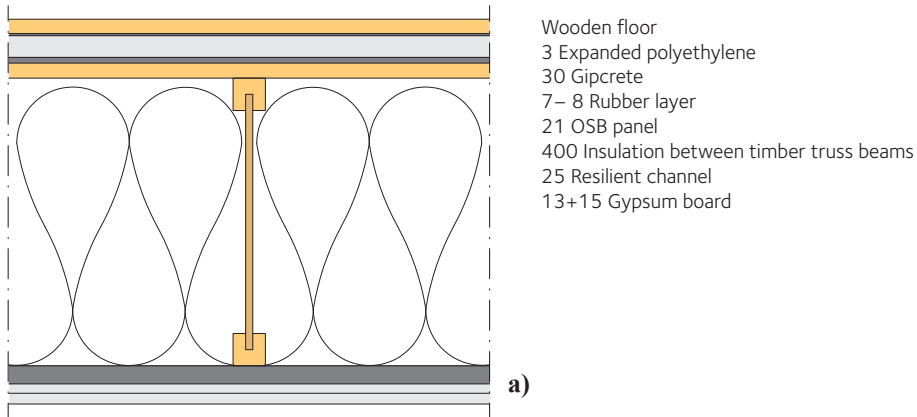


Figure 8.19: a) Typical floor design in timber frame building system. b) Photo of the bottom part of the floor with electric installations.

A light-weight floor of the type shown in Figure 8.19 is quite complex in design with many layers and is not robust to handle during construction. It is quite sensitive to errors which may lead to reduced sound performance, for example if installations unintentionally creates contact between the suspended ceiling and the structure above. For this reason floors in light-weight systems are often designed with complete separation between the load-bearing structure and the ceiling below. The ceiling needs a load-bearing structure separate from the floor itself and cannot be connected to the floor. An example of this approach is shown in Figure 8.20. As described in Section 8.2.2 separation between top and bottom parts of the floor is an advantage automatically obtained in prefabricated volumetric type construction leading to good acoustic performance.

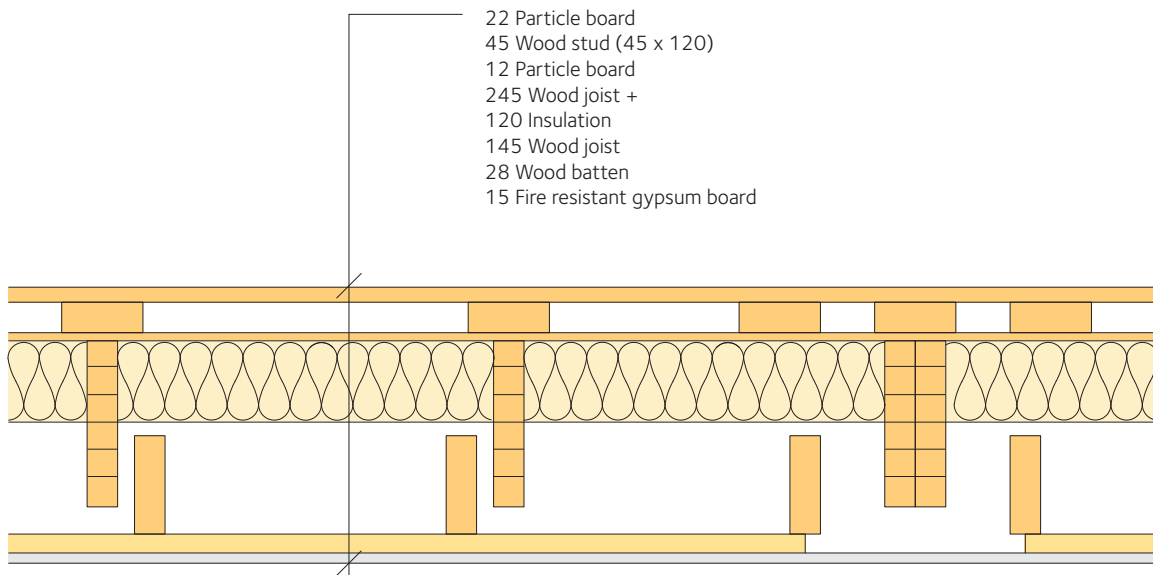


Figure 8.20: Apartment separating floor design (Tyréns & Derome).

Floor elements utilising solid wood elements can be divided into three main types, see Figure 8.21:

- **Plane elements:** nailed or dowelled plates, stress-laminated plates, cross-laminated plates.
- **Cassettes:** plates with empty spaces, for example H-beams (made out of thinner beams of structural timber, LVL or glulam as webs) and solid wood or wood based panels as flanges.
- **Composite floors** with timber and concrete (different designs are possible).

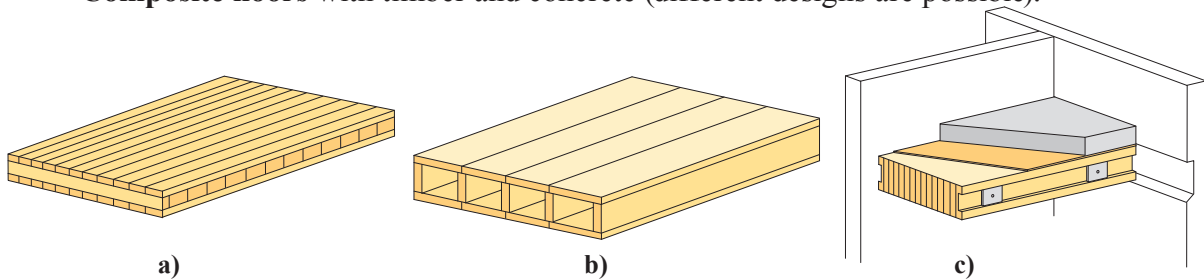


Figure 8.21: Illustration of a) cross-laminated element, b) cassette element and c) composite floor design.

Which solution is used is basically dependent on the acoustic requirements the floor needs to meet and the span needed. The plane floor elements usually must be accompanied by either a floating floor or a suspended ceiling due to the sound insulation requirements (for floors between apartments, in particular), see for example Figure 8.22.

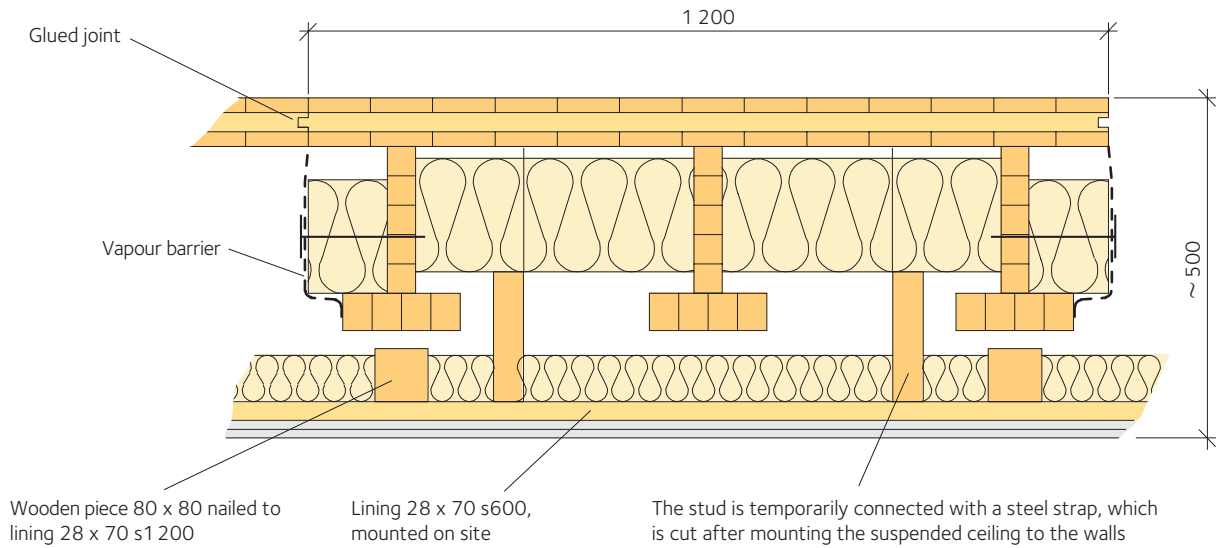


Figure 8.22: Example of a floor cassette that can be used for span lengths up to 6,5 m. The deck is made of a 3-layered cross laminated plate, supported by glulam beams with bottom flanges with a suspended ceiling (Martinsons).

Composite floors, where timber is combined with another material, present many possibilities for good technical performance, but are somewhat expensive to produce, see Figure 8.23. The idea is to have timber in tension and concrete in compression, while the fasteners are loaded mainly in shear. The stiffness of the connections completely governs the performance of the floor, see Chapter 5. Composite timber-concrete structures can also be used to restore existing timber floors by casting a concrete plate on top of already existing joists. Floor spans can reach up to 12 m with this technique and it can also be combined with pre-stressing to reach even longer spans.

