

# *The European Union*

## EDICT OF GOVERNMENT

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EN 1995-1-1 (2004) (English): Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]



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English version

## Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings

Eurocode 5: Conception et calcul des structures en bois -  
Partie 1-1 : Généralités - Règles communes et règles pour  
les bâtiments

Eurocode 5: Bemessung und Konstruktion von Holzbauten  
- Teil 1-1: Allgemeines - Allgemeine Regeln und Regeln für  
den Hochbau

This European Standard was approved by CEN on 16 April 2004.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Central Secretariat or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Central Secretariat has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.



EUROPEAN COMMITTEE FOR STANDARDIZATION  
COMITÉ EUROPÉEN DE NORMALISATION  
EUROPÄISCHES KOMITEE FÜR NORMUNG

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## Foreword

This European Standard EN 1995-1-1 has been prepared by Technical Committee CEN/TC250 "Structural Eurocodes", the Secretariat of which is held by BSI.

This European Standard shall be given the status of a National Standard, either by publication of an identical text or by endorsement, at the latest by May 2005, and conflicting national standards shall be withdrawn at the latest by March 2010.

This European Standard supersedes ENV 1995-1-1:1993.

CEN/TC250 is responsible for all Structural Eurocodes.

According to the CEN/CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxemburg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

## Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement<sup>1</sup> between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links de facto the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products – CPD – and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990:2002	Eurocode: Basis of Structural Design
EN 1991	Eurocode 1: Actions on structures
EN 1992	Eurocode 2: Design of concrete structures
EN 1993	Eurocode 3: Design of steel structures
EN 1994	Eurocode 4: Design of composite steel and concrete structures
EN 1995	Eurocode 5: Design of timber structures
EN 1996	Eurocode 6: Design of masonry structures
EN 1997	Eurocode 7: Geotechnical design

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<sup>1</sup> Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

EN 1998	Eurocode 8: Design of structures for earthquake resistance
EN 1999	Eurocode 9: Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

### **Status and field of application of Eurocodes**

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire ;
- as a basis for specifying contracts for construction works and related engineering services ;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents<sup>2</sup> referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards<sup>3</sup>. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

### **National Standards implementing Eurocodes**

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex.

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e.:

- values and/or classes where alternatives are given in the Eurocode;
- values to be used where a symbol only is given in the Eurocode;
- country specific data (geographical, climatic, etc.), e.g. snow map;

<sup>2</sup> According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

<sup>3</sup> According to Art. 12 of the CPD the interpretative documents shall:  
give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;  
indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;  
serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

- the procedure to be used where alternative procedures are given in the Eurocode;
- decisions on the application of informative annexes;
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

### **Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products**

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works<sup>4</sup>. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

### **Additional information specific to EN 1995-1-1**

EN 1995 describes the Principles and requirements for safety, serviceability and durability of timber structures. It is based on the limit state concept used in conjunction with a partial factor method.

For the design of new structures, EN 1995 is intended to be used, for direct application, together with EN 1990:2002 and relevant Parts of EN 1991.

Numerical values for partial factors and other reliability parameters are recommended as basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and of quality management applies. When EN 1995-1-1 is used as a base document by other CEN/TCs the same values need to be taken.

### **National annex for EN 1995-1-1**

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1995-1-1 should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1995-1-1 through clauses:

2.3.1.2(2)P	Assignment of loads to load-duration classes;
2.3.1.3(1)P	Assignment of structures to service classes;
2.4.1(1)P	Partial factors for material properties;
6.1.7(2)	Shear;
6.4.3(8)	Double tapered, curved and pitched cambered beams;
7.2(2)	Limiting values for deflections;
7.3.3(2)	Limiting values for vibrations;
8.3.1.2(4)	Nailed timber-to-timber connections: Rules for nails in end grain;
8.3.1.2(7)	Nailed timber-to-timber connections: Species sensitive to splitting;
9.2.4.1(7)	Design method for wall diaphragms;
9.2.5.3(1)	Bracing modification factors for beam or truss systems;
10.9.2(3)	Erection of trusses with punched metal plate fasteners: Maximum bow;
10.9.2(4)	Erection of trusses with punched metal plate fasteners: Maximum deviation.

## **Foreword to amendment A1**

This document (EN 1995-1-1:2004/A1:2008) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI.

This Amendment to the European Standard EN 1995-1-1:2004 shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by December 2008, and conflicting national standards shall be withdrawn at the latest by March 2010.

According to the CEN/CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Bulgaria, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

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<sup>4</sup> see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

## **Section 1    General**

### **1.1    Scope**

#### **1.1.1    Scope of EN 1995**

(1)P EN 1995 applies to the design of buildings and civil engineering works in timber (solid timber, sawn, planed or in pole form, glued laminated timber or wood-based structural products, e.g. LVL) or wood-based panels jointed together with adhesives or mechanical fasteners. It complies with the principles and requirements for the safety and serviceability of structures and the basis of design and verification given in EN 1990:2002.

(2)P EN 1995 is only concerned with requirements for mechanical resistance, serviceability, durability and fire resistance of timber structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(3) EN 1995 is intended to be used in conjunction with:  
EN 1990:2002 Eurocode – Basis of design  
EN 1991 “Actions on structures”  
EN’s for construction products relevant to timber structures  
EN 1998 “Design of structures for earthquake resistance”, when timber structures are built in seismic regions

(4) EN 1995 is subdivided into various parts:  
EN 1995-1    General  
EN 1995-2    Bridges

(5) EN 1995-1 “General” comprises:  
EN 1995-1-1    General – Common rules and rules for buildings  
EN 1995-1-2    General rules – Structural Fire Design

(6) EN 1995-2 refers to the common rules in EN 1995-1-1. The clauses in EN 1995-2 supplement the clauses in EN 1995-1.

#### **1.1.2    Scope of EN 1995-1-1**

(1) EN 1995-1-1 gives general design rules for timber structures together with specific design rules for buildings.

(2) The following subjects are dealt with in EN 1995-1-1:

- Section 1:    General
- Section 2:    Basis of design
- Section 3:    Material properties
- Section 4:    Durability

- Section 5: Basis of structural analysis
- Section 6: Ultimate limit states
- Section 7: Serviceability limit states
- Section 8: Connections with metal fasteners
- Section 9: Components and assemblies
- Section 10: Structural detailing and control.

(3)P EN 1995-1-1 does not cover the design of structures subject to prolonged exposure to temperatures over 60°C.

## 1.2 Normative references



**[A1]** (1) This European Standard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

ISO standards:


- |                 |                                                                                                                                 |
|-----------------|---------------------------------------------------------------------------------------------------------------------------------|
| ISO 2081        | Metallic coatings. Electroplated coatings of zinc on iron or steel                                                              |
| ISO 2631-2:1989 | Evaluation of human exposure to whole-body vibration. Part 2: Continuous and shock-induced vibrations in buildings (1 to 80 Hz) |

European Standards:

- |          |                                                                                                                                                                                       |
|----------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| EN 300   | Oriented Strand Board (OSB) – Definition, classification and specifications                                                                                                           |
| EN 301   | Adhesives, phenolic and aminoplastic for load-bearing timber structures; Classification and performance requirements                                                                  |
| EN 312   | Particleboards – Specifications                                                                                                                                                       |
| EN 335-1 | Durability of wood and wood-based products – definition of hazard classes of biological attack – Part 1: General                                                                      |
| EN 335-2 | Durability of wood and wood-based products – definition of hazard classes of biological attack – Part 2: Application to solid wood                                                    |
| EN 335-3 | Durability of wood and wood-based products – Definition of hazard classes of biological attack – Part 3: Application to wood-based panels                                             |
| EN 350-2 | Durability of wood and wood-based products – Natural durability of solid wood – Part 2: Guide to natural durability and treatability of selected wood species of importance in Europe |
| EN 351-1 | Durability of wood and wood-based products – Preservative treated solid wood – Part 1: Classification of preservative penetration and retention                                       |
| EN 383   | Timber structures – Test methods – Determination of embedding strength and foundation values for dowel type fasteners                                                                 |
| EN 385   | Finger jointed structural timber – Performance requirements and minimum production requirements                                                                                       |
| EN 387   | Glued laminated timber – Large finger joints – Performance requirements and minimum production requirements                                                                           |
| EN 409   | Timber structures – Test methods. Determination of the yield moment of dowel type fasteners – Nails <b>[A1]</b>                                                                       |

 EN 460	Durability of wood and wood-based products – Natural durability of solid wood – Guide of the durability requirements for wood to be used in hazard classes
EN 594	Timber structures – Test methods – Racking strength and stiffness of timber frame wall panels
EN 622-2	Fibreboards – Specifications. Part 2: Requirements for hardboards
EN 622-3	Fibreboards – Specifications. Part 3: Requirements for medium boards
EN 622-4	Fibreboards – Specifications. Part 4: Requirements for softboards
EN 622-5	Fibreboards – Specifications. Part 5: Requirements for dry process boards (MDF)
EN 636	Plywood – Specifications
EN 912	Timber fasteners – Specifications for connectors for timber
EN 1075	Timber structures – Test methods – Testing of joints made with punched metal plate fasteners
EN 1380	Timber structures – Test methods – Load bearing nailed joints
EN 1381	Timber structures – Test methods – Load bearing stapled joints
EN 1382	Timber structures – Test methods – Withdrawal capacity of timber fasteners
EN 1383	Timber structures – Test methods – Pull through testing of timber fasteners
EN 1990:2002	Eurocode – Basis of structural design
EN 1991-1-1	Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight and imposed loads
EN 1991-1-3	Eurocode 1: Actions on structures – Part 1-3: General actions – Snow loads
EN 1991-1-4	Eurocode 1: Actions on structures – Part 1-4: General actions – Wind loads
EN 1991-1-5	Eurocode 1: Actions on structures – Part 1-5: General actions – Thermal actions
EN 1991-1-6	Eurocode 1: Actions on structures – Part 1-6: General actions – Actions during execution
EN 1991-1-7	Eurocode 1: Actions on structures – Part 1-7: General actions – Accidental actions due to impact and explosions
EN 10147	Specification for continuously hot-dip zinc coated structural steel sheet and strip – Technical delivery conditions
EN 13271	Timber fasteners – Characteristic load-carrying capacities and slip moduli for connector joints
EN 13986	Wood-based panels for use in construction – Characteristics, evaluation of conformity and marking
EN 14080	Timber structures – Glued laminated timber – Requirements
EN 14081-1	Timber structures – Strength graded structural timber with rectangular cross-section – Part 1, General requirements
EN 14250	Timber structures – Production requirements for fabricated trusses using punched metal plate fasteners
EN 14279	Laminated veneer lumber (LVL) – Specifications, definitions, classification and requirements 

EN 14358	Timber structures – Fasteners and wood-based products – Calculation of characteristic 5-percentile value and acceptance criteria for a sample
EN 14374	Timber structures – Structural laminated veneer lumber – Requirements
EN 14545	Timber structures – Connectors – Requirements
EN 14592	Timber structures – Fasteners – Requirements
EN 26891	Timber structures – Joints made with mechanical fasteners – General principles for the determination of strength and deformation characteristics
EN 28970	Timber structures – Testing of joints made with mechanical fasteners; Requirements for wood density (ISO 8970:1989)

NOTE: As long as EN 14545 and EN 14592 are not available as European standards, more information may be given in the National annex. 