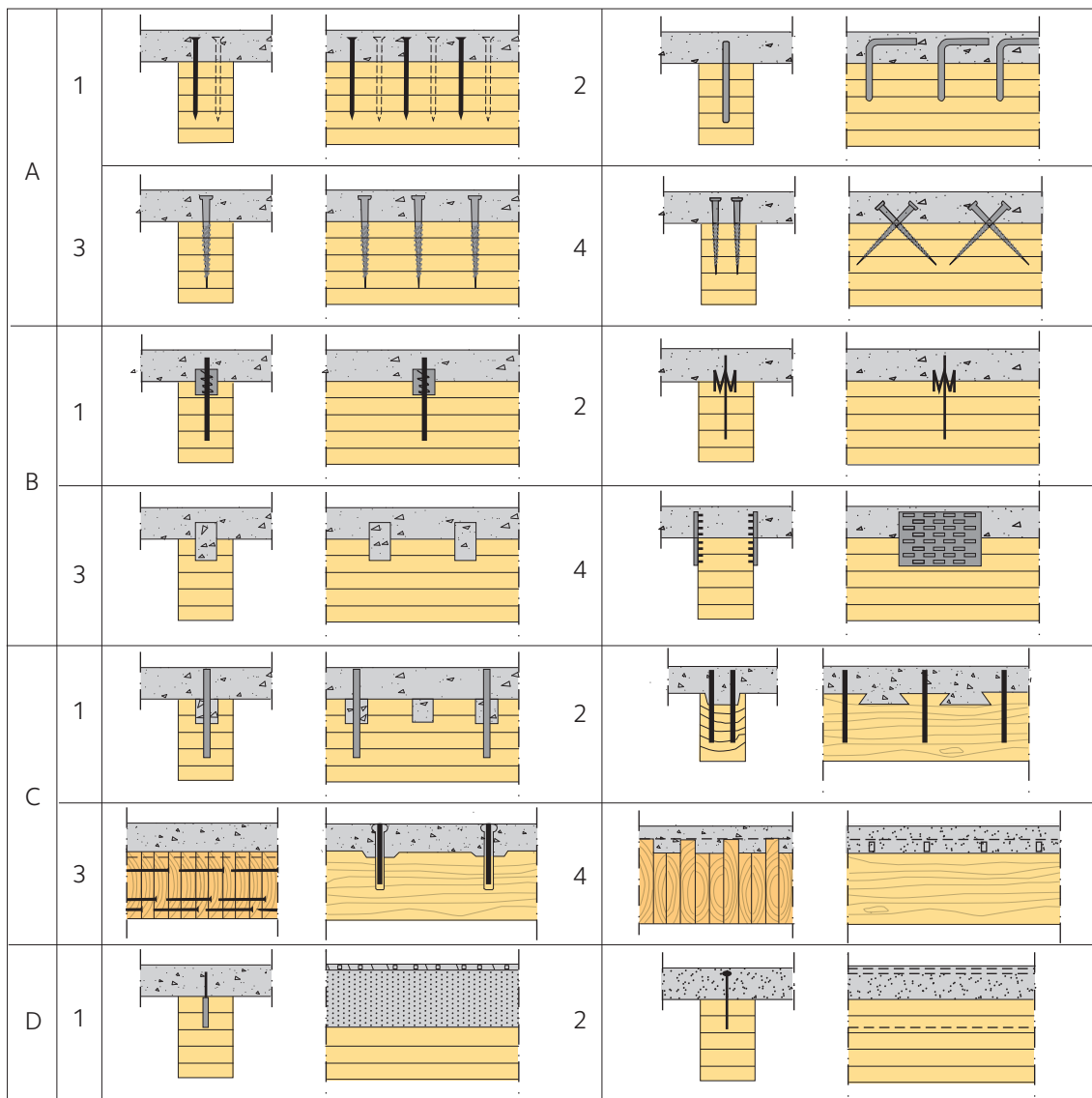
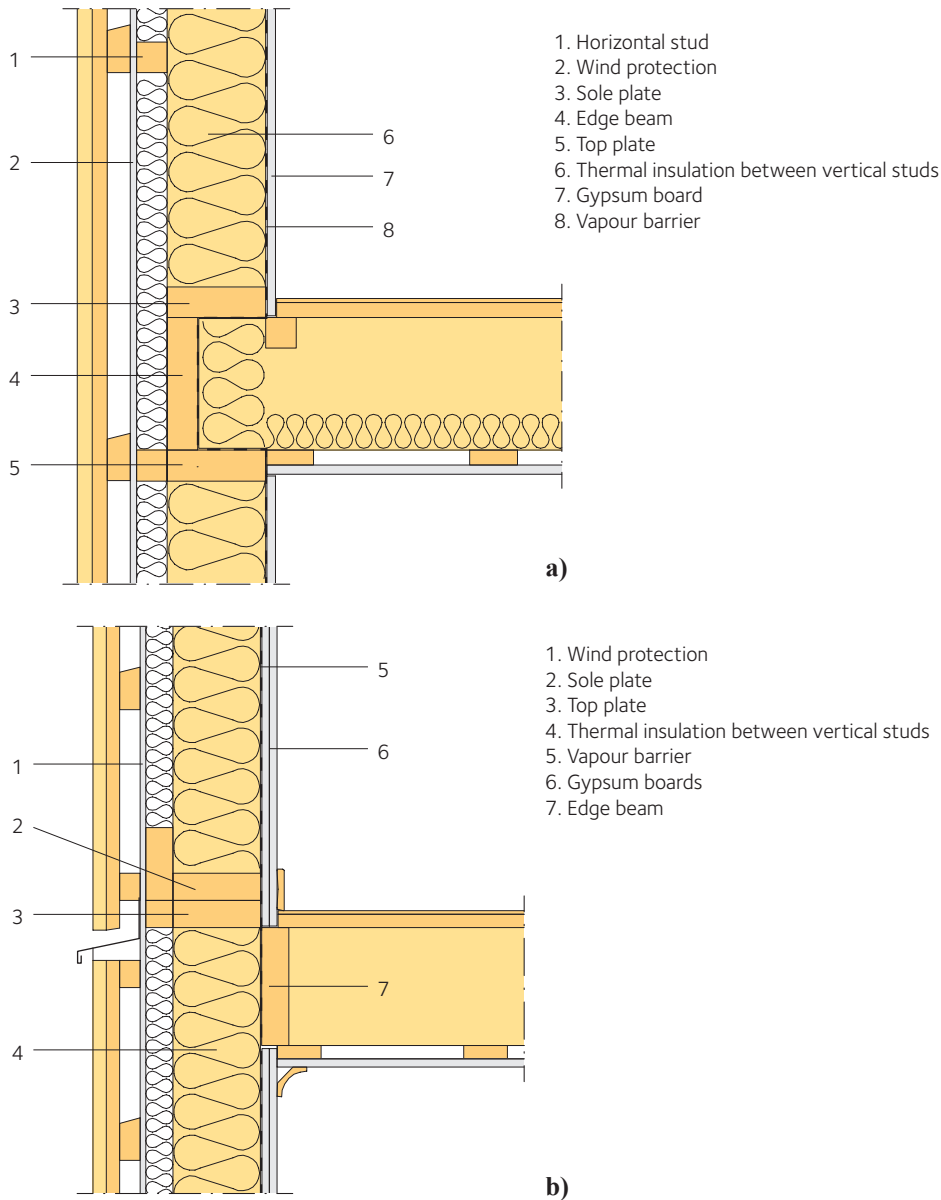


Figure 8.23: Different types of connectors in timber-concrete floors, Lukaszewska (2009).

8.3.3 Wall-floor connections

There are two main principles of placing floor elements; they can be either laid on top of the load-bearing walls, see Figure 8.24 a) or hung on the walls, see Figure 8.24 b). Placing it on top of the walls can lead to large forces in compression perpendicular to the grain and difficulties to achieve air tightness. Hanging floors can result in a weaker joint with a risk for increased deflection (note that the connectors are not visible in Figure 8.24 b). It can also hang only on the flanges, which often is the case for cassette floors, see Figure 8.22.



**Figure 8.24: a) Example light-frame structure with floor element placed on top of the walls.
b) Example light-frame structure with floor element hanging from the walls.**

In many cases the structural details are not commonly available. Solutions can be found only as examples of existing structures. In the wall-floor connection, many functional requirements must be met. Firstly, the vertical load bearing must be granted. In wood, the compression stresses perpendicular to the grain must be monitored for the vertical load. Secondly, the horizontal resistance must be sufficient; any connections must be able to transfer horizontal forces between stories. If the floor is placed on top of the wall as in Figure 8.24 a), the horizontal load must pass the floor element.

In contradiction to the requirements on force transfer, good acoustic performance is achieved when having loose connections between elements. Oftentimes, a solution is to have some kind of vibration damping material between the elements. This weakens the structural stiffness of

the connection though. In the wall-floor connection of an external wall, the vapour barrier must be continuous when passing the connection. Finally, the connection itself must have sufficient fire resistance, for example connectors of steel must be covered and wooden pieces with small dimensions should be covered by plasterboard or equivalent. An example of a detailed solution of a wall-floor connection can be observed in Figure 8.25. In this two-storey project stress laminated floors were used. This kind of floor elements exhibits worse acoustic performance (especially regarding step sound insulation) than cassette floors, which can be acceptable for non-residential buildings.

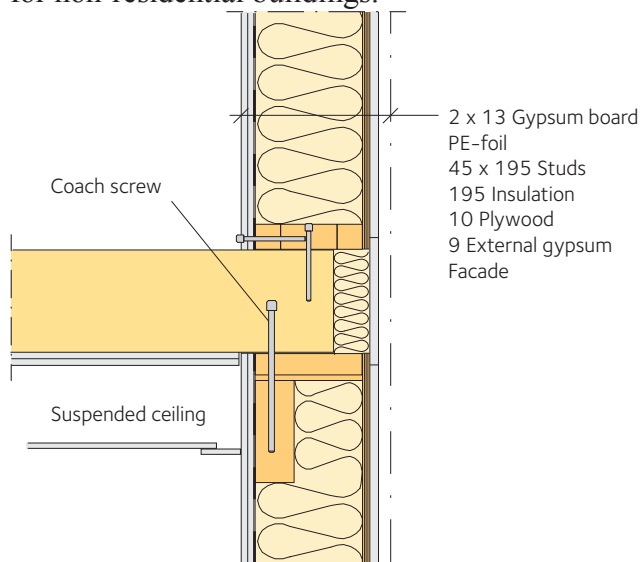


Figure 8.25: Solid floor element supported on a timber frame wall, Massivträhandboken (2006).

When hanging floor elements on the wall a fastener is needed. A special hanger has been developed in the example in Figure 8.26.

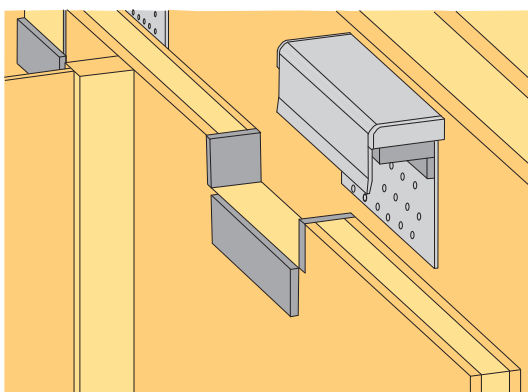


Figure 8.26: Hanger, MFB Handbok (2010).

To avoid problems with acoustic performance and vibrations being transferred in the wallfloor connection some vibrational damping needs to be applied. In Figure 8.26 the vibrational damping is applied inside the hanger. In Figure 8.24 b) it is instead accomplished by inserting damping between the sole plate of the upper wall and the top plate of the lower wall. A special example is shown in Figure 8.27 where a patented roller support between modules is used.

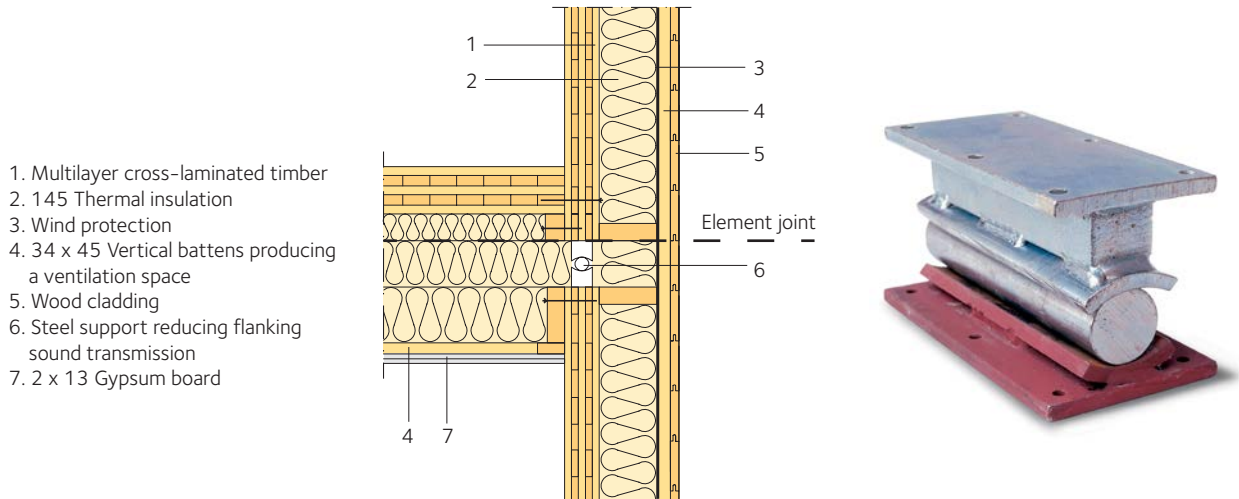


Figure 8.27: a) Floor-wall connection. b) Roller support (Setra Trälyftet).

8.3.4 Services

Timber structures are definitely advantageous compared to concrete and steel structures regarding the services, primarily due to easiness of making openings on site. On the other hand it is always more complicated to integrate the services in prefabricated building components than during on-site construction. Clients always demand a documentation of the installation system when a building system is used. Generally, in prefabricated building systems, it is favourable to lead the services vertically to reduce the number of passages between fire cells, see Lennartsson (2009). In multi-storey residential buildings it is more advantageous to have one shaft for each staircase so a large number of horizontal pipes can be avoided, instead of one shaft for the whole building accompanied by a large number of horizontal installations, Figure 8.28.

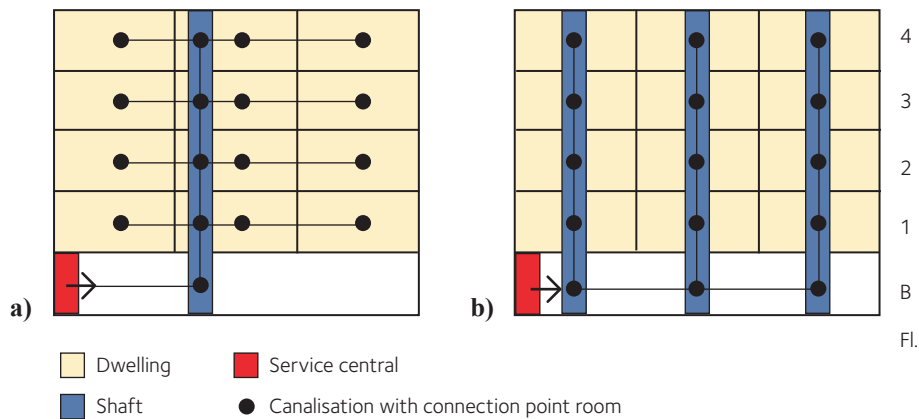


Figure 8.28: a) Single and b) multiple shafts for services, Lennartsson (2009).

With the new energy standards, which demand in practice heat exchange and reuse of pre-warmed supplied air to all the rooms, it is seldom possible to gather all the services in one shaft, especially for larger apartments, Figure 8.29. Supply to the apartments includes water, electricity and low current. Generally, these are small dimension installations. The only

exceptions are ventilation ducts that are space-demanding. The small dimension installations are usually placed in the shafts close to or inside the staircase, which makes them accessible for service work.

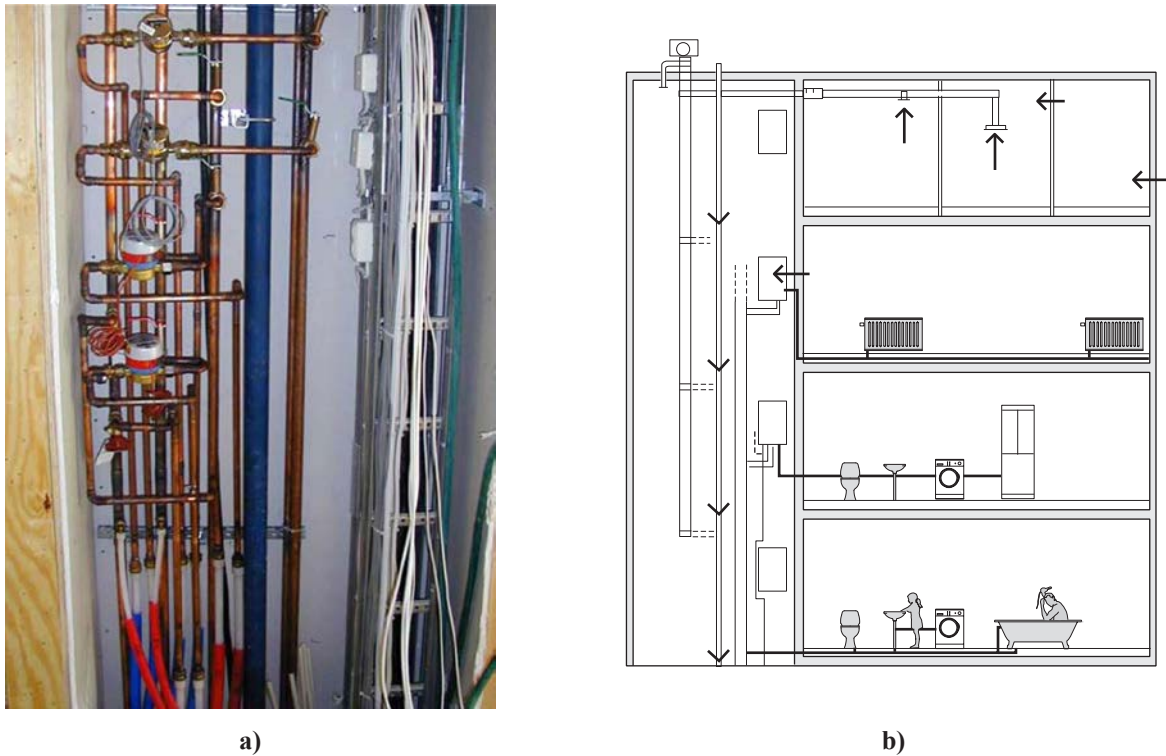


Figure 8.29: a) Installation shaft. b) Service layout.

The evacuation of air from apartments requires larger dimensions (normally 100 mm), see Figure 8.30. For these ducts the maintenance demand is much lower and thus the installation can be integrated in the wall structure (with 120 mm studs) or in separate shafts.

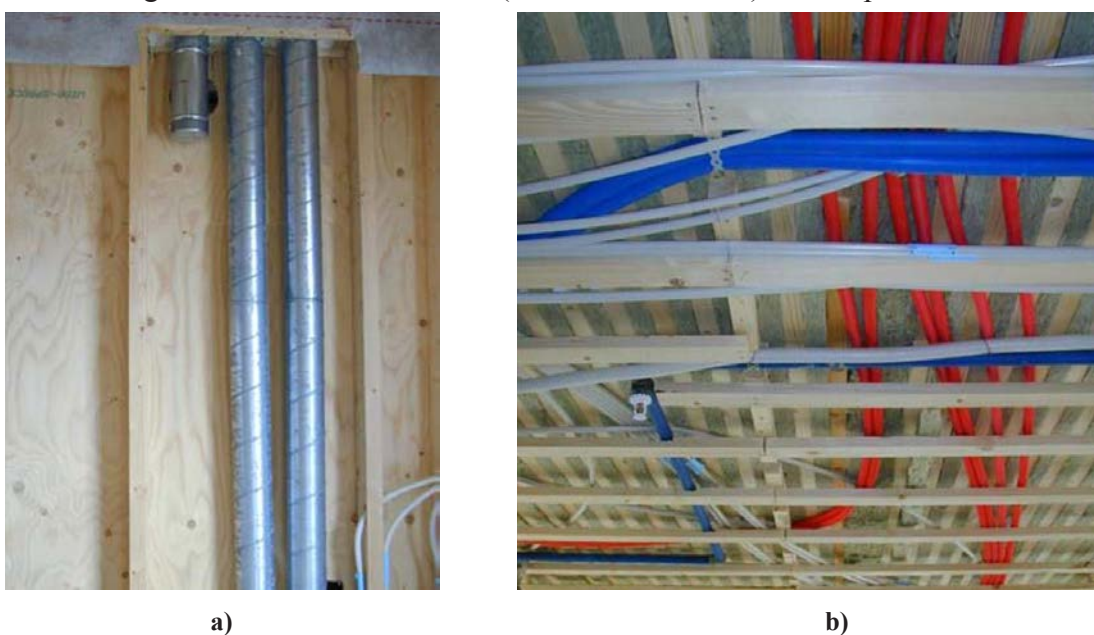


Figure 8.30: a) Ventilation ducts placed in a special shaft. b) Services placed under a floor element before finishing.

Regarding the horizontal installations within the apartments, there are two applied principles:

- placed in the service space created by the suspended ceiling
- integrated in the building part.

For a system based on panel elements (for example X-Lam), the service layer can be placed either on top in a floating floor or under the solid wood panel in the suspended ceiling. The choice of solution is strictly connected to the step sound insulation. If the ceiling is not included in the floor element, the services can be placed in this space before finishing, Figure 8.30. If the ceiling is integrated in the floor element, services are installed at the factory. With cassette floor elements, the height of the floor does not need to be increased because of services since the spaces between joists can be used for this purpose, Figure 8.31.

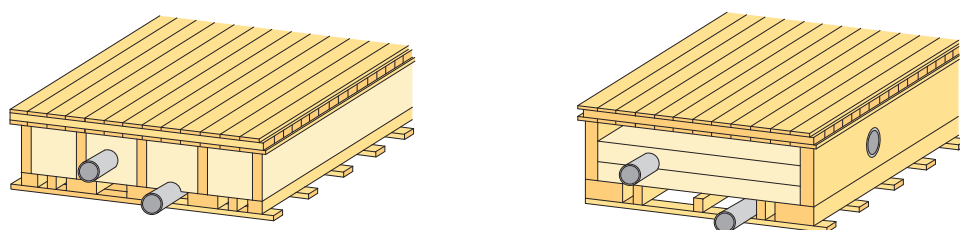


Figure 8.31: Placement of tubes between joists in a cassette floor (Martinsons).

8.3.5 Balconies

An example of a solution of a 1 200 mm × 1 200 mm balcony for a multi-storey timber building is presented in Figure 8.32. The solution consists of a X-Lam plate placed in a steel frame anchored in the X-Lam structure, see Martinsons. However, many different system solutions for timber balconies have been developed the last years, see for example Pousette et al. (2011).

8.3.6 Staircases and elevators

The walls of the staircase or the elevator shaft can have a stabilising function in the building. Solid wood elements (for example X-Lam) fit this purpose very well due to good lateral stiffness of the material. On the other hand, the two most important functions of these building parts are communication and evacuation. Therefore the structures in the staircase shaft have to meet the fire safety requirements. Several different combinations of structural materials can be used in the stairs. Wooden staircases are in general relatively cost effective, but they tend to result in bad acoustic performance. Staircases in steel are most suitable for complicated shapes, but on the other hand they are the most expensive ones and they might result in an industrial appearance, unless they are combined with wood or tiles. The design of stairs is usually made in collaboration with the stair supplier, who has profound knowledge in the field and uses specialised design software.