### 1.3 Assumptions

(1)P The general assumptions of EN 1990:2002 apply.

(2) Additional requirements for structural detailing and control are given in section 10.

### 1.4 Distinction between Principles and Application Rules

(1)P The rules in EN 1990:2002 clause 1.4 apply.

### 1.5 Terms and definitions

#### 1.5.1 General

(1)P The terms and definitions of EN 1990:2002 clause 1.5 apply.

#### 1.5.2 Additional terms and definitions used in this present standard

##### 1.5.2.1

##### Characteristic value

Refer to EN 1990:2002 subclause 1.5.4.1.

##### 1.5.2.2

##### Dowelled connection

Connection made with a circular cylindrical rod usually of steel, with or without a head, fitting tightly in prebored holes and used for transferring loads perpendicular to the dowel axis.

##### 1.5.2.3

##### Equilibrium moisture content

The moisture content at which wood neither gains nor loses moisture to the surrounding air.

##### 1.5.2.4

##### Fibre saturation point

Moisture content at which the wood cells are completely saturated.

#### 1.5.2.5

##### LVL

Laminated veneer lumber, defined according to EN 14279 and EN 14374

#### 1.5.2.6

##### Laminated timber deck

A plate made of abutting parallel and solid laminations connected together by nails or screws or prestressing or gluing.

#### 1.5.2.7

##### Moisture content

The mass of water in wood expressed as a proportion of its oven-dry mass.

#### 1.5.2.8

##### Racking

Effect caused by horizontal actions in the plane of a wall.

#### 1.5.2.9

##### Stiffness property

A property used in the calculation of the deformation of the structure, such as modulus of elasticity, shear modulus, slip modulus.

#### 1.5.2.10

##### Slip modulus

A property used in the calculation of the deformation between two members of a structure.

### 1.6 Symbols used in EN 1995-1-1

For the purpose of EN 1995-1-1, the following symbols apply.

*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 contact area in compression perpendicular to the grain $A_1$
$A_f$	Cross-sectional area of flange
$A_{net,t}$	Net cross-sectional area perpendicular to the grain
$A_{net,v}$	Net shear area parallel to the grain
$C$	Spring stiffness
$E_{0,05}$	Fifth percentile value of modulus of elasticity;
$E_d$	Design value of modulus of elasticity;
$E_{mean}$	Mean value of modulus of elasticity;
$E_{mean,fin}$	Final mean value of modulus of elasticity;
$F$	Force
$F_{A,Ed}$	Design force acting on a punched metal plate fastener at the centroid of the effective area
$F_{A,min,d}$	Minimum design force acting on a punched metal plate fastener at the centroid of the effective area
$F_{ax,Ed}$	Design axial force on fastener;
$F_{ax,Rd}$	Design value of axial withdrawal capacity of the fastener;
$F_{ax,Rk}$	Characteristic axial withdrawal capacity of the fastener;
$F_c$	Compressive force

$F_d$	Design force
$F_{d,ser}$	Design force at the serviceability limit state
$F_{f,Rd}$	Design load-carrying capacity per fastener in wall diaphragm
$F_{i,c,Ed}$	Design compressive reaction force at end of shear wall
$F_{i,t,Ed}$	Design tensile reaction force at end of shear wall
$F_{i,vert,Ed}$	Vertical load on wall
$F_{i,v,Rd}$	Design racking resistance of panel $i$ (in 9.2.4.2) or wall $i$ (in 9.2.4.3)
$F_{la}$	Lateral load
$F_{M,Ed}$	Design force from a design moment
$F_t$	Tensile force
$F_{t,Rk}$	Characteristic tensile resistance of connection $A_1$
$F_{v,0,Rk}$	Characteristic load-carrying capacity of a connector along the grain;
$F_{v,Ed}$	Design shear force per shear plane of fastener; Horizontal design effect on wall diaphragm
$F_{v,Rd}$	Design load-carrying capacity per shear plane per fastener; Design racking load capacity
$F_{v,Rk}$	Characteristic load-carrying capacity per shear plane per fastener
$F_{v,w,Ed}$	Design shear force acting on web;
$F_{x,Ed}$	Design value of a force in x-direction
$F_{y,Ed}$	Design value of a force in y-direction
$F_{x,Rd}$	Design value of plate capacity in x-direction;
$F_{y,Rd}$	Design value of plate capacity in y-direction;
$F_{x,Rk}$	Characteristic plate capacity in x-direction;
$F_{y,Rk}$	Characteristic plate capacity in y-direction;
$G_{0,05}$	Fifth percentile value of shear modulus
$G_d$	Design value of shear modulus
$G_{mean}$	Mean value of shear modulus
$H$	Overall rise of a truss
$I_f$	Second moment of area of flange
$I_{tor}$	Torsional moment of inertia
$I_z$	Second moment of area about the weak axis
$K_{ser}$	Slip modulus
$K_{ser,fin}$	Final slip modulus
$K_u$	Instantaneous slip modulus for ultimate limit states
$L_{net,t}$	Net width of the cross-section perpendicular to the grain
$L_{net,v}$	Net length of the fracture area in shear
$M_{A,Ed}$	Design moment acting on a punched metal plate fastener
$M_{ap,d}$	Design moment at apex zone
$M_d$	Design moment
$M_{y,Rk}$	Characteristic yield moment of fastener
$N$	Axial force
$R_{90,d}$	Design splitting capacity
$R_{90,k}$	Characteristic splitting capacity
$R_{ax,d}$	Design load-carrying capacity of an axially loaded connection
$R_{ax,k}$	Characteristic load-carrying capacity of an axially loaded connection
$R_{ax,\alpha,k}$	Characteristic load-carrying capacity at an angle to grain
$R_d$	Design value of a load-carrying capacity
$R_{ef,k}$	Effective characteristic load-carrying capacity of a connection
$R_{iv,d}$	Design racking capacity of a wall
$R_k$	Characteristic load-carrying capacity
$R_{sp,k}$	Characteristic splitting capacity
$R_{to,k}$	Characteristic load-carrying capacity of a toothed plate connector
$R_{v,d}$	Design racking capacity of a wall diaphragm
$V$	Shear force; Volume
$V_u, V_l$	Shear forces in upper and lower part of beam with a hole
$W_y$	Section modulus about axis $y$
$X_d$	Design value of a strength property
$X_k$	Characteristic value of a strength property

*Latin lower case letters*

$a$	Distance
$a_1$	Spacing, parallel to grain, of fasteners within one row
$\overline{A_1} a_{1,CG}$	End distance of centre of gravity of the threaded part of screw in the member $\overline{A_1}$
$a_2$	Spacing, perpendicular to grain, between rows of fasteners
$\overline{A_1} a_{2,CG}$	Edge distance of centre of gravity of the threaded part of screw in the member $\overline{A_1}$
$a_{3,c}$	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 $i$ (in 9.2.4.2) or wall $i$ (in 9.2.4.3)
$b_{net}$	Clear distance between studs
$b_w$	Web width
$\overline{A_1} d$	Diameter; Outer thread diameter
$d_1$	Inner thread diameter $\overline{A_1}$
$d_c$	Connector diameter
$d_{ef}$	Effective diameter
$\overline{A_1} d_h$	Head diameter of screws $\overline{A_1}$
$f_{h,i,k}$	Characteristic embedment strength of timber member $i$
$f_{a,0,0}$	Characteristic anchorage capacity per unit area for $\alpha = 0^\circ$ and $\beta = 0^\circ$
$f_{a,90,90}$	Characteristic anchorage capacity per unit area for $\alpha = 90^\circ$ and $\beta = 90^\circ$
$f_{a,\alpha,\beta,k}$	Characteristic anchorage strength
$\overline{A_1} f_{ax,k}$	Characteristic pointside withdrawal strength for nails; Characteristic withdrawal strength $\overline{A_1}$
$f_{c,0,d}$	Design compressive strength along the grain
$f_{c,w,d}$	Design compressive strength of web
$f_{f,c,d}$	Design compressive strength of flange
$f_{c,90,k}$	Characteristic compressive strength perpendicular to grain
$f_{f,t,d}$	Design tensile strength of flange
$f_{h,k}$	Characteristic embedment strength
$f_{head,k}$	Characteristic pull through parameter for nails
$f_1$	Fundamental frequency
$f_{m,k}$	Characteristic bending strength
$f_{m,y,d}$	Design bending strength about the principal y-axis
$f_{m,z,d}$	Design bending strength about the principal z-axis
$f_{m,\alpha,d}$	Design bending strength at an angle $\alpha$ 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 to grain
$f_{v,\alpha,\alpha,k}$	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
$h$	Loaded edge distance
$h_{ef}$	Effective depth
$h_{f,c}$	Depth of compression flange