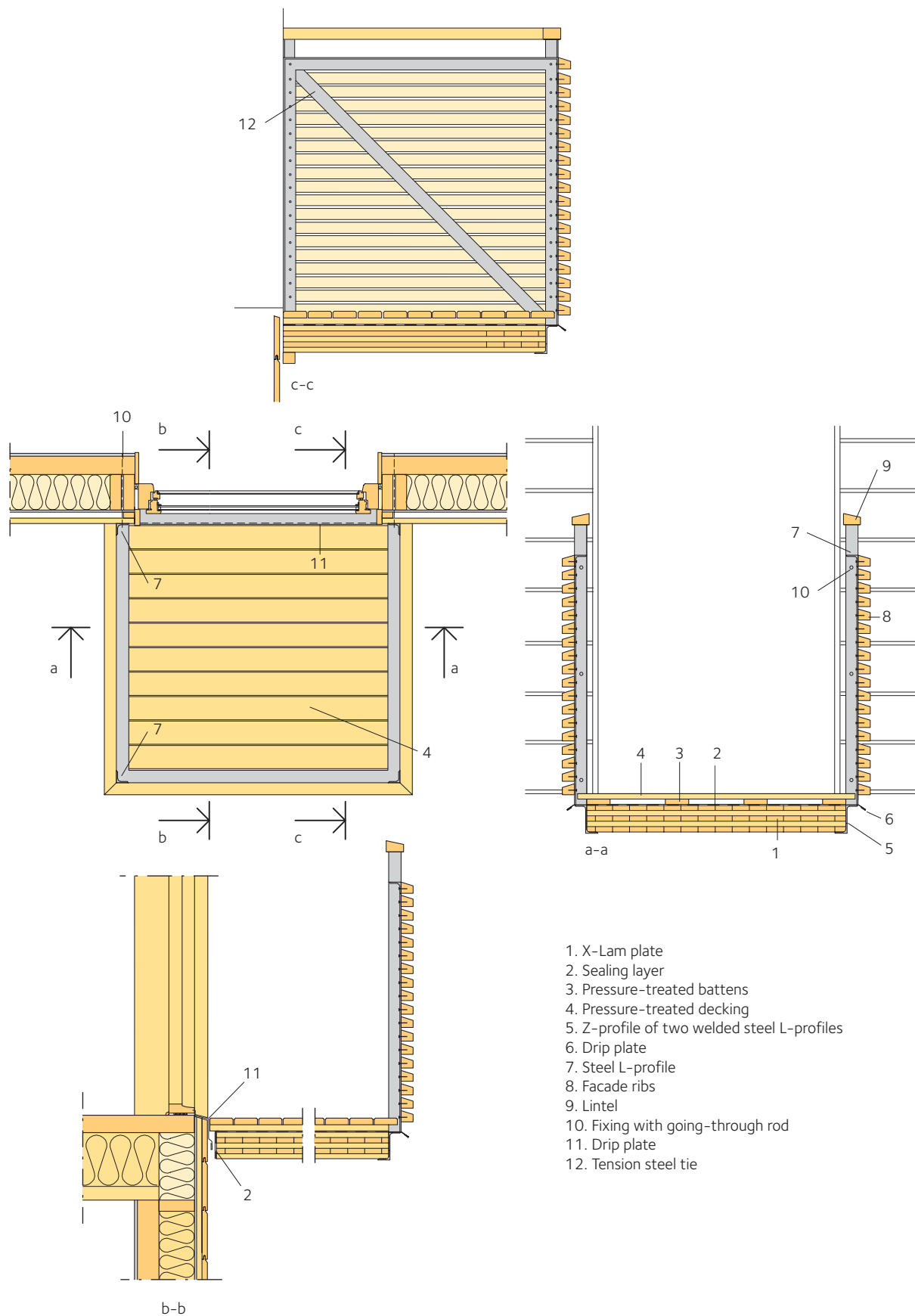


Figure 8.32: Solution for small balcony (Martinsons).

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Chapter 9

Structural systems for infrastructure

Anna Pousette

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9 Structural systems for infrastructure

9.1 Timber bridges

Wood is a suitable material for bridges since it is a light material with high load capacity in relation to its own weight. Timber is used as structural material both for pedestrian bridges and road bridges. For road bridges the requirements include sufficient carrying capacity for vehicular traffic with large concentrated wheel loads and this results in different methods, measures and types of bridges compared to pedestrian bridges. Timber bridges have proved to be strong enough to carry full traffic loads for small and moderate spans.

Bridges are built to pass obstacles and facilitate the movement of people and materials. Timber bridges have undergone a renaissance over the last twenty years. New techniques for timber structures have made it possible and competitive to build new timber bridges. The important development steps for wooden bridges are new structures, new materials and new connections, as described by Taylor et al. (1992). Durable and economic bridges for wide spans or high loads can be built with these new concepts together with structural protection. The technical development includes for example more efficient use of materials such as glued laminated wood, new types of connections, and better knowledge of structural behaviour of materials and components. Timber bridges in the old times were often made of timber beams with a wearing surface of wooden planks. A breakthrough for the modern timber bridges came with the stress-laminated bridge deck that improved the possibilities to build durable wooden bridges, described in Ritter (1990). Because of the limited movements of the deck, they could also be used with a durable asphalt layer on the surface.

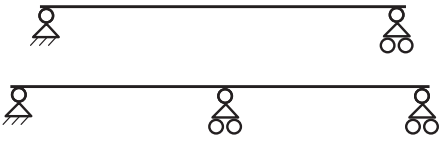
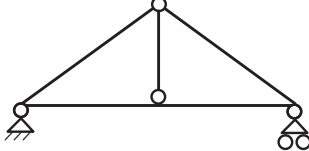
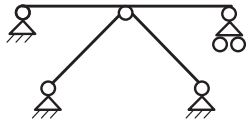
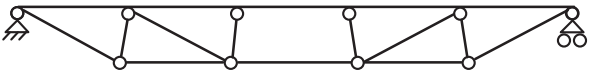
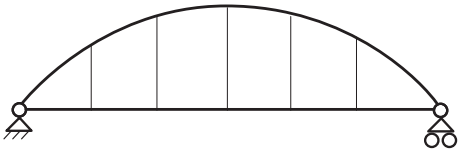
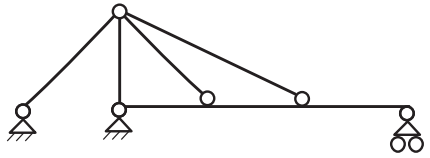
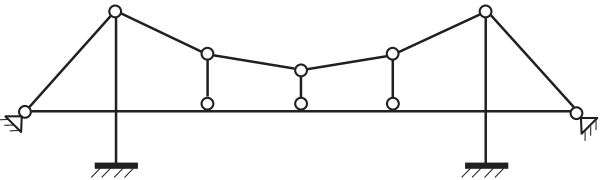
The selection of a timber bridge is often based on costs, aesthetics, environmental concerns, construction time and weight. There are many ways to design a timber bridge and there are many different bridge types. It is important for the planning and construction of timber bridges to consider a general protection concept, which includes design, choice of materials and structural detailing. With a good design, timber bridges can be competitive both in terms of construction and maintenance. With good architectural design timber bridges can also be attractive.

9.1.1 Types of timber bridges

Which type of bridge that is most suitable in each case depends on the prerequisites for spans, free height and type of traffic. The bridge should also fit into the landscape and it should be attractive both for those who travel on the road and for those who see it from the surroundings. The planning of a new timber bridge should take into account:

- topography at the bridge location
- type of crossing, a valley, a rail, river or other obstacles
- connections to surrounding roads
- nearby buildings or landscape
- ground conditions on site
- standards and regulations.

Table 9.1 Bridge types, static systems.

Bridge type	Term	Suitable spans ¹⁾
	Beams	5 – 30 m
	Kings post	10 – 50 m
	Strut frame	20 – 40 m
	Truss	10 – 40 m
	Arch	20 – 70 m
	Cable-stayed	20 – 130 m
	Suspension	20 – 130 m

¹⁾ Suitable spans apply to pedestrian bridges.

There are many types of timber bridges, see Table 9.1. The bridge deck typically consists of beams or a deck plate. For longer spans the bridge deck can be complemented by other structural parts. Different bridge types such as trusses, arches, cable stayed bridges and suspension bridges can be used for both road bridges and pedestrian bridges, see TräGuiden (www.traguiden.se).

Kings post truss bridges: have vertical rods that transfer loads from the bridge deck to diagonal members at the sides of the bridge deck. Kings post bridges can have large spans with a relatively small height of the bridge deck.

Strut frames: are old building systems that have diagonal compression struts under the bridge deck. The construction below the deck enhances the load-bearing capacity of the bridge deck or beam and transfers the compressive forces to the abutments or walls.

Trussed beams and trusses: can span longer with less material than homogeneous beam sections, but require larger structural depth. These bridges are built of glulam and are sometimes combined with tie rods of steel. They have many joints, which are very important for the strength, stiffness and durability of the structure. The trusses can be placed under the bridge deck or at the sides of the bridge deck. When possible, trusses below the bridge deck are favourable because several trusses can then be used, the secondary system will be simpler and the trusses will be protected against weather. When trusses are placed at the sides of a road bridge the depth of the truss should be big enough for a horizontal wind bracing system to be installed above the traffic lanes. The truss chords can also be designed with curved elements.

Trusses at the sides of the deck are used for pedestrian bridges with a width of 2 – 3 m and trusses with depth of about 1 – 1,5 m can act as bridge railing together with complementary parts. Experience shows that protection against moisture is important, but usually for aesthetic reasons the cladding does not cover the entire trusses. Yet for truss bridges, all exposed upper surfaces of the lower chord and the lower diagonal joints should be covered using metal plate cladding.

Arches: are usually made of glulam. The lamellas are bent during the gluing process and they can be performed with different radii. Due to limitations and efficiency of production and transportation the glulam arches are often made as 3-hinge arches. The arches can be placed below or above the bridge deck. Arches above the deck are placed at the sides and are combined with crossbeams under the deck and steel hangers. The joints are critical to the load-bearing capacity and durability, and the arches should be protected with cover plates and cladding. The arches can be treated with chemical impregnation or be untreated if they are well covered.

Suspension bridges and cable-stayed bridges: consist of a wooden bridge deck which is suspended from flexible steel cables or tie bars from pylons or towers. They are suitable for long clear spans. Most timber suspension bridges have been built for pedestrians.

9.1.2 Loads on bridges

Bridge design includes verification in serviceability and ultimate limit states for combinations of actions with design values of permanent, variable, and accidental actions as well as actions during execution. Loads on timber bridges should be calculated according to the relevant parts of Eurocode EN 1991; densities, self weight, snow loads, wind actions, thermal actions, actions during execution, accidental actions, traffic loads on bridges. Self weight is a permanent action. According to the material standards, SS-EN 338 respectively SS-EN 14080, the self weight of wood is based on the strength class, for example mean value 3,5 – 5,0 kN/m³ for C14 – C40, respectively 3,7 – 4,9 kN/m³ for GL20 – GL32. The self weight of asphalt and asphalt concrete is about 24 – 25 kN/m³. Bridges with roofs must be calculated for snow load on the roof. Ski and snowmobile bridges should also be designed for snow loads. Other bridges are assumed to have the snow removed and are not designed for snow loads.

Traffic loads on bridges correspond to the impact of traffic on the bridge in vertical and horizontal direction due to the weight of vehicles, braking and acceleration forces, centrifugal and transverse forces, loads on footways and cycle tracks, crowd loading, etc. They are free, variable loads and should be considered as short-term loads. For road bridges there are in accordance with Eurocode SS-EN 1991-2 several load models with values corresponding to

regular traffic with cars, busses and trucks. The load models contain large concentrated loads (wheel loads) acting on a surface of $0,4 \text{ m} \times 0,4 \text{ m}$ and a uniformly distributed load for each lane. Load Model 1 is shown in Figure 9.1.

There are two types of traffic loads for pedestrian bridges according to Eurocode SS-EN 1991-2. They are uniformly distributed load corresponding to people and crowds, and concentrated loads (wheel loads) of a service vehicle, shown in Figure 9.2. These loads should not be combined.

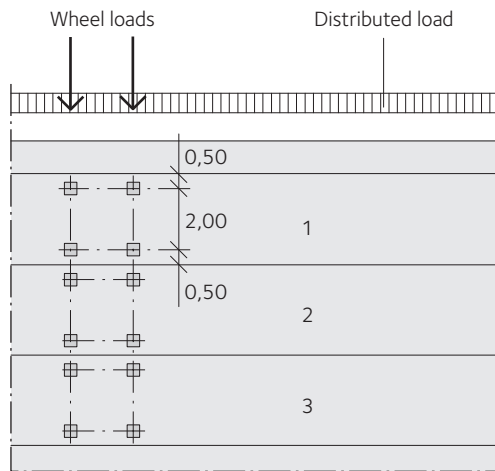


Figure 9.1: Example of traffic load on road bridge (distributed load and wheel loads).