$h_{f,t}$	Depth of tension flange
$h_{rl}$	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
$k_{c,y}$ or $k_{c,z}$	Instability factor
$\sqrt{A_1} k_{cr}$	Crack factor for shear resistance $\sqrt{A_1}$
$k_{crit}$	Factor used for lateral buckling
$k_d$	Dimension factor for panel
$k_{def}$	Deformation factor
$k_{dis}$	Factor taking into account the distribution of stresses in an apex zone
$k_{f,1}, k_{f,2}, k_{f,3}$	Modification factors for bracing resistance
$k_h$	Depth factor
$k_{i,q}$	Uniformly distributed load factor
$k_m$	Factor considering re-distribution of bending stresses in a cross-section
$k_{mod}$	Modification factor for duration of load and moisture content
$k_n$	Sheathing material factor
$k_r$	Reduction factor
$k_{R,red}$	Reduction factor for load-carrying capacity
$k_s$	Fastener spacing factor; Modification factor for spring stiffness
$k_{s,red}$	Reduction factor for spacing
$k_{shape}$	Factor depending on the shape of the cross-section
$k_{sys}$	System strength factor
$k_v$	Reduction factor for notched beams
$k_{vol}$	Volume factor
$k_y$ or $k_z$	Instability factor
$\ell_{a,min}$	Minimum anchorage length for a glued-in rod
$\ell$	Span; contact length
$\ell_A$	Support distance of a hole
$\ell_{ef}$	Effective length; Effective length of distribution
$\ell_V$	Distance from a hole to the end of the member
$\ell_Z$	Spacing between holes
$m$	Mass per unit area
$n_{40}$	Number of frequencies below 40 Hz
$n_{ef}$	Effective number of fasteners
$p_d$	Distributed load
$q_i$	Equivalent uniformly distributed load
$r$	Radius of curvature
$s$	Spacing
$s_0$	Basic fastener spacing
$r_{in}$	Inner radius
$t$	Thickness
$t_{pen}$	Penetration depth
$u_{creep}$	Creep deformation
$u_{fin}$	Final deformation
$u_{fin,G}$	Final deformation for a permanent action $G$
$u_{fin,Q,1}$	Final deformation for the leading variable action $Q_1$
$u_{fin,Q,i}$	Final deformation for accompanying variable actions $Q_i$
$u_{inst}$	Instantaneous deformation
$u_{inst,G}$	Instantaneous deformation for a permanent action $G$
$u_{inst,Q,1}$	Instantaneous deformation for the leading variable action $Q_1$
$u_{inst,Q,i}$	Instantaneous deformation for accompanying variable actions $Q_i$
$w_c$	Precamber
$w_{creep}$	Creep deflection
$w_{fin}$	Final deflection
$w_{inst}$	Instantaneous deflection
$w_{net,fin}$	Net final deflection
$v$	Unit impulse velocity response

*Greek lower case letters*

$\alpha$	Angle between the x-direction and the force for a punched metal plate; Angle between a force and the direction of grain; Angle between the direction of the load and the loaded edge (or end)
$\beta$	Angle between the grain direction and the force for a punched metal plate
$\beta_c$	Straightness factor
$\gamma$	Angle between the x-direction and the timber connection line for a punched metal plate
$\gamma_M$	Partial factor for material properties, also accounting for model uncertainties and dimensional variations
$\lambda_y$	Slenderness ratio corresponding to bending about the y-axis
$\lambda_z$	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
$\overline{\rho_a}$	Associated density $\overline{\rho_a}$
$\rho_k$	Characteristic density
$\rho_m$	Mean density
$\sigma_{c,0,d}$	Design compressive stress along the grain
$\sigma_{c,\alpha,d}$	Design compressive stress at an angle $\alpha$ 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 $\alpha$ to the grain
$\sigma_N$	Axial stress
$\sigma_{t,0,d}$	Design tensile stress along the grain
$\sigma_{t,90,d}$	Design tensile stress perpendicular to the grain
$\sigma_{w,c,d}$	Design compressive stress of web
$\sigma_{w,t,d}$	Design tensile stress of web
$\tau_d$	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
$\psi_0$	Factor for combination value of a variable action
$\psi_2$	Factor for quasi-permanent value of a variable action
$\zeta$	Modal damping ratio

## Section 2 Basis of design

### 2.1 Requirements

#### 2.1.1 Basic requirements

(1)P The design of timber structures shall be in accordance with EN 1990:2002.

(2)P The supplementary provisions for timber structures given in this section shall also be applied.

(3) The basic requirements of EN 1990:2002 section 2 are deemed to be satisfied for timber structures when limit state design, in conjunction with the partial factor method using EN 1990:2002 and EN 1991 for actions and their combinations and EN 1995 for resistances, rules for serviceability and durability, is applied.

#### 2.1.2 Reliability management

(1) When different levels of reliability are required, these levels should be preferably achieved by an appropriate choice of quality management in design and execution, according to EN 1990:2002 Annex C.

#### 2.1.3 Design working life and durability

**A1** (1) EN 1990:2002 clauses 2.3 and 2.4 apply. **A1**

### 2.2 Principles of limit state design

#### 2.2.1 General

(1)P The design models for the different limit states shall, as appropriate, take into account the following:

- different material properties (e.g. strength and stiffness);
- different time-dependent behaviour of the materials (duration of load, creep);
- different climatic conditions (temperature, moisture variations);
- different design situations (stages of construction, change of support conditions).

#### 2.2.2 Ultimate limit states

(1)P The analysis of structures shall be carried out using the following values for stiffness properties:

- for a first order linear elastic analysis of a structure, whose distribution of internal forces is not affected by the stiffness distribution within the structure (eg. all members have the same time-dependent properties), mean values shall be used;
- for a first order linear elastic analysis of a structure, whose distribution of internal forces is affected by the stiffness distribution within the structure (eg. composite members containing materials having different time-dependent properties), final mean values adjusted to the load component causing the largest stress in relation to strength shall be used;
- for a second order linear elastic analysis of a structure, design values, not adjusted for duration of load, shall be used.

NOTE 1: For final mean values adjusted to the duration of load, see 2.3.2.2(2).

NOTE 2: For design values of stiffness properties, see 2.4.1(2)P.

(2) The slip modulus of a connection for the ultimate limit state,  $K_{u,}$  should be taken as:

$$K_u = \frac{2}{3} K_{ser} \quad (2.1)$$

Ⓐ where  $K_{ser}$  is the slip modulus, see 7.1(1). Ⓐ

### 2.2.3 Serviceability limit states

(1)P The deformation of a structure which results from the effects of actions (such as axial and shear forces, bending moments and joint slip) and from moisture shall remain within appropriate limits, having regard to the possibility of damage to surfacing materials, ceilings, floors, partitions and finishes, and to the functional needs as well as any appearance requirements.

(2) The instantaneous deformation,  $u_{inst}$ , see figure 7.1, should be calculated for the characteristic combination of actions, see EN 1990, clause 6.5.3(2) a), using mean values of the appropriate moduli of elasticity, shear moduli and slip moduli.

(3) The final deformation,  $u_{fin}$ , see figure 7.1, should be calculated for the quasi-permanent combination of actions, see EN 1990, clause 6.5.3(2) c).

(4) If the structure consists of members or components having different creep behaviour, the final deformation should be calculated using final mean values of the appropriate moduli of elasticity, shear moduli and slip moduli, according to 2.3.2.2(1).

(5) For structures consisting of members, components and connections with the same creep behaviour and under the assumption of a linear relationship between the actions and the corresponding deformations, as a simplification of 2.2.3(3), the final deformation,  $u_{fin}$ , may be taken as:

$$\text{Ⓐ} \quad u_{fin} = u_{fin,G} + u_{fin,Q_1} + \sum u_{fin,Q_i} \quad \text{Ⓐ} \quad (2.2)$$

where:

$$u_{fin,G} = u_{inst,G} (1 + k_{def}) \quad \text{for a permanent action, } G \quad (2.3)$$

$$u_{fin,Q,1} = u_{inst,Q,1} (1 + \psi_{2,1} k_{def}) \quad \text{for the leading variable action, } Q_1 \quad (2.4)$$

$$u_{fin,Q,i} = u_{inst,Q,i} (\psi_{0,i} + \psi_{2,i} k_{def}) \quad \text{for accompanying variable actions, } Q_i \text{ (} i > 1 \text{)} \quad (2.5)$$

$u_{inst,G}$ ,  $u_{inst,Q,1}$ ,  $u_{inst,Q,i}$  are the instantaneous deformations for action  $G$ ,  $Q_1$ ,  $Q_i$  respectively;

$\psi_{2,1}$ ,  $\psi_{2,i}$  are the factors for the quasi-permanent value of variable actions;

$\psi_{0,i}$  are the factors for the combination value of variable actions;

$k_{def}$  is given in table 3.2 for timber and wood-based materials, and in 2.3.2.2 (3) and 2.3.2.2 (4) for connections.

When expressions (2.3) to (2.5) are used, the  $\psi_2$  factors should be omitted from expressions (6.16a) and (6.16b) of EN1990:2002.

Note: In most cases, it will be appropriate to apply the simplified method.

(6) For serviceability limit states with respect to vibrations, mean values of the appropriate stiffness moduli should be used.

## 2.3 Basic variables

### 2.3.1 Actions and environmental influences

#### 2.3.1.1 General

(1) Actions to be used in design may be obtained from the relevant parts of EN 1991.

Note 1: The relevant parts of EN 1991 for use in design include:

EN 1991-1-1	Densities, self-weight and imposed loads
EN 1991-1-3	Snow loads
EN 1991-1-4	Wind actions
EN 1991-1-5	Thermal actions
EN 1991-1-6	Actions during execution
EN 1991-1-7	Accidental actions

(2)P Duration of load and moisture content affect the strength and stiffness properties of timber and wood-based elements and shall be taken into account in the design for mechanical resistance and serviceability.

(3)P Actions caused by the effects of moisture content changes in the timber shall be taken into account.

#### 2.3.1.2 Load-duration classes

(1)P The load-duration classes are characterised by the effect of a constant load acting for a certain period of time in the life of the structure. For a variable action the appropriate class shall be determined on the basis of an estimate of the typical variation of the load with time.

(2)P Actions shall be assigned to one of the load-duration classes given in Table 2.1 for strength and stiffness calculations.

Table 2.1 – Load-duration classes

Load-duration class	Order of accumulated duration of characteristic load
Permanent	more than 10 years
Long-term	6 months – 10 years
Medium-term	1 week – 6 months
Short-term	less than one week
Instantaneous	

NOTE: Examples of load-duration assignment are given in Table 2.2. Since climatic loads (snow, wind) vary between countries, the assignment of load-duration classes may be specified in the National annex.

**Table 2.2 – Examples of load-duration assignment**

Load-duration class	Examples of loading
Permanent	self-weight
Long-term	storage
Medium-term	imposed floor load, snow
Short-term	snow, wind
Instantaneous	wind, accidental load

### 2.3.1.3 Service classes

(1)P Structures shall be assigned to one of the service classes given below:

NOTE 1: The service class system is mainly aimed at assigning strength values and for calculating deformations under defined environmental conditions.

NOTE 2: Information on the assignment of structures to service classes given in (2)P, (3)P and (4)P may be given in the National annex.

(2)P Service class 1 is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 65 % for a few weeks per year.

NOTE: In service class 1 the average moisture content in most softwoods will not exceed 12 %.

(3)P Service class 2 is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 85 % for a few weeks per year.

NOTE: In service class 2 the average moisture content in most softwoods will not exceed 20 %.

(4)P Service class 3 is characterised by climatic conditions leading to higher moisture contents than in service class 2.