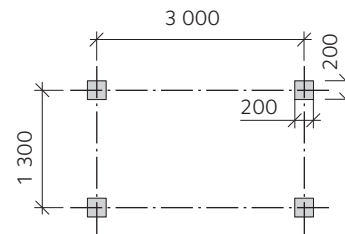


Figure 9.2: Example of traffic load on pedestrian bridge (wheel loads of service vehicle).

There are static and also aesthetic demands on railings and moreover, the railings are subject to weathering. Wooden pedestrian bridges usually have timber railings. Railings on road bridges must withstand the required impact forces of vehicles. This is evaluated by full-scale crash tests according to standards. Most often steel railings are used on wooden road bridges. The connection of the rail posts to the timber deck is very important and needs to be verified.

Vibrations caused by pedestrians are important when designing timber structures, due to the low weight of the wood material. Both vertical and horizontal vibrations can cause a feeling of unsafety for pedestrians standing or walking on the structure. Verifications concerning vibration for pedestrian bridges due to pedestrian traffic should be done and a verification of the comfort criteria should be performed if the fundamental frequency of the deck is less than 5 Hz for vertical vibrations and 2,5 Hz for horizontal and torsional vibrations. The comfort criterion is defined in terms of maximum acceptable acceleration of any part of the deck. Damping ratios for timber bridges can be taken according to Eurocode SS-EN 1995-2. Fatigue is rarely decisive for timber structures, and is usually not checked for pedestrian bridges.

9.1.3 Beam superstructures

For beam bridges the beams constitute the main structure. For other bridge types beams can be combined with additional structures such as arches or struts, see Table 9.1. The beam bridges are often constructed with glued laminated beams located below the bridge deck and with transverse or longitudinal plank decks, as described in Pousette (2008). The traditional beam bridge typically consists of the main beams, cross beams, transversal bracing, transverse floor

beams and a plank deck, see Figures 9.3 and 9.4. This type of bridge is suitable for pedestrian bridges, but also for small road bridges. The glulam beams are usually manufactured with a pre-camber so that the bridge is slightly curved. Beam bridges have a simple static system and they are easy to manufacture and build. The beams must be transversely braced to provide lateral strength and rigidity and a horizontal truss between the main beams can be utilised to care of the wind loads across the bridge. The bridge decks are normally open plank decks, but also flatwise glued laminated beams or other woodbased panels can be used together with a waterproofing layer under the wearing surface.

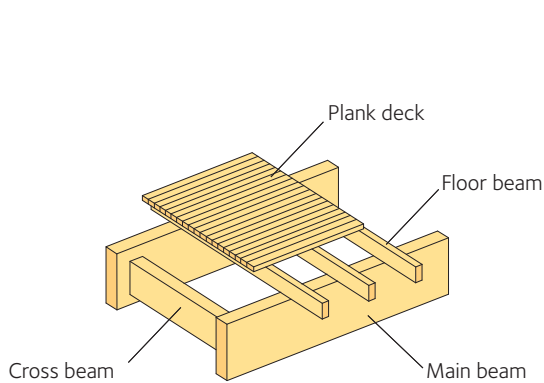


Figure 9.3: Build-up of beam bridge.

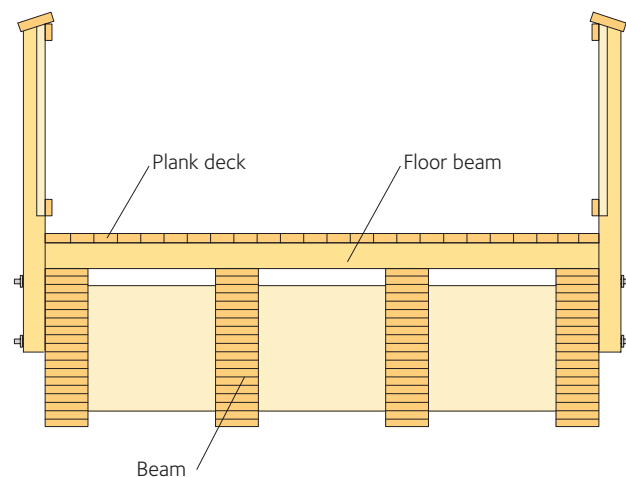


Figure 9.4: Beam bridge with four glulam beams.

Simply supported beams are easy to design. Longer bridges over several supports can be continuous, which is favourable in design. Bridges over several supports can also be designed with Gerber joints in a multi-span bridge system. Gerber beam means that the exterior span beams are cantilevered and support a drop-in beam in the interior span. Bridge superstructures can be prefabricated into parts or complete units that are transported to the building site and lifted into place.

The beam depth depends on the span, the bridge width, the number of beams and the beam widths. A pedestrian and bicycle bridge can have two beams for widths up to about 2,5 m. Three beams can handle widths up to about 4 m. Service vehicles usually require a bit deeper beams than the surface load alone. For a simply supported beam with a span of 10 m the required beam depth is about 600 – 800 mm, and for a 20 m span approximately 1 200 – 1 400 mm.

Concentrated vertical loads should be considered at a reference plane in the middle of the bridge deck (floor beams). For concentrated loads an effective load area with respect to the middle plane should be assumed, see Figure 9.5. This gives the number of floor beams that are loaded with the wheel load.

If structural timber members are exposed to abrasion by traffic, the depth used in the design should be the minimum depth allowed before replacement.

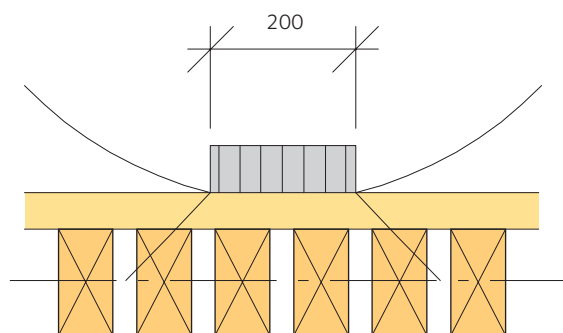


Figure 9.5: Beam bridge.

9.1.4 Deck superstructures

Stress-laminated deck superstructures consist of timber laminations of planks, or at larger spans of glued laminated beams, that are placed edgewise and compressed transversely with pre-stressing high-strength steel bars, see Figure 9.6. The bars are placed in pre-drilled holes through the laminations and are stressed in tension using a hydraulic jack. The tension force compresses the laminations so that the deck behaves like one large, solid plate of wood, as described by Pousette et al. (2002). This leads to a stable and stiff deck which is suitable for very high wheel loads.

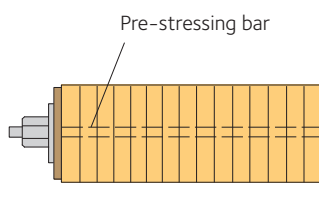


Figure 9.6: Stress-laminated deck.

A stress-laminated deck functions as both main structure and bridge deck. The deck is stiff in its transverse direction, thus normally no special wind bracing is needed to withstand e.g. wind loads across the bridge. A deck plate can also be combined with additional structures such as arches or struts, see Table 9.1.

Deck superstructures are suitable for road bridges, but also for pedestrian and bicycle bridges. The bridges are usually provided with a water-tight layer protecting the wood against moisture from above and an asphalt pavement, functioning as wearing protection, see Figure 9.7. Together with a cladding on the edges this gives the bearing structure a good weather protection. The bridges have either steel or wood railing.

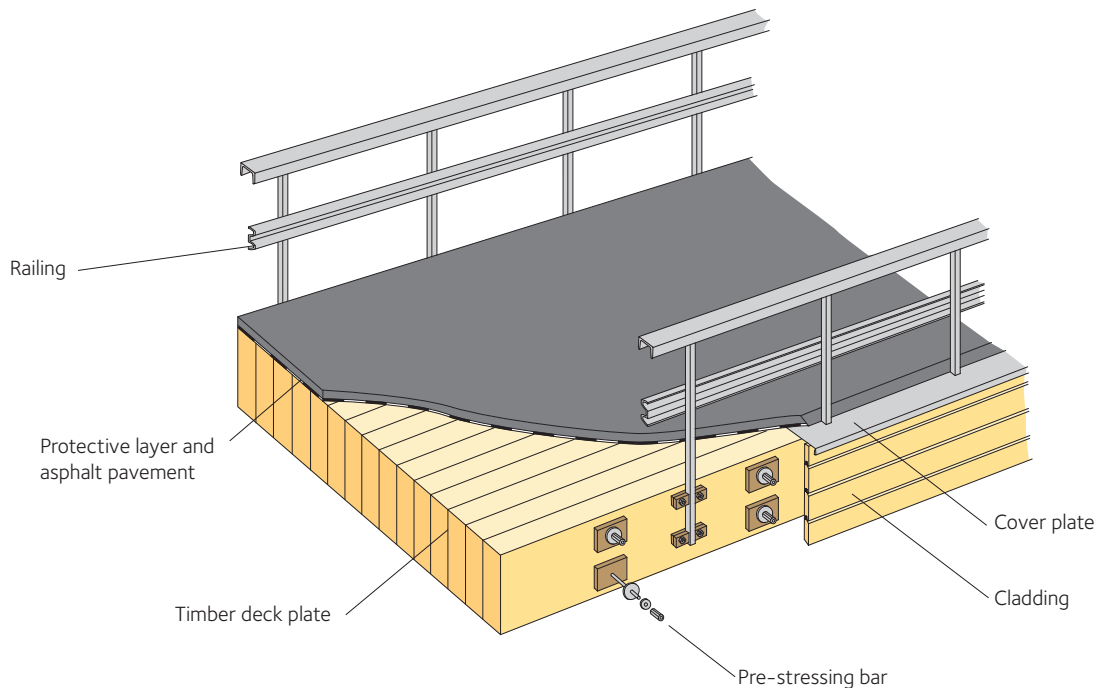


Figure 9.7: Stress-laminated bridge.

A stress-laminated deck is easy to build and has a good load distribution of the wheel loads. Load transfer between the laminations is developed by compression and friction. The friction is created by the transverse compression applied to the deck. The deck acts as a continuous slab without transverse or longitudinal joints that can adversely affect wearing surface performance. Since the load transfer between the laminations is developed from friction, butt joints are permitted within certain limitations and thus all laminations do not have to be continuous. There are few limitations in production of length and width of a stress-laminated bridge. It is also possible to manufacture curved bridge decks according to specific needs and/or locations.

The analysis of laminated timber deck plates should be based upon one of the following (according to Eurocode SS-EN 1995-2): orthotropic plate theory, modelling deck plate by a grid or simplified method. In an advanced analysis, for deck plates made of softwood laminations, the relationships for the system properties (elasticity and shear modulus in different directions) are given. The Poisson ratio may be taken as zero.

Concentrated vertical loads should be considered at a reference plane in the middle of the deck plate and an effective load area should be assumed, see Figure 9.8. The width b_w is the wheel width and $b_{w,middle}$ is the width at the middle plane calculated from the load dispersion angles β_1 and β_2 for different materials. Dispersion angle β of concentrated loads for various materials can be found in Table 5.2 in Eurocode SS-EN 1995-2.