## 2.3.2 Materials and product properties

### 2.3.2.1 Load-duration and moisture influences on strength

(1) Modification factors for the influence of load-duration and moisture content on strength, see 2.4.1, are given in 3.1.3.

(2) Where a connection is constituted of two timber elements having different time-dependent behaviour, the calculation of the design load-carrying capacity should be made with the following modification factor  $k_{\text{mod}}$ :

$$k_{\text{mod}} = \sqrt{k_{\text{mod},1} k_{\text{mod},2}} \quad (2.6)$$

where  $k_{\text{mod},1}$  and  $k_{\text{mod},2}$  are the modification factors for the two timber elements.

### 2.3.2.2 Load-duration and moisture influences on deformations

(1) For serviceability limit states, if the structure consists of members or components having different time-dependent properties, the final mean value of modulus of elasticity,  $E_{\text{mean,fin}}$ , shear modulus  $G_{\text{mean,fin}}$ , and slip modulus,  $K_{\text{ser,fin}}$ , which are used to calculate the final deformation should be taken from the following expressions:

$$E_{\text{mean,fin}} = \frac{E_{\text{mean}}}{(1 + k_{\text{def}})} \quad (2.7)$$

$$G_{\text{mean,fin}} = \frac{G_{\text{mean}}}{(1 + k_{\text{def}})} \quad (2.8)$$

$$K_{\text{ser,fin}} = \frac{K_{\text{ser}}}{(1 + k_{\text{def}})} \quad (2.9)$$

(2) For ultimate limit states, where the distribution of member forces and moments is affected by the stiffness distribution in the structure, the final mean value of modulus of elasticity,  $E_{\text{mean,fin}}$ , shear modulus  $G_{\text{mean,fin}}$ , and slip modulus,  $K_{\text{ser,fin}}$ , should be calculated from the following expressions :

$$E_{\text{mean,fin}} = \frac{E_{\text{mean}}}{(1 + \psi_2 k_{\text{def}})} \quad (2.10)$$

$$G_{\text{mean,fin}} = \frac{G_{\text{mean}}}{(1 + \psi_2 k_{\text{def}})} \quad (2.11)$$

$$K_{\text{ser,fin}} = \frac{K_{\text{ser}}}{(1 + \psi_2 k_{\text{def}})} \quad (2.12)$$

where:

$E_{\text{mean}}$  is the mean value of modulus of elasticity;

$G_{\text{mean}}$  is the mean value of shear modulus;

$K_{\text{ser}}$  is the slip modulus;

$k_{\text{def}}$  is a factor for the evaluation of creep deformation taking into account the relevant service class;

$\psi_2$  is the factor for the quasi-permanent value of the action causing the largest stress in relation to the strength (if this action is a permanent action,  $\psi_2$  should be replaced by 1).

NOTE 1: Values of  $k_{\text{def}}$  are given in 3.1.4.

NOTE 2: Values of  $\psi_2$  are given in EN 1990:2002.

(3) Where a connection is constituted of timber elements with the same time-dependent behaviour, the value of  $k_{\text{def}}$  should be doubled.

(4) Where a connection is constituted of two wood-based elements having different time-dependent behaviour, the calculation of the final deformation should be made with the following deformation factor  $k_{\text{def}}$ :

$$k_{\text{def}} = 2 \sqrt{k_{\text{def},1} k_{\text{def},2}} \quad (2.13)$$

where  $k_{\text{def},1}$  and  $k_{\text{def},2}$  are the deformation factors for the two timber elements.

## 2.4 Verification by the partial factor method

### 2.4.1 Design value of material property

(1)P The design value  $X_d$  of a strength property shall be calculated as:

$$X_d = k_{\text{mod}} \frac{X_k}{\gamma_M} \quad (2.14)$$

where:

$X_k$  is the characteristic value of a strength property;

$\gamma_M$  is the partial factor for a material property;

$k_{\text{mod}}$  is a modification factor taking into account the effect of the duration of load and moisture content.

NOTE 1: Values of  $k_{\text{mod}}$  are given in 3.1.3.

NOTE 2: The recommended partial factors for material properties ( $\gamma_M$ ) are given in Table 2.3. Information on the National choice may be found in the National annex.

**Table 2.3 – Recommended partial factors  $\gamma_M$  for material properties and resistances**

Fundamental combinations:	
Solid timber	1,3
Glued laminated timber	1,25
LVL, plywood, OSB,	1,2
Particleboards	1,3
Fibreboards, hard	1,3
Fibreboards, medium	1,3
Fibreboards, MDF	1,3
Fibreboards, soft	1,3
Connections	1,3
Punched metal plate fasteners	1,25
Accidental combinations	1,0

(2)P The design member stiffness property  $E_d$  or  $G_d$  shall be calculated as:

$$E_d = \frac{E_{\text{mean}}}{\gamma_M} \quad (2.15)$$

$$G_d = \frac{G_{\text{mean}}}{\gamma_M} \quad (2.16)$$

where:

$E_{\text{mean}}$  is the mean value of modulus of elasticity;

$G_{\text{mean}}$  is the mean value of shear modulus.

#### 2.4.2 Design value of geometrical data

- (1) Geometrical data for cross-sections and systems may be taken as nominal values from product standards hEN or drawings for the execution.
- (2) Design values of geometrical imperfections specified in this standard comprise the effects of
- geometrical imperfections of members;
  - the effects of structural imperfections from fabrication and erection;
  - inhomogeneity of materials (e.g. due to knots).

#### 2.4.3 Design resistances

- (1)P The design value  $R_d$  of a resistance (load-carrying capacity) shall be calculated as:

$$R_d = k_{\text{mod}} \frac{R_k}{\gamma_M} \quad (2.17)$$

where:

$R_k$  is the characteristic value of load-carrying capacity;

$\gamma_M$  is the partial factor for a material property,

$k_{\text{mod}}$  is a modification factor taking into account the effect of the duration of load and moisture content.

NOTE 1: Values of  $k_{\text{mod}}$  are given in 3.1.3.

NOTE 2: For partial factors, see 2.4.1.

#### 2.4.4 Verification of equilibrium (EQU)

- (1) The reliability format for the verification of static equilibrium given in Table A1.2 (A) in Annex A1 of EN 1990:2002 applies, where appropriate, to the design of timber structures, e.g. for the design of holding-down anchors or the verification of bearings subject to uplift from continuous beams.

## Section 3 Material properties

### 3.1 General

#### 3.1.1 Strength and stiffness parameters

(1)P Strength and stiffness parameters shall be determined on the basis of tests for the types of action effects to which the material will be subjected in the structure, or on the basis of comparisons with similar timber species and grades or wood-based materials, or on well-established relations between the different properties.

#### 3.1.2 Stress-strain relations

(1)P Since the characteristic values are determined on the assumption of a linear relation between stress and strain until failure, the strength verification of individual members shall also be based on such a linear relation.

(2) For members or parts of members subjected to compression, a non-linear relationship (elastic-plastic) may be used.

#### 3.1.3 Strength modification factors for service classes and load-duration classes

(1) The values of the modification factor  $k_{\text{mod}}$  given in Table 3.1 should be used.

(2) If a load combination consists of actions belonging to different load-duration classes a value of  $k_{\text{mod}}$  should be chosen which corresponds to the action with the shortest duration, e.g. for a combination of dead load and a short-term load, a value of  $k_{\text{mod}}$  corresponding to the short-term load should be used.

#### 3.1.4 Deformation modification factors for service classes

(1) The values of the deformation factors  $k_{\text{def}}$  given in Table 3.2 should be used.

### 3.2 Solid timber

**A1** (1)P Timber members shall comply with EN 14081-1.

NOTE: Strength classes for timber are given in EN 338. **A1**

(2) The effect of member size on strength may be taken into account.

(3) For rectangular solid timber with a characteristic timber density  $\rho_k \leq 700 \text{ kg/m}^3$ , the reference depth in bending or width (maximum cross-sectional dimension) in tension is 150 mm. For depths in bending or widths in tension of solid timber less than 150 mm the characteristic values for  $f_{\text{m,k}}$  and  $f_{\text{t,0,k}}$  may be increased by the factor  $k_{\text{h}}$ , given by:

$$k_{\text{h}} = \min \left\{ \left( \frac{150}{h} \right)^{0,2} \right. \\ \left. 1,3 \right. \quad (3.1)$$

where  $h$  is the depth for bending members or width for tension members, in mm.

**Table 3.1 – Values of  $k_{mod}$**

Material	Standard	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instantaneous action
Solid timber	EN 14081-1	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Glued laminated timber	EN 14080	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
LVL	EN 14374, EN 14279	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Plywood	EN 636						
	Type EN 636-1	1	0,60	0,70	0,80	0,90	1,10
	Type EN 636-2	2	0,60	0,70	0,80	0,90	1,10
OSB	Type EN 636-3	3	0,50	0,55	0,65	0,70	0,90
	EN 300						
	OSB/2	1	0,30	0,45	0,65	0,85	1,10
Particle-board	OSB/3, OSB/4	1	0,40	0,50	0,70	0,90	1,10
	OSB/3, OSB/4	2	0,30	0,40	0,55	0,70	0,90
	EN 312						
Fibreboard, hard	Type P4, Type P5	1	0,30	0,45	0,65	0,85	1,10
	Type P5	2	0,20	0,30	0,45	0,60	0,80
	Type P6, Type P7	1	0,40	0,50	0,70	0,90	1,10
Fibreboard, medium	Type P7	2	0,30	0,40	0,55	0,70	0,90
	EN 622-2						
	HB.LA, HB.HLA 1 or 2	1	0,30	0,45	0,65	0,85	1,10
Fibreboard, MDF	HB.HLA1 or 2	2	0,20	0,30	0,45	0,60	0,80
	EN 622-3						
	MBH.LA1 or 2	1	0,20	0,40	0,60	0,80	1,10
Fibreboard, MDF	MBH.HLS1 or 2	1	0,20	0,40	0,60	0,80	1,10
	MBH.HLS1 or 2	2	–	–	–	0,45	0,80
	EN 622-5						
Fibreboard, MDF	MDF.LA, MDF.HLS	1	0,20	0,40	0,60	0,80	1,10
	MDF.HLS	2	–	–	–	0,45	0,80

(4) For timber which is installed at or near its fibre saturation point, and which is likely to dry out under load, the values of  $k_{def}$ , given in Table 3.2, should be increased by 1,0.

(5)P Finger joints shall comply with EN 385.

### 3.3 Glued laminated timber

(1)P Glued laminated timber members shall comply with EN 14080.

NOTE: In EN 1194 values of strength and stiffness properties are given for glued laminated timber allocated to strength classes, see annex D (Informative).

(2) The effect of member size on strength may be taken into account.

(3) For rectangular glued laminated timber, the reference depth in bending or width in tension is 600 mm. For depths in bending or widths in tension of glued laminated timber less than 600 mm

the characteristic values for  $f_{m,k}$  and  $f_{t,0,k}$  may be increased by the factor  $k_h$ , given by

$$k_h = \min \left\{ \left( \frac{600}{h} \right)^{0,1} ; 1,1 \right\} \quad (3.2)$$

where  $h$  is the depth for bending members or width for tensile members, in mm.

**[A1]** (4)P Large finger joints complying with the requirements of EN 387 shall not be used for products to be installed in service class 3, where the direction of grain changes at the joint. **[A1]**

(5)P The effect of member size on the tensile strength perpendicular to the grain shall be taken into account.

**[A1]** Table 3.2 – Values of  $k_{def}$  for timber and wood-based materials

Material	Standard	Service class		
		1	2	3
Solid timber	EN 14081-1	0,60	0,80	2,00
Glued Laminated timber	EN 14080	0,60	0,80	2,00
LVL	EN 14374, EN 14279	0,60	0,80	2,00
Plywood	EN 636			
	Type EN 636-1	0,80	–	–
	Type EN 636-2	0,80	1,00	–
	Type EN 636-3	0,80	1,00	2,50
OSB	EN 300			
	OSB/2	2,25	–	–
	OSB/3, OSB/4	1,50	2,25	–
Particleboard	EN 312			
	Type P4	2,25	–	–
	Type P5	2,25	3,00	–
	Type P6	1,50	–	–
	Type P7	1,50	2,25	–
Fibreboard, hard	EN 622-2			
	HB.LA	2,25	–	–
	HB.HLA1, HB.HLA2	2,25	3,00	–
Fibreboard, medium	EN 622-3			
	MBH.LA1, MBH.LA2	3,00	–	–
	MBH.HLS1, MBH.HLS2	3,00	4,00	–
Fibreboard, MDF	EN 622-5			
	MDF.LA	2,25	–	–
	MDF.HLS	2,25	3,00	–

**[A1]**

### 3.4 Laminated veneer lumber (LVL)

(1)P LVL structural members shall comply with EN 14374.

(2)P For rectangular LVL with the grain of all veneers running essentially in one direction, the effect of member size on bending and tensile strength shall be taken into account.

(3) The reference depth in bending is 300 mm. For depths in bending not equal to 300 mm the characteristic value for  $f_{m,k}$  should be multiplied by the factor  $k_h$ , given by

$$k_h = \min \left\{ \left( \frac{300}{h} \right)^s, 1,2 \right\} \quad (3.3)$$

where:

$h$  is the depth of the member, in mm;

$s$  is the size effect exponent, refer to 3.4(5)P.

(4) The reference length in tension is 3000 mm. For lengths in tension not equal to 3000 mm the characteristic value for  $f_{t,0,k}$  should be multiplied by the factor  $k_\ell$  given by

$$k_\ell = \min \left\{ \left( \frac{3000}{\ell} \right)^{s/2}, 1,1 \right\} \quad (3.4)$$

where  $\ell$  is the length, in mm.

(5)P The size effect exponent  $s$  for LVL shall be taken as declared in accordance with EN 14374.

**A1** (6)P Large finger joints complying with the requirements of EN 387 shall not be used for products to be installed in service class 3, where the direction of grain changes at the joint. **A1**

(7)P For LVL with the grain of all veneers running essentially in one direction, the effect of member size on the tensile strength perpendicular to the grain shall be taken into account.

### 3.5 Wood-based panels

(1)P Wood-based panels shall comply with EN 13986 and LVL used as panels shall comply with EN 14279.

(2) The use of softboards according to EN 622-4 should be restricted to wind bracing and should be designed by testing.

### 3.6 Adhesives

(1)P Adhesives for structural purposes shall produce joints of such strength and durability that the integrity of the bond is maintained in the assigned service class throughout the expected life of the structure.

(2) Adhesives which comply with Type I specification as defined in EN 301 may be used in all service classes.

(3) Adhesives which comply with Type II specification as defined in EN 301 should only be used in service classes 1 or 2 and not under prolonged exposure to temperatures in excess of 50°C.

### 3.7 Metal fasteners

(1)P Metal fasteners shall comply with EN 14592 and metal connectors shall comply with EN 14545.

## Section 4 Durability

### 4.1 Resistance to biological organisms

(1)P Timber and wood-based materials shall either have adequate natural durability in accordance with EN 350-2 for the particular hazard class (defined in EN 335-1, EN 335-2 and EN 335-3), or be given a preservative treatment selected in accordance with EN 351-1 and EN 460.

NOTE 1: Preservative treatment may affect the strength and stiffness properties.

NOTE 2: Rules for specification of preservation treatments are given in EN 350-2 and EN 335.

### 4.2 Resistance to corrosion

(1)P Metal fasteners and other structural connections shall, where necessary, either be inherently corrosion-resistant or be protected against corrosion.

(2) Examples of minimum corrosion protection or material specifications for different service classes (see 2.3.1.3) are given in Table 4.1.

**Table 4.1 – Examples of minimum specifications for material protection against corrosion for fasteners (related to ISO 2081)**

Fastener	Service Class <sup>b</sup>		
	1	2	3
Nails and screws with $d \leq 4$ mm	None	Fe/Zn 12c <sup>a</sup>	Fe/Zn 25c <sup>a</sup>
Bolts, dowels, nails and screws with $d > 4$ mm	None	None	Fe/Zn 25c <sup>a</sup>
Staples	Fe/Zn 12c <sup>a</sup>	Fe/Zn 12c <sup>a</sup>	Stainless steel
Punched metal plate fasteners and steel plates up to 3 mm thickness	Fe/Zn 12c <sup>a</sup>	Fe/Zn 12c <sup>a</sup>	Stainless steel
Steel plates from 3 mm up to 5 mm in thickness	None	Fe/Zn 12c <sup>a</sup>	Fe/Zn 25c <sup>a</sup>
Steel plates over 5 mm thickness	None	None	Fe/Zn 25c <sup>a</sup>
<sup>a</sup> If hot dip zinc coating is used, Fe/Zn 12c should be replaced by Z275 and Fe/Zn 25c by Z350 in accordance with EN 10147			
<sup>b</sup> For especially corrosive conditions consideration should be given to heavier hot dip coatings or stainless steel.			

## Section 5 Basis of structural analysis

### 5.1 General

(1)P Calculations shall be performed using appropriate design models (supplemented, if necessary, by tests) involving all relevant variables. The models shall be sufficiently precise to predict the structural behaviour, commensurate with the standard of workmanship likely to be achieved, and with the reliability of the information on which the design is based.

(2) The global structural behaviour should be assessed by calculating the action effects with a linear material model (elastic behaviour).

(3) For structures able to redistribute the internal forces via connections of adequate ductility, elastic-plastic methods may be used for the calculation of the internal forces in the members.

(4)P The model for the calculation of internal forces in the structure or in part of it shall take into account the effects of deformations of the connections.

(5) In general, the influence of deformations in the connections should be taken into account through their stiffness (rotational or translational for instance) or through prescribed slip values as a function of the load level in the connection.

### 5.2 Members

(1)P The following shall be taken into account by the structural analysis:

- deviations from straightness;
- inhomogeneities of the material.

NOTE: Deviations from straightness and inhomogeneities are taken into account implicitly by the design methods given in this standard.

(2)P Reductions in the cross-sectional area shall be taken into account in the member strength verification.

(3) Reductions in the cross-sectional area may be ignored for the following cases:

- nails and screws with a diameter of 6 mm or less, driven without pre-drilling;
- holes in the compression area of members, if the holes are filled with a material of higher stiffness than the wood.

(4) When assessing the effective cross-section at a joint with multiple fasteners, all holes within a distance of half the minimum fastener spacing measured parallel to the grain from a given cross-section should be considered as occurring at that cross-section.

### 5.3 Connections

(1)P The load-carrying-capacity of the connections shall be verified taking into account the forces and the moments between the members determined by the global structural analysis.

(2)P The deformation of the connection shall be compatible with that assumed in the global analysis.

(3)P The analysis of a connection shall take into account the behaviour of all the elements which constitute the connection.

## 5.4 Assemblies

### 5.4.1 General

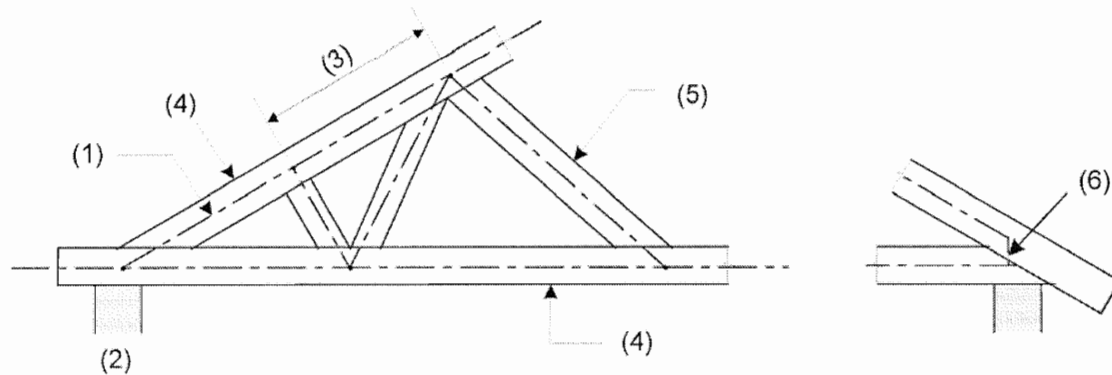
(1)P The analysis of structures shall be carried out using static models which consider to an acceptable level of accuracy the behaviour of the structure and of the supports.

(2) The analysis should be performed by frame models in accordance with 5.4.2 or by a simplified analysis in accordance with 5.4.3 for trusses with punched metal plate fasteners.

(3) Second order analysis of plane frames or arches should be performed in accordance with 5.4.4.

### 5.4.2 Frame structures

(1)P Frame structures shall be analysed such that the deformations of the members and joints, the influence of support eccentricities and the stiffness of the supporting structure are taken into account in the determination of the member forces and moments, see Figure 5.1 for definitions of structure configurations and model elements.



Key:

- (1) System line
- (2) Support
- (3) Bay
- (4) External member
- (5) Internal member
- (6) Fictitious beam element

**Figure 5.1 – Examples of frame analysis model elements**

(2)P In a frame analysis, the system lines for all members shall lie within the member profile. For the main members, e.g. the external members of a truss, the system lines shall coincide with the member centre-line.