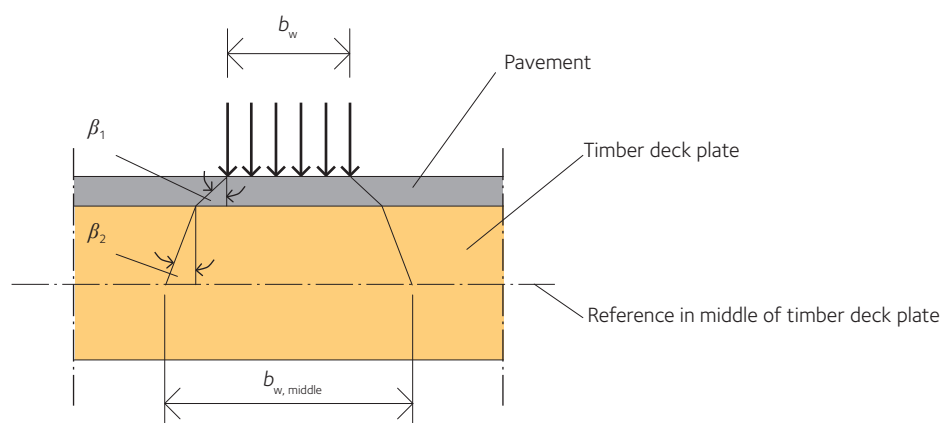


Figure 9.8: Stress-laminated bridge.

In a simplified analysis the deck plate may be replaced by one or several beams in the direction of the laminations with the effective width b_{ef} calculated as:

$$b_{ef} = b_{w,middle} + a \quad \text{Eq. 9.1}$$

where:

$b_{w,middle}$ should be calculated according to Figure 9.8,

a is given in the standard, for example 0,3 m for a stress-laminated deck plate.

This beam is loaded with distributed load and wheel loads and the required deck thickness can be calculated.

When wheel loads are placed on the deck, the transverse bending produces a tendency of gaps forming between the laminations on the lower side of the deck and the transverse shear creates a tendency of vertical slip of the laminations, see Figure 9.9. A sufficient level of compressive pre-stress between the laminations prevents both actions. For transverse bending, the compressive stress directly counteracts the tension on the lower side of the deck. Vertical slip is prevented by the friction between the laminations resulting from the compressive pre-stress. Maintaining an adequate level of pre-stress is the most important issue regarding a stress-laminated deck plate. If the pre-stressing forces are reduced over time due to shrinkage or creep, it is always possible to re-stress them. The loss of pre-stressing force is considerable during the first period after construction, but after re-stressing a number of times during the service life the force seems to stabilize as the moisture content also stabilizes. However, the pre-stressing force will always vary significantly with temperature.

The long-term pre-stressing forces shall be high enough to ensure that no inter-laminar slip occurs. The maximum allowed shear force caused by vertical and horizontal actions depends on the coefficient of friction, the minimum long-term residual compressive stress due to prestressing, and the thickness of the plate. The coefficient of friction is determined based on wood species, roughness of contact surface, treatment of the timber, moisture content, and residual stress level between laminations. In areas subjected to concentrated loads, the minimum long-term residual compressive stress due to pre-stressing between laminations should not be below 0,35 N/mm².

According to Eurocode SS-EN 1995-2 the long-term residual pre-stressing can normally be assumed to be greater than $0,35 \text{ N/mm}^2$, provided that the initial pre-stress is at least $1,0 \text{ N/mm}^2$, the moisture content of the laminations at the time of pre-stressing is not more than 16 %, and the variation of in-service moisture content in the deck plate is limited by an adequate protection layer.

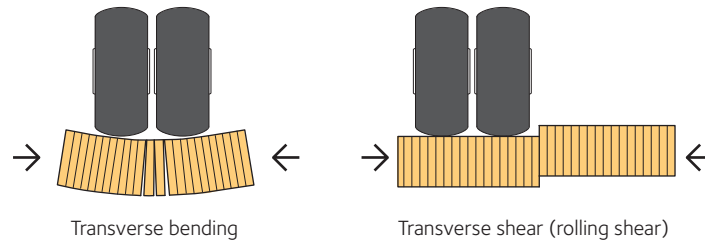


Figure 9.9: Actions of wheel loads on stress-laminated decks.

The resulting pre-stressing forces should act centrally on the timber cross-section. The compressive stress perpendicular to the grain during pre-stressing in the contact area of the anchorage plate shall be verified. As the pre-stressing forces at the edge can be too high for the softwood of the deck, hardwood parts are often used for distribution of the pre-stressing.

Stress-laminated bridges are common for both pedestrian bridges and road bridges. There are three kinds of stress-laminated structures: deck plates, T-beam bridges and box-beam bridges. Deck plates are built as either simply supported or continuous over several supports. T- and box-beam bridges are usually simply supported with joints at each support. T- and box-beam bridges are used for longer spans. This design becomes more complex, see Figures 9.10 and 9.11. Transfer of shear forces between web and flanges as well as deformations for asymmetrical loads are significant in the design of T-beams and box-beams.

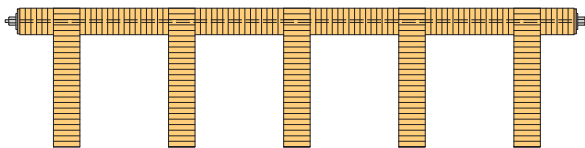


Figure 9.10: Stress-laminated T-beam bridge.

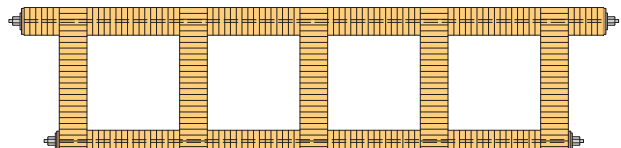


Figure 9.11: Stress-laminated box-beam bridge.

T-beam bridges and box-beam bridges become somewhat deeper than a deck plate with the same span, but they are built with less material. Suitable spans and an approximate indication of required depths are shown in Table 9.2 for simply supported bridges with a deck width of about 3 m. The depth depends on the load, the paving, the wood material, and the width. For T- and box-beam bridges it also depends on the number of webs and their width and material.

Table 9.2 Stress-laminated deck plates, width 3 m.

Deck type	Bridge type	Suitable span (m)	Depth (m)
Stress-laminated deck plate	Road bridge	5 – 25	0,3 – 1,2
Stress-laminated deck plate	Pedestrian bridge	5 – 20	0,2 – 0,6
Stress-laminated T-beam bridge	Pedestrian bridge	20 – 30	0,8 – 1,5 ¹⁾
Stress-laminated box-beam bridge	Pedestrian bridge	20 – 30	0,6 – 1,2 ¹⁾

¹⁾ Depends also on the number and type of webs.

There are also other types of decks manufactured of planks or glulam beams placed side by side and nailed or glued together, or of cross-laminated timber (X-Lam) or laminated veneer lumber (LVL). X-Lam consists of at least three layers of crosswise glued boards (most commonly of softwood), but also five or seven layers are used, see also Section 2.7. The X-Lam boards can be combined with glued laminated beams to form a T-section. The connection between the bridge deck and the beams is important to transmit the shear forces.

The idea of timber-concrete composites is to combine the high tensile strength of timber with the high compression resistance of concrete. The first applications were in structural systems for new buildings and in renovation. The shear forces can be transmitted by mechanical connectors or by direct contact between timber and concrete (grooved connection). The stress distribution depends on the rigidity of the connectors.

Timber-concrete connections in composite beams can be designed as laterally loaded dowel-type fasteners. Where there is an intermediate non-structural layer between the timber and the concrete (for example for formwork), see Figure 9.12, the strength and stiffness parameters should be determined by applying a special analysis or by tests.

For grooved connections, the shear force should be taken through direct contact pressure between the wood and the concrete cast in the groove. Sufficient resistance of the concrete and timber parts of the connection should be verified. The concrete and timber parts must be held together to avoid separation and the connection should be designed for a small tensile force between the timber and the concrete.

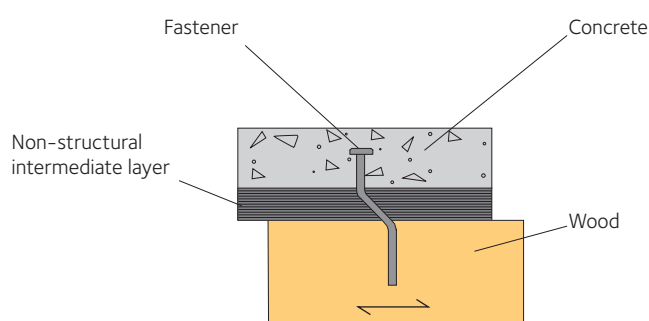


Figure 9.12: Timber-concrete composite deck, with intermediate layer.

9.1.5 Details

The design of connections in timber bridges is important for the load-bearing capacity and durability. Connections such as nails, bolts and timber connectors can be used, but some connections have too low capacity and give greater slip than desirable. For rigid connections with large forces, for example in truss bridges, embedded steel plates in sawn slots in the members to be connected can be used together with dowels. Steel is also used in other components of timber bridges. Steel members can be used to transfer moments, for example rigid frames for lateral stability. For cross beams, steel beams are often used due to the low shear capacity of wood which would require large cross-sections. Also, for tensile members as vertical suspension bars, steel is the preferred material.

Railings are usually made of wood or steel, see Figures 9.13 and 9.14. They must comply with load and safety requirements. There are also requirements for railing height and dimensions of the various components of a railing. Especially for pedestrian bridges there are recommendations for space between the bars and clear distance between parts. This means that to meet the requirements, the railing has to be designed so that it can not be climbed. In addition, the aesthetic design is of great importance for railings. Especially for pedestrian bridges the railings should be aesthetically pleasing, and there are many alternative designs to choose from.

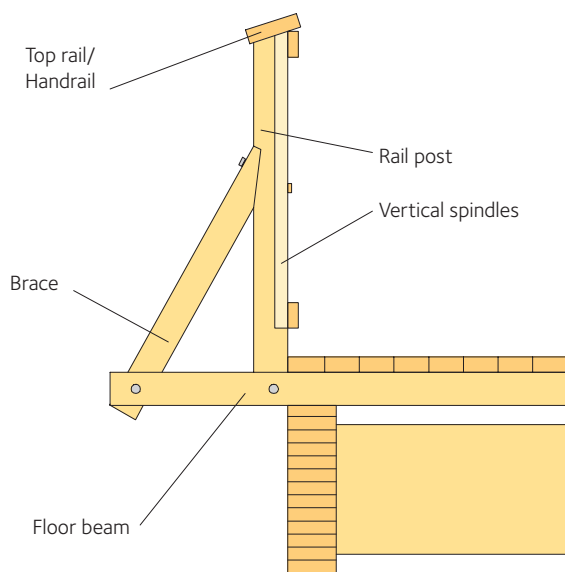


Figure 9.13: Wooden railing on pedestrian bridge.

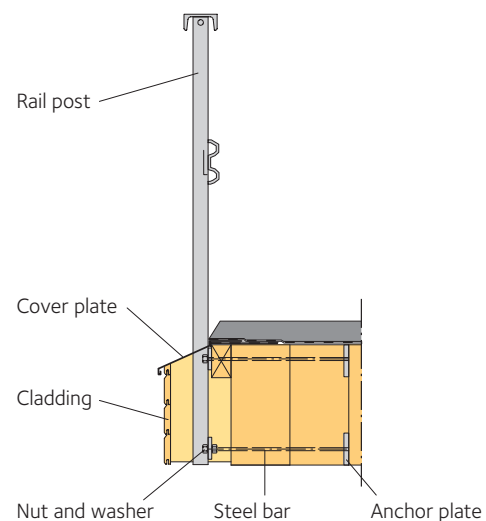


Figure 9.14: Steel railing on road bridge.

Support attachments for timber bridges must be designed to resist the vertical and lateral forces transmitted from the superstructure to the substructure. Bearings for beam bridges can often be simple with a beam attached to the substructure. Stress-laminated deck bridges are most frequently placed on rubber strips and connected with steel connections, see Figure 9.15. The movements of wooden bridges are usually relatively small. The transition between road and bridge must be tight, so that water cannot flow down to the abutment under the bridge. The abutment should be designed so that water will drain away.

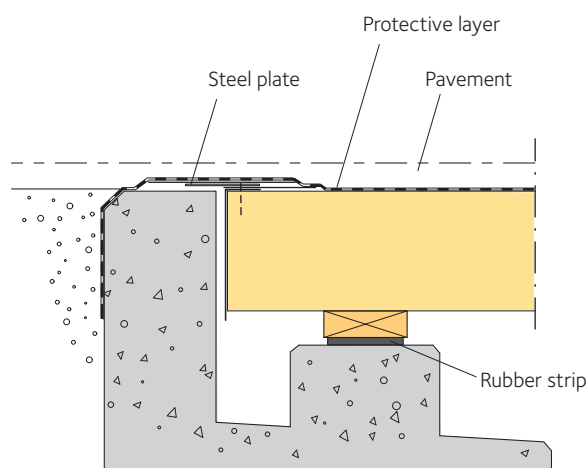


Figure 9.15: End support of timber deck.

9.2 Other exterior applications

Except for timber bridges there are a number of other applications of exterior wooden structures in the infrastructure, for example noise barriers, vehicle barriers, utility poles, lamp posts, bike shelters, bus shelters and street furniture. Some of these are described below.

9.2.1 Noise barriers

Noise is disturbing and unwanted sound. A noise barrier is an exterior structure designed to reduce the spreading of noise from roads, railways, or industrial sources of disturbing sound. Traffic noise from roads consists of sound from tires, vehicle engines, transmission systems etc. At speeds above 50 km/h, tyre noise is the dominating source of noise and it varies by vehicle type and speed. Other methods of reducing the noise include improving automobile aerodynamics and tyre design, and choosing low-noise paving material. Despite this work on noise reduction at the source and also the use of noise reducing windows and buildings, many people who live along and near main highways are exposed to high noise levels.

Noise barriers can be efficient means for noise reduction. Location, topography, cost and aesthetics are of additional importance in the final choice of a noise barrier. Normally, the benefits of noise reduction far outweigh aesthetic impacts for residents protected from unwanted sound. These benefits include lessened sleep disturbance, improved ability to enjoy outdoor life, reduced speech interference and reduced stress, as noise can give rise to physiological changes and various health problems. Disadvantages of noise barriers include aesthetic impacts for motorists and neighbours (particularly if attractive views are blocked), costs for design, construction, and maintenance and also necessity to design special solutions for drainage that the barrier might interrupt.

When sound waves spread from a sound source the noise level decreases with increasing distance from the source. Sound can be reflected, refracted or absorbed. The sound is reflected when the sound wave hits a hard surface. When a barrier is placed in a sound field the noise decreases behind the barrier, see Figure 9.16. The decrease is mainly due to the height of the noise barrier. Higher barriers than 3 – 4 meters are not being built, mostly for practical reasons. The noise shadow behind the barrier is not complete, since some of the sound will be

spread over the edge. The dampening effect of the barrier is greatest immediately behind it, and decreases as the distance to the barrier increases.

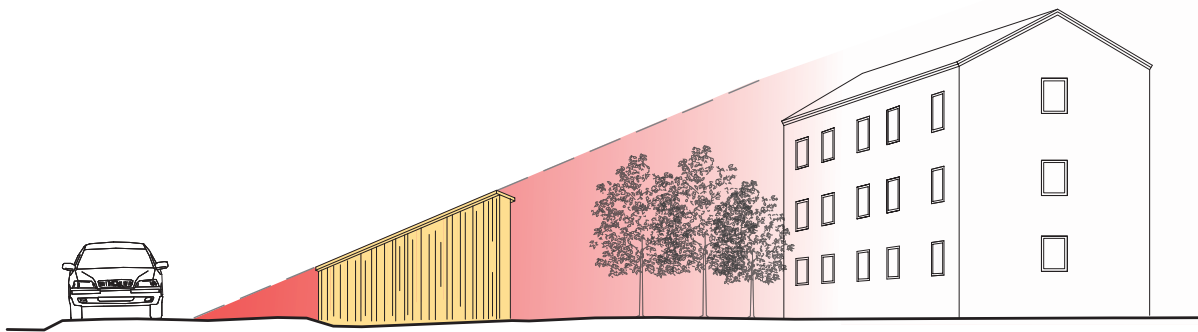


Figure 9.16: Noise barrier.

Noise that is reflected by noise barriers raises the noise level on the opposite side of the road. Sound absorbing materials such as mineral wool on the barrier can reduce the effects of this reflected sound. It is used especially in urban areas. A noise barrier must be completely airtight. Cracks or gaps in the barrier reduce the sound insulation. The barrier should also fit tightly to the ground. A few centimetres gap between the barrier and the ground can remove half the noise reduction or more. The openings for the passage of road users should be designed with overlap.

To be efficient the noise barrier must also have the ability to isolate the airborne sound. Noise insulation for a barrier should be at least 20 – 25 dB. This is achieved through good woodwork and a noise barrier with double panels.

Loads on noise barriers are wind load, dead load, snow load, snow clearance effects, and impact of vehicles. Wind loads on free-standing walls should according to Eurocode SS-EN 1991-1-4 be specified for the zones A, B, C and D, see Figure 9.17. A wall with a return corner will get a reduced value of the pressure coefficient in zones A-C. In zone D the pressure coefficient is always 1,2.

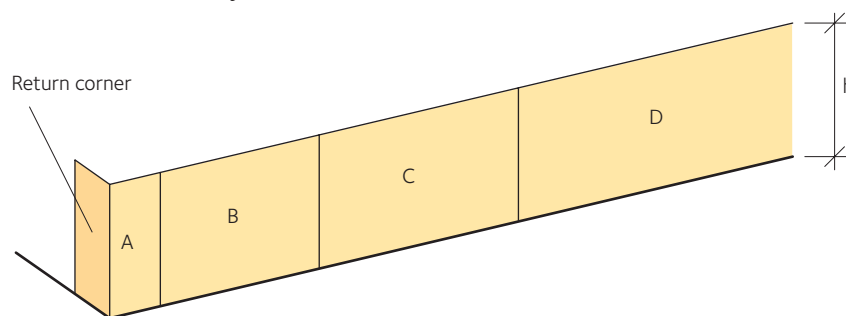


Figure 9.17: Wind load on noise barrier (highest load in zone A near the end).

Noise barriers made of wood can be constructed as walls. The noise barrier is built up with posts at 2 – 5 m distance and between the posts there is a single or double panel. Wood in contact with the ground should be resistant or pressure-treated wood. The posts of the noise barrier must be anchored to the ground. Ground and foundation work stands for a large share of the cost for noise barriers. Therefore, the distance between the foundation points should be as large as possible. There is however a need for finding a reasonable combination of number

of support points and robustness of the intermediate panels. The greater the distance between the posts, the longer and thereby more robust the panel elements have to be to stand wind loads. The foundation can be made with concrete blocks or piles. Frost damages of the foundation should be prevented, due to the risk of damages on posts and connections.

Due to exposure to splashing of water from the ground, the lower part of the barrier close to the support points run the biggest risk of damages. Vegetation adjacent to the barrier increases the risk of damages. The panel should preferably be at least at a distance of 300 mm from the ground, or at least 100 mm if there is an effective splash guard. The wall must fit tightly to the ground and the connections should be able to take up displacements in the ground.

Material selection for noise barriers is important to make them fit in with the surroundings and with existing buildings. Variation of materials along the barrier may help to create experience of variety along a long and monotonous wall. Also solutions based on glass may reduce the effect of a dominant wall, see Figure 9.18, and vegetation along the noise barriers may be included as part of the design. Noise barriers along a road should be built with two fronts, one towards the road and one towards residents. The road front should be varied, but given a whole and large scale so that the drivers are not distracted. Drivers move fast and do not experience the details of the design. Detailed design is however important regarding the resident front where people move slowly and will get close the barrier.

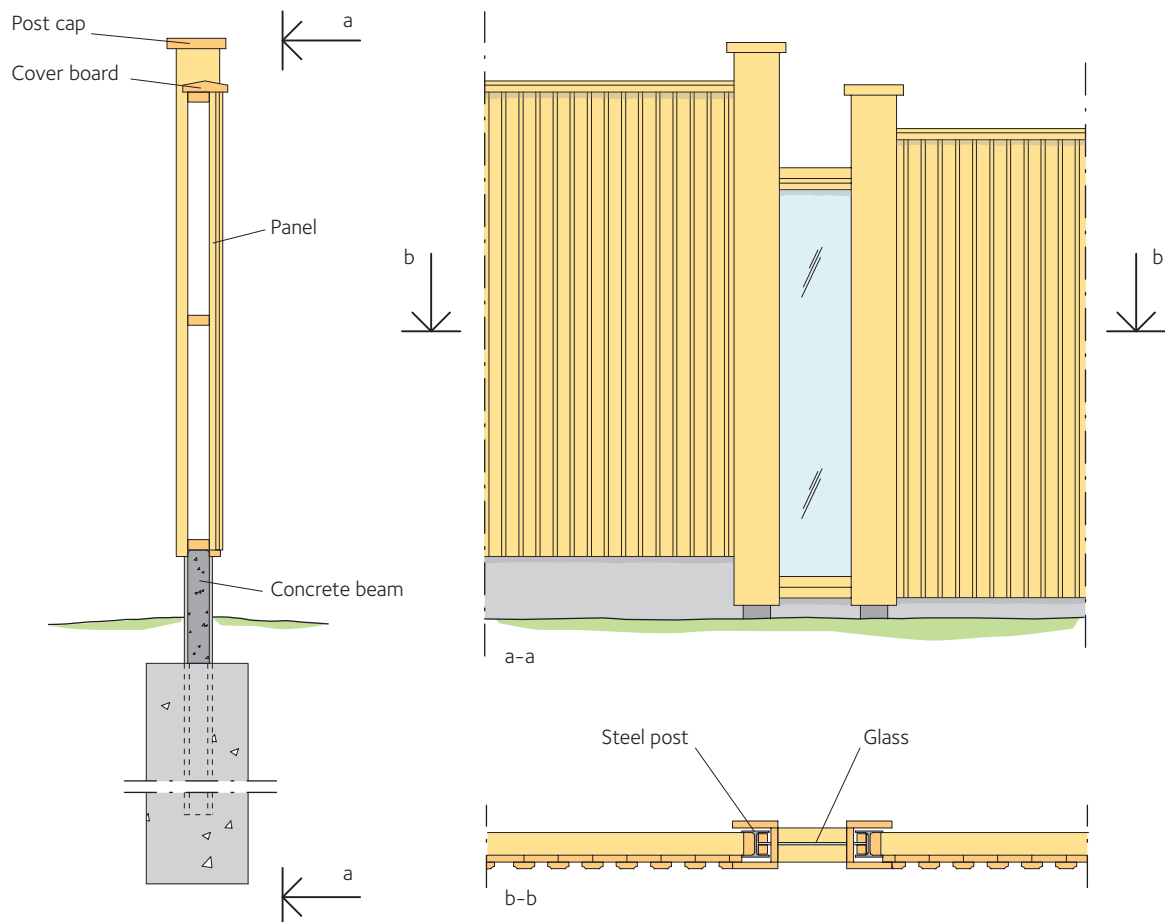


Figure 9.18: Noise barrier with glass part.