(3)P If the system lines for internal members do not coincide with the centre lines, the influence of the eccentricity shall be taken into account in the strength verification of these members.

(4) Fictitious beam elements and spring elements may be used to model eccentric connections or supports. The orientation of fictitious beam elements and the location of the spring elements should coincide as closely as possible with the actual joint configuration.

(5) In a first order linear elastic analysis, the effect of initial deformations and induced deflections may be disregarded if taken into account by the strength verification of the member.

(6) The frame analysis should be carried out using the appropriate values of member stiffness defined in 2.2.2. Fictitious beam elements should be assumed to have a stiffness corresponding to that of the actual connections.

(7) Connections may be assumed to be rotationally stiff, if their deformation has no significant effect upon the distribution of member forces and moments. Otherwise, connections may be generally assumed to be rotationally pinned.

(8) Translational slip at the joints may be disregarded for the strength verification unless it significantly affects the distribution of internal forces and moments.

(9) Splice connections used in lattice structures may be modelled as rotationally stiff if the actual rotation under load would have no significant effect upon member forces. This requirement is fulfilled if one of the following conditions is satisfied:

- The splice connection has a load-carrying capacity which corresponds to at least 1,5 times the combination of applied force and moment
- The splice connection has a load-carrying capacity which corresponds to at least the combination of applied force and moment, provided that the timber members are not subject to bending stresses which are greater than 0,3 times the member bending strength, and the assembly would be stable if all such connections acted as pins.

#### **5.4.3 Simplified analysis of trusses with punched metal plate fasteners**

(1) A simplified analysis of fully triangulated trusses should comply with the following conditions:

- there are no re-entrant angles in the external profile;
- the bearing width is situated within the length  $a_1$ , and the distance  $a_2$  in Figure 5.2 is not greater than  $a_1/3$  or 100 mm, whichever is the greater;
- the truss height is greater than 0,15 times the span and 10 times the maximum external member depth.

(2) The axial forces in the members should be determined on the basis that every node is pin-jointed.

(3) The bending moments in single-bay members should be determined on the basis that the end nodes are pin-jointed. Bending moments in members that are continuous over several bays should be determined on the basis that the member is a beam with a simple support at each node. The effect of deflection at the nodes and partial fixity at the connections should be taken into account by a reduction of 10 % of the moments at the inner supports of the member. The inner support moments should be used to calculate the span bending moments.

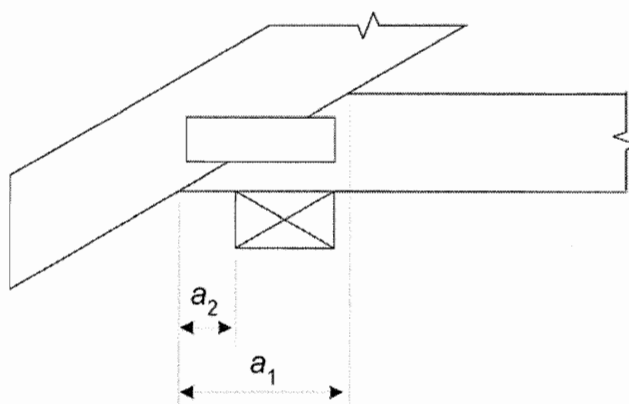


Figure 5.2 – Geometry of support

#### 5.4.4 Plane frames and arches

(1)P The requirements of 5.2 apply. The effects of induced deflection on internal forces and moments shall be taken into account.

(2) The effects of induced deflection on internal forces and moments may be taken into account by carrying out a second order linear analysis with the following assumptions:

- the imperfect shape of the structure should be assumed to correspond to an initial deformation which is found by applying an angle  $\phi$  of inclination to the structure or relevant parts, together with an initial sinusoidal curvature between the nodes of the structure corresponding to a maximum eccentricity  $e$ .
- the value of  $\phi$  in radians should as a minimum be taken as

$$\begin{aligned} \phi &= 0,005 & \text{for } h \leq 5 \text{ m} \\ \phi &= 0,005 \sqrt{5/h} & \text{for } h > 5 \text{ m} \end{aligned} \quad (5.1)$$

where  $h$  is the height of the structure or the length of the member, in m.

- the value of  $e$  should as a minimum be taken as:

$$e = 0,0025 \ell \quad (5.2)$$

Examples of assumed initial deviations in the geometry and the definition of  $\ell$  are given in Figure 5.3.

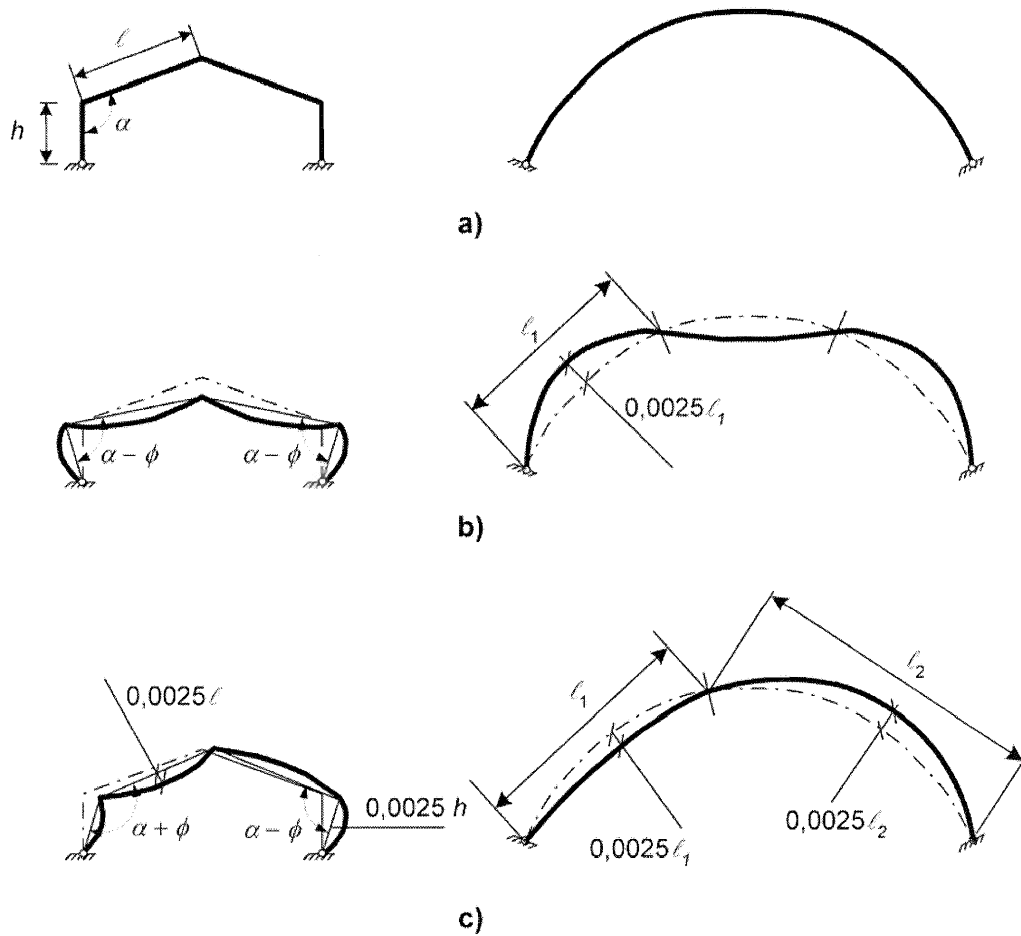


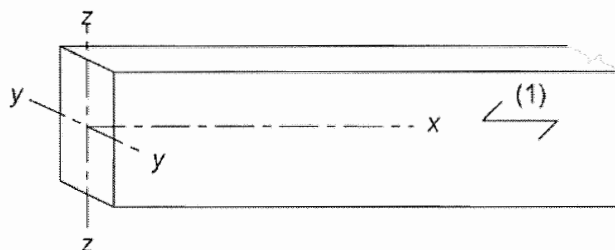
Figure 5.3 – Examples of assumed initial deviations in the geometry for a frame (a), corresponding to a symmetrical load (b) and non-symmetrical load (c)

## Section 6 Ultimate limit states

### 6.1 Design of cross-sections subjected to stress in one principal direction

#### 6.1.1 General

(1) Clause 6.1 applies to straight solid timber, glued laminated timber or wood-based structural products of constant cross-section, whose grain runs essentially parallel to the length of the member. The member is assumed to be subjected to stresses in the direction of only one of its principal axes (see Figure 6.1).



Key:  
(1) direction of grain

Figure 6.1 – Member Axes

#### 6.1.2 Tension parallel to the grain

(1)P The following expression shall be satisfied:

$$\sigma_{t,0,d} \leq f_{t,0,d} \quad (6.1)$$

where:

$\sigma_{t,0,d}$  is the design tensile stress along the grain;  
 $f_{t,0,d}$  is the design tensile strength along the grain.

#### 6.1.3 Tension perpendicular to the grain

(1)P The effect of member size shall be taken into account.

#### 6.1.4 Compression parallel to the grain

(1)P The following expression shall be satisfied:

$$\sigma_{c,0,d} \leq f_{c,0,d} \quad (6.2)$$

where:

$\sigma_{c,0,d}$  is the design compressive stress along the grain;  
 $f_{c,0,d}$  is the design compressive strength along the grain.

NOTE: Rules for the instability of members are given in 6.3.

#### 6.1.5 Compression perpendicular to the grain

$\boxed{A_1}$  (1)P The following expression shall be satisfied:

$$\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d} \quad (6.3)$$

with:  $\boxed{A_1}$

$$\boxed{A1} \sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} \quad (6.4)$$

where:

- $\sigma_{c,90,d}$  is the design compressive stress in the effective contact area perpendicular to the grain;
- $F_{c,90,d}$  is the design compressive load perpendicular to the grain;
- $A_{ef}$  is the effective contact area in compression perpendicular to the grain;
- $f_{c,90,d}$  is the design compressive strength perpendicular to the grain;
- $k_{c,90}$  is a factor taking into account the load configuration, the possibility of splitting and the degree of compressive deformation.

The effective contact area perpendicular to the grain,  $A_{ef}$ , should be determined taking into account an effective contact length parallel to the grain, where the actual contact length,  $\ell$ , at each side is increased by 30 mm, but not more than  $a$ ,  $\ell$  or  $\ell_1/2$ , see Figure 6.2.

(2) The value of  $k_{c,90}$  should be taken as 1,0 unless the conditions in the following paragraphs apply. In these cases the higher value of  $k_{c,90}$  specified may be taken, with a limiting value of  $k_{c,90} = 1,75$ .

(3) For members on continuous supports, provided that  $\ell_1 \geq 2h$ , see Figure 6.2a, the value of  $k_{c,90}$  should be taken as:

- $k_{c,90} = 1,25$  for solid softwood timber
- $k_{c,90} = 1,5$  for glued laminated softwood timber

where  $h$  is the depth of the member and  $\ell$  is the contact length.

(4) For members on discrete supports, provided that  $\ell_1 \geq 2h$ , see Figure 6.2b, the value of  $k_{c,90}$  should be taken as:

- $k_{c,90} = 1,5$  for solid softwood timber
- $k_{c,90} = 1,75$  for glued laminated softwood timber provided that  $\ell \leq 400$  mm

where  $h$  is the depth of the member and  $\ell$  is the contact length.

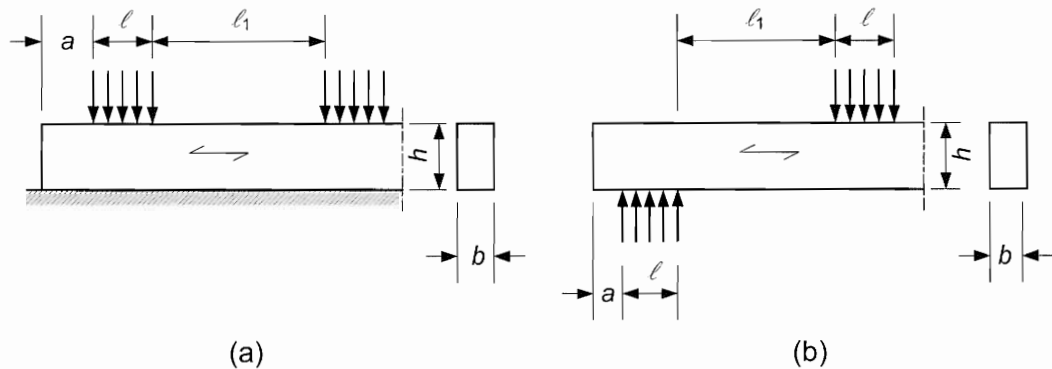


Figure 6.2 – Member on (a) continuous and (b) discrete supports

### 6.1.6 Bending

(1)P The following expressions shall be satisfied:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (6.11)$$

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (6.12)$$

where:

$\sigma_{m,y,d}$  and  $\sigma_{m,z,d}$  are the design bending stresses about the principal axes as shown in Figure 6.1;

$f_{m,y,d}$  and  $f_{m,z,d}$  are the corresponding design bending strengths.

NOTE: The factor  $k_m$  makes allowance for re-distribution of stresses and the effect of inhomogeneities of the material in a cross-section.

(2) The value of the factor  $k_m$  should be taken as follows:

For solid timber, glued laminated timber and LVL:

for rectangular sections:  $k_m = 0,7$

for other cross-sections:  $k_m = 1,0$

For other wood-based structural products, for all cross-sections:  $k_m = 1,0$

(3)P A check shall also be made of the instability condition (see 6.3).

### 6.1.7 Shear

**[A1]** (1)P For shear with a stress component parallel to the grain, see Figure 6.5(a), as well as for shear with both stress components perpendicular to the grain, see Figure 6.5(b), the following expression shall be satisfied:

$$\tau_d \leq f_{v,d} \quad (6.13)$$

where:

$\tau_d$  is the design shear stress;

$f_{v,d}$  is the design shear strength for the actual condition.

NOTE: The shear strength for rolling shear is approximately equal to twice the tensile strength perpendicular to grain.

(2) For the verification of shear resistance of members in bending, the influence of cracks should be taken into account using an effective width of the member given as:

$$b_{ef} = k_{cr} b \quad (6.13a)$$

where  $b$  is the width of the relevant section of the member.

NOTE: The recommended value for  $k_{cr}$  is given as

$k_{cr} = 0,67$  for solid timber

$k_{cr} = 0,67$  for glued laminated timber

$k_{cr} = 1,0$  for other wood-based products in accordance with EN 13986 and EN 14374.

Information on the National choice may be found in the National annex. **[A1]**

A1

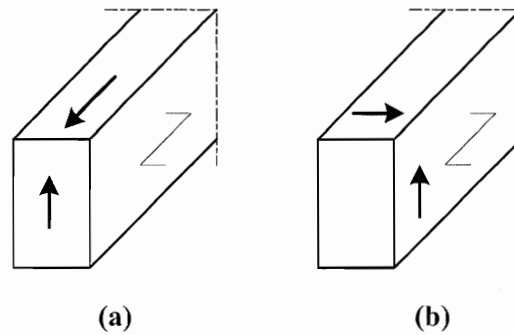


Figure 6.5 – (a) Member with a shear stress component parallel to the grain (b) Member with both stress components perpendicular to the grain (rolling shear)

(3) At supports, the contribution to the total shear force of a concentrated load  $F$  acting on the top side of the beam and within a distance  $h$  or  $h_{ef}$  from the edge of the support may be disregarded (see Figure 6.6). For beams with a notch at the support this reduction in the shear force applies only when the notch is on the opposite side to the support.

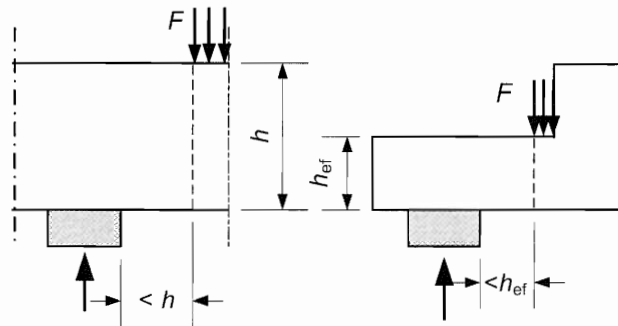


Figure 6.6 – Conditions at a support, for which the concentrated force  $F$  may be disregarded in the calculation of the shear force A1

### 6.1.8 Torsion

(1)P The following expression shall be satisfied:

$$\tau_{\text{tor,d}} \leq k_{\text{shape}} f_{\text{v,d}} \quad (6.14)$$

with

$$k_{\text{shape}} = \begin{cases} 1,2 & \text{for a circular cross section} \\ \min \left\{ \begin{array}{l} 1+0,15 \frac{h}{b} \\ 2,0 \end{array} \right\} & \text{for a rectangular cross section} \end{cases} \quad (6.15)$$

where:

- $\tau_{\text{tor,d}}$  is the design torsional stress;
- $f_{\text{v,d}}$  is the design shear strength;
- $k_{\text{shape}}$  is a factor depending on the shape of the cross-section;
- $h$  is the larger cross-sectional dimension;
- $b$  is the smaller cross-sectional dimension.

## 6.2 Design of cross-sections subjected to combined stresses

### 6.2.1 General

(1)P Clause 6.2 applies to straight solid timber, glued laminated timber or wood-based structural products of constant cross-section, whose grain runs essentially parallel to the length of the member. The member is assumed to be subjected to stresses from combined actions or to stresses acting in two or three of its principal axes.

### 6.2.2 Compression stresses at an angle to the grain

(1)P Interaction of compressive stresses in two or more directions shall be taken into account.

(2) The compressive stresses at an angle  $\alpha$  to the grain, (see Figure 6.7), should satisfy the following expression:

$$\sigma_{c,\alpha,d} \leq \frac{f_{c,0,d}}{\frac{f_{c,0,d}}{k_{c,90} f_{c,90,d}} \sin^2 \alpha + \cos^2 \alpha} \quad (6.16)$$

where:

$\sigma_{c,\alpha,d}$  is the compressive stress at an angle  $\alpha$  to the grain;

$k_{c,90}$  is a factor given in 6.1.5 taking into account the effect of any of stresses perpendicular to the grain.

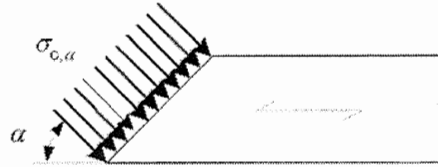


Figure 6.7 – Compressive stresses at an angle to the grain

### 6.2.3 Combined bending and axial tension

(1)P The following expressions shall be satisfied:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (6.17)$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (6.18)$$

(2) The values of  $k_m$  given in 6.1.6 apply.

### 6.2.4 Combined bending and axial compression

(1)P The following expressions shall be satisfied:

$$\left( \frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (6.19)$$

$$\left( \frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (6.20)$$

(2)P The values of  $k_m$  given in 6.1.6 apply.

NOTE: To check the instability condition, a method is given in 6.3.

### 6.3 Stability of members

#### 6.3.1 General

(1)P The bending stresses due to initial curvature, eccentricities and induced deflection shall be taken into account, in addition to those due to any lateral load.

(2)P Column stability and lateral torsional stability shall be verified using the characteristic properties, e.g.  $E_{0,05}$

(3) The stability of columns subjected to either compression or combined compression and bending should be verified in accordance with 6.3.2.

(4) The lateral torsional stability of beams subjected to either bending or combined bending and compression should be verified in accordance with 6.3.3.

#### 6.3.2 Columns subjected to either compression or combined compression and bending

(1) The relative slenderness ratios should be taken as:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad (6.21)$$

and

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad (6.22)$$

where:

$\lambda_y$  and  $\lambda_{rel,y}$  are slenderness ratios corresponding to bending about the y-axis (deflection in the z-direction);

$\lambda_z$  and  $\lambda_{rel,z}$  are slenderness ratios corresponding to bending about the z-axis (deflection in the y-direction);

$E_{0,05}$  is the fifth percentile value of the modulus of elasticity parallel to the grain.

(2) Where both  $\lambda_{rel,z} \leq 0,3$  and  $\lambda_{rel,y} \leq 0,3$  the stresses should satisfy the expressions (6.19) and (6.20) in 6.2.4.

(3) In all other cases the stresses, which will be increased due to deflection, should satisfy the following expressions:

$$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (6.23)$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (6.24)$$

where the symbols are defined as follows:

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \quad (6.25)$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} \quad (6.26)$$

$$k_y = 0,5 \left( 1 + \beta_c (\lambda_{rel,y} - 0,3) + \lambda_{rel,y}^2 \right) \quad (6.27)$$

$$k_z = 0,5 \left( 1 + \beta_c (\lambda_{rel,z} - 0,3) + \lambda_{rel,z}^2 \right) \quad (6.28)$$

where:

$\beta_c$  is a factor for members within the straightness limits defined in Section 10:

$$\beta_c = \begin{cases} 0,2 & \text{for solid timber} \\ 0,1 & \text{for glued laminated timber and LVL} \end{cases} \quad (6.29)$$

$k_m$  as given in 6.1.6.