9.2.2 Vehicle barriers

Barrier design depends on the intended use of an area. Dimensions and materials are chosen depending on the requirements and the surrounding environment. Some barriers keep vehicles within defined spaces such as a road, or specific parking areas. Vehicle barriers can be divided into different types such as parapets, bollards, fences, and wooden guard-rails.

Barriers and parapets in car parking areas should be designed to resist the required horizontal loads to withstand the impact of a vehicle. The horizontal load is normal to and uniformly distributed over the length of the barrier. The force is calculated based the mass and velocity of the vehicle and can be assumed to act at bumper height.

Wooden guardrails are series of low posts tied together by wooden rails. Guardrails block and control vehicular access, and are used to line roads. They should be designed to resist the required horizontal loads of a vehicle. A fence is a structure functioning as a boundary or barrier, usually made of posts, boards, wire or rails. The purpose is to keep people in or out of an area, and fences are not designed to withstand vehicular impact.

A bollard is a post with no stringer or rail, commonly 0,5 to 1,2 m tall, and used one by one or grouped. The purpose is to block vehicle entry and they are used to define boundaries e.g. around a parking lot, outside buildings, or at a pedestrian entrance. When bollards define a pedestrian entrance, they should be at least 0,9 m apart to allow pedestrian and wheelchair passage. They can be more closely spaced when they define a boundary. Using bollards provided with lightning, creates lit up zones revealing details in the immediate surroundings and can provide guiding lights for the safety of pedestrians and motorists.

9.3 Design for durability, wood protection

Serviceability, working life and durability is often included in the requirements for exterior wood applications. Durability is the property of service during a given time without losing defined functions. Economically reasonable working life is the time during which the performance will be maintained at a level compatible with the requirements when all relevant aspects are taken into account. Considered aspects comprise costs of design, construction and use, as well as inspections, maintenance and environmental aspects. They also include consequences of failure and costs for repair, partial renewal and disposal.

The durability of outdoor timber structures is affected by precipitation, wind and solar radiation. The effects of direct weathering of structural timber members can be reduced by structural protection, or by using timber with sufficient natural durability, or timber treated with preservatives against biological attacks. Exterior structures of wood should be designed for optimal durability when selecting for example bridge type, appropriate design of details, dimensions, joints, and treatments including coatings, which may influence the durability. Decay fungi are the main threat, and the most important issue is to keep the moisture content of the timber below critical levels. The growth of fungi in wood depends on the humidity and if the moisture content is below 20 % no growth will occur. The interaction between materials, design, construction and maintenance is important. Materials and treatments should be selected based on requirements of durability and service life, including the risk of decay fungi, and the ability to inspect, maintain and repair.

Most critical in the design of details is the influence of moisture on timber-based materials, and this can be minimized mainly by:

- Minimizing moisture penetration by efficient water shelter, appropriate cladding, end grain protection, avoiding exposure of end joints, using fastener types minimizing the risk of cracks, ensuring appropriate distance between timber parts and the ground, appropriate surface coatings or other treatments.
- Ensuring rapid drying if the material takes up excessive moisture, using diffusion open treatments, avoiding narrow gaps in joints, avoiding vegetation close to the exposed structure.

Covering of the structural elements will protect the structure from rain, for example a roof on a bridge or a cladding on an arch, as shown by Kleppe (2010). Where a partial or complete covering of the main structural elements is not feasible, durability can be improved through one or more of the following measures:

- Limiting standing water on timber surfaces through appropriate inclination of surfaces.
- Limiting openings, slots, etc., where water may accumulate or infiltrate.
- Limiting direct absorption of water (for example capillary absorption from concrete foundation) through use of appropriate barriers.
- Limiting fissures and delaminations, especially at locations where the end grain would be exposed, by appropriate sealing and/or cover plates.
- Limiting swelling and shrinking movements by ensuring an appropriate initial moisture content and by reducing in-service moisture changes through adequate surface protection.
- Selecting a geometry for the structure that ensures natural ventilation of all timber parts.

The risk of increased moisture content close to the ground can be reduced by:

- Covering of the ground with coarse gravel or similar to limit vegetation.
- Using an increased distance between the timber parts and the ground level.

Resistance to corrosion is important. An example of especially corrosive conditions is a timber bridge where corrosive de-icing cannot be excluded. The risk of stress corrosion should be taken into account. The effect of chemical treatment of timber, or timber with high acidic content, on the corrosion protection of fasteners should be taken into account. For steel parts and fasteners, galvanization is sufficient in most cases but stainless steel is also used.

Treated timber can provide protection against rot and/or insects. Pressure-treated wood is typically used when there is a risk of long-lasting moisture impact, when safety measures are important, structural elements are difficult to replace or when the wood is located close to the ground. Pressure-treated wood shall be manufactured according to European standards.

Some countries such as Norway prescribe 100 years of service life for bridges. In Norway the wood in bridges is often impregnated using chemicals to provide sufficiently long durability. Two methods are used, either copper salts or creosote or a combination of both. A double impregnation using salt and creosote is common, as described by Ekeberg (2010). Each lamella is pressure-treated with salt solution before planing and gluing. Then the entire structural member with holes etc. is given a pressure treatment with creosote oil. This double impregnation has been applied for arches and trusses. Stress-laminated deck plates protected by a water-tight membrane are usually only impregnated with creosote oil.

Sweden and several other countries have prohibited the usage of chemical treatments containing creosote due to it being considered more or less harmful to nature. Therefore the design is very important as well as the design of details and the maintenance. Sweden has requirements concerning wooden bridges with technical service life of 40 and 80 years respectively. The difference between these cases is that extensive sheltering is required and a maintenance plan has to be prepared for technical service life of 80 years. Impregnation with copper salts is permitted for exposed members, but bridge decks and other structural components that are protected by sheltering can be made of softwood without any chemical treatment or by other wood species with acceptable natural durability.

9.3.1 Inspections and maintenance

Wood protection and maintenance should be considered during planning since the reason for decay is usually poor detailing and/or neglected maintenance. Regular maintenance is needed to ensure a long life of outdoor structures of any material, timber as well as other materials. This includes all measures to ensure proper performance of the structure to prevent damage from weathering and decaying organisms. For bridges, it is usually required that they should be inspected regularly to provide a basis for assessing their condition and function, and the planning of future actions. Timber bridges are inspected in a similar way compared to other bridges, but the methods and measurements are adapted to the material performance characteristics of wood, as described in Pousette et al. (2006).

Thorough cleaning is recommended once a year to remove dirt like soil and/or leaves from the structure. Openings should be kept free for efficient drainage. Coated surfaces can be washed with clean water and the appearance and function of components of the structure should be checked. It is important to note the signs of decay in wood components, observable as for example very soft spots etc.

Maintenance painting should be carried out after cleaning and/or sanding. Usually the heavily exposed surfaces such as the top of handrails need more frequent maintenance painting than vertical surfaces. If there is an excessive thickness developed or cracking and flaking occurs after several painting interventions, the structure requires a total renewal of the paint layer.

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Symbols

Symbols used in SS-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
$A_{\text{net,t}}$	Net cross-sectional area perpendicular to the grain
$A_{\text{net,v}}$	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
$E_{\text{mean,fin}}$	Final mean value of modulus of elasticity
F	Force
$F_{A,\text{Ed}}$	Design force acting on a punched metal plate fastener at the centroid of the effective area
$F_{A,\text{min,d}}$	Minimum design force acting on a punched metal plate fastener at the centroid of the effective area
$F_{\text{ax,Ed}}$	Design axial force on fastener
$F_{\text{ax,Rd}}$	Design value of axial withdrawal capacity of the fastener
$F_{\text{ax,Rk}}$	Characteristic axial withdrawal capacity of the fastener
F_c	Compressive force
F_d	Design force
$F_{d,\text{ser}}$	Design force at the serviceability limit state
$F_{f,\text{Rd}}$	Design load-carrying capacity per fastener in wall diaphragm
$F_{i,c,\text{Ed}}$	Design compressive reaction force at end of shear wall
$F_{i,t,\text{Ed}}$	Design tensile reaction force at end of shear wall
$F_{i,\text{vert,Ed}}$	Vertical load on wall
$F_{i,v,\text{Rd}}$	Design racking resistance of panel i or wall i
$F_{i\alpha}$	Lateral load
$F_{M,\text{Ed}}$	Design force from a design moment
F_t	Tensile force
$F_{t,\text{Rk}}$	Characteristic tensile capacity of a connection
$F_{V,0,\text{Rk}}$	Characteristic load-carrying capacity of a connector along the grain
$F_{V,\text{Ed}}$	Design shear force per shear plane of fastener; Horizontal design effect on wall diaphragm

$F_{V,\text{Rd}}$	Design load-carrying capacity per shear plane per fastener; Design racking load capacity
$F_{V,\text{Rk}}$	Characteristic load-carrying capacity per shear plane per fastener
$F_{V,w,\text{Ed}}$	Design shear force acting on web
$F_{x,\text{Ed}}$	Design value of a force in x -direction
$F_{y,\text{Ed}}$	Design value of a force in y -direction
$F_{x,\text{Rd}}$	Design value of plate capacity in x -direction
$F_{y,\text{Rd}}$	Design value of plate capacity in y -direction
$F_{x,\text{Rk}}$	Characteristic plate capacity in x -direction
$F_{y,\text{Rk}}$	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
$K_{\text{ser,fin}}$	Final slip modulus
K_u	Instantaneous slip modulus for ultimate limit states
$L_{\text{net,t}}$	Net width of the cross-section perpendicular to the grain
$L_{\text{net,v}}$	Net length of the fracture area in shear
$M_{A,\text{Ed}}$	Design moment acting on a punched metal plate fastener
$M_{\text{ap,d}}$	Design moment at apex zone
M_d	Design moment
$M_{y,\text{Rk}}$	Characteristic yield moment of fastener
N	Axial force
$R_{90,d}$	Design splitting capacity
$R_{90,k}$	Characteristic splitting capacity
$R_{\text{ax,d}}$	Design load-carrying capacity of an axially loaded connection
$R_{\text{ax,k}}$	Characteristic load-carrying capacity of an axially loaded connection
$R_{\text{ax},\alpha,k}$	Characteristic load-carrying capacity at an angle α to grain
R_d	Design value of a load-carrying capacity
$R_{\text{ef,k}}$	Effective characteristic load-carrying capacity of a connection
$R_{iv,d}$	Design racking capacity of a wall
R_k	Characteristic load-carrying capacity
$R_{\text{sp,k}}$	Characteristic splitting capacity

Source: SS-EN 1995-1-1:2004, 1.6

$R_{t0,k}$	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 $\alpha = 0^\circ$ och $\beta = 0^\circ$
$f_{a,90,90}$	Characteristic anchorage capacity per unit area for $\alpha = 90^\circ$ och $\beta = 90^\circ$
$f_{a,\alpha,\beta,k}$	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
$f_{r,t,d}$	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
$f_{v,\alpha,\alpha,k}$	Characteristic withdrawal strength at an angle α to grain
$f_{v,\alpha,\alpha,90,k}$	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

Source: SS-EN 1995-1-1:2004, 1.6

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_f	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 eller 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_l	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

Source: SS-EN 1995-1-1:2004, 1.6

$\sigma_{w,c,d}$	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: SS-EN 1995-1-1:2004, 1.6

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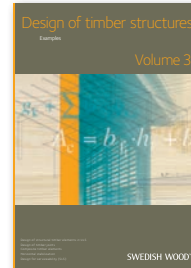
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