### 6.3.3 Beams subjected to either bending or combined bending and compression

(1)P Lateral torsional stability shall be verified both in the case where only a moment  $M_y$  exists about the strong axis  $y$  and where a combination of moment  $M_y$  and compressive force  $N_c$  exists.

(2) The relative slenderness for bending should be taken as:

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} \quad (6.30)$$

where  $\sigma_{m,crit}$  is the critical bending stress calculated according to the classical theory of stability, using 5-percentile stiffness values.

The critical bending stress should be taken as:

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi \sqrt{E_{0,05} I_z G_{0,05} I_{tor}}}{\ell_{ef} W_y} \quad (6.31)$$

where:

$E_{0,05}$  is the fifth percentile value of modulus of elasticity parallel to grain;

$G_{0,05}$  is the fifth percentile value of shear modulus parallel to grain;

$I_z$  is the second moment of area about the weak axis  $z$ .

$I_{tor}$  is the torsional moment of inertia;

$\ell_{ef}$  is the effective length of the beam, depending on the support conditions and the load configuration, according to Table 6.1;

$W_y$  is the section modulus about the strong axis  $y$ .

For softwood with solid rectangular cross-section,  $\sigma_{m,crit}$  should be taken as:

$$\sigma_{m,crit} = \frac{0,78 b^2}{h \ell_{ef}} E_{0,05} \quad (6.32)$$

where:

$b$  is the width of the beam;

$h$  is the depth of the beam.

(3) In the case where only a moment  $M_y$  exists about the strong axis  $y$ , the stresses should satisfy the following expression:

$$\sigma_{m,d} \leq k_{crit} f_{m,d} \quad (6.33)$$

where:

$\sigma_{m,d}$  is the design bending stress;

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

$k_{crit}$  is a factor which takes into account the reduced bending strength due to lateral buckling.

**Table 6.1 – Effective length as a ratio of the span**

Beam type	Loading type	$\ell_{ef}/\ell^a$
Simply supported	Constant moment	1,0
	Uniformly distributed load	0,9
	Concentrated force at the middle of the span	0,8
Cantilever	Uniformly distributed load	0,5
	Concentrated force at the free end	0,8
<sup>a</sup> The ratio between the effective length $\ell_{ef}$ and the span $\ell$ is valid for a beam with torsionally restrained supports and loaded at the centre of gravity. If the load is applied at the compression edge of the beam, $\ell_{ef}$ should be increased by $2h$ and may be decreased by $0,5h$ for a load at the tension edge of the beam.		

(4) For beams with an initial lateral deviation from straightness within the limits defined in Section 10,  $k_{crit}$  may be determined from expression (6.34)

$$k_{crit} = \begin{cases} 1 & \text{for } \lambda_{rel,m} \leq 0,75 \\ 1,56 - 0,75 \lambda_{rel,m} & \text{for } 0,75 < \lambda_{rel,m} \leq 1,4 \\ \frac{1}{\lambda_{rel,m}^2} & \text{for } 1,4 < \lambda_{rel,m} \end{cases} \quad (6.34)$$

(5) The factor  $k_{crit}$  may be taken as 1,0 for a beam where lateral displacement of its compressive edge is prevented throughout its length and where torsional rotation is prevented at its supports.

ⓘ (6) In the case where a combination of moment  $M_y$  about the strong axis  $y$  and compressive force  $N_c$  exists, the stresses should satisfy the following expression:

$$\left( \frac{\sigma_{m,d}}{k_{crit} f_{m,d}} \right)^2 + \frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} \leq 1 \quad (6.35)$$

where:

$\sigma_{m,d}$  is the design bending stress;

$\sigma_{c,0,d}$  is the design compressive stress parallel to grain;

$f_{c,0,d}$  is the design compressive strength parallel to grain;

$k_{c,z}$  is given by expression (6.26). ⓘ

## 6.4 Design of cross-sections in members with varying cross-section or curved shape

### 6.4.1 General

(1)P The effects of combined axial force and bending moment shall be taken into account.

(2) The relevant parts of 6.2 and 6.3 should be verified.

(3) The stress at a cross-section from an axial force may be calculated from

$$\sigma_N = \frac{N}{A} \quad (6.36)$$

where:

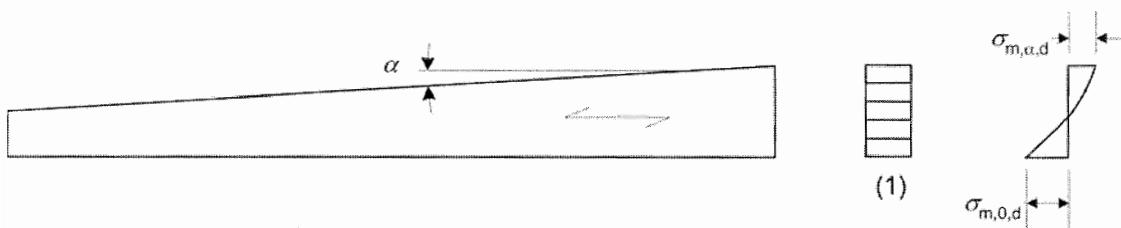
$\sigma_N$  is the axial stress;

$N$  is the axial force;

$A$  is the area of the cross-section.

### 6.4.2 Single tapered beams

(1)P The influence of the taper on the bending stresses parallel to the surface shall be taken into account.



Key:

(1) cross-section

Figure 6.8 – Single tapered beam

(2) The design bending stresses,  $\sigma_{m,\alpha,d}$  and  $\sigma_{m,0,d}$  (see Figure 6.8) may be taken as:

$$\sigma_{m,\alpha,d} = \sigma_{m,0,d} = \frac{6M_d}{bh^2} \quad (6.37)$$

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

$$\sigma_{m,\alpha,d} \leq k_{m,\alpha} f_{m,d} \quad (6.38)$$

where:

$\sigma_{m,\alpha,d}$  is the design bending stress at an angle to grain;

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

$k_{m,\alpha}$  should be calculated as:

For tensile stresses parallel to the tapered edge:

$$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left( \frac{f_{m,d}}{0,75 f_{v,d}} \tan \alpha \right)^2 + \left( \frac{f_{m,d}}{f_{t,90,d}} \tan^2 \alpha \right)^2}} \quad (6.39)$$

For compressive stresses parallel to the tapered edge:

$$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left( \frac{f_{m,d}}{1,5 f_{v,d}} \tan \alpha \right)^2 + \left( \frac{f_{m,d}}{f_{c,90,d}} \tan^2 \alpha \right)^2}} \quad (6.40)$$

### 6.4.3 Double tapered, curved and pitched cambered beams

(1) This clause applies only to glued laminated timber and LVL.

(2) The requirements of 6.4.2 apply to the parts of the beam which have a single taper.

(3) In the apex zone (see Figure 6.9), the bending stresses should satisfy the following expression:

$$\sigma_{m,d} \leq k_r f_{m,d} \quad (6.41)$$

where  $k_r$  takes into account the strength reduction due to bending of the laminates during production.

NOTE: In curved and pitched cambered beams the apex zone extends over the curved part of the beam

(4) The apex bending stress should be calculated as follows:

$$\sigma_{m,d} = k_\ell \frac{6M_{ap,d}}{bh_{ap}^2} \quad (6.42)$$

with:

$$k_\ell = k_1 + k_2 \left( \frac{h_{ap}}{r} \right) + k_3 \left( \frac{h_{ap}}{r} \right)^2 + k_4 \left( \frac{h_{ap}}{r} \right)^3 \quad (6.43)$$

$$k_1 = 1 + 1,4 \tan \alpha_{ap} + 5,4 \tan^2 \alpha_{ap} \quad (6.44)$$

$$k_2 = 0,35 - 8 \tan \alpha_{ap} \quad (6.45)$$

$$k_3 = 0,6 + 8,3 \tan \alpha_{ap} - 7,8 \tan^2 \alpha_{ap} \quad (6.46)$$

$$k_4 = 6 \tan^2 \alpha_{ap} \quad (6.47)$$

$$r = r_{in} + 0,5 h_{ap} \quad (6.48)$$

where:

$M_{ap,d}$  is the design moment at the apex;

$h_{ap}$  is the depth of the beam at the apex, see Figure 6.9;

$b$  is the width of the beam;

$r_{in}$  is the inner radius, see Figure 6.9;

$\alpha_{ap}$  is the angle of the taper in the middle of the apex zone, see Figure 6.9.

(5) For double tapered beams  $k_r = 1,0$ . For curved and pitched cambered beams  $k_r$  should be taken as:

$$k_r = \begin{cases} 1 & \text{for } \frac{r_{in}}{t} \geq 240 \\ 0,76 + 0,001 \frac{r_{in}}{t} & \text{for } \frac{r_{in}}{t} < 240 \end{cases} \quad (6.49)$$

where

$r_{in}$  is the inner radius, see Figure 6.9;

$t$  is the lamination thickness.

(6) In the apex zone the greatest tensile stress perpendicular to the grain,  $\sigma_{t,90,d}$ , should satisfy the following expression:

$$\sigma_{t,90,d} \leq k_{dis} k_{vol} f_{t,90,d} \quad (6.50)$$

with

$$k_{vol} = \begin{cases} 1,0 & \text{for solid timber} \\ \left( \frac{V_0}{V} \right)^{0,2} & \text{for glued laminated timber and LVL with} \\ & \text{all veneers parallel to the beam axis} \end{cases} \quad (6.51)$$

$$k_{dis} = \begin{cases} 1,4 & \text{for double tapered and curved beams} \\ 1,7 & \text{for pitched cambered beams} \end{cases} \quad (6.52)$$

where:

$k_{dis}$  is a factor which takes into account the effect of the stress distribution in the apex zone;

$k_{vol}$  is a volume factor;

$f_{t,90,d}$  is the design tensile strength perpendicular to the grain;

$V_0$  is the reference volume of 0,01m<sup>3</sup>;

$V$  is the stressed volume of the apex zone, in m<sup>3</sup>, (see Figure 6.9) and should not be taken greater than  $2V_b/3$ , where  $V_b$  is the total volume of the beam.

**[A1]** (7) For combined tension perpendicular to grain and shear the following expression should be satisfied: **[A1]**

$$\frac{\tau_d}{f_{v,d}} + \frac{\sigma_{t,90,d}}{k_{dis} k_{vol} f_{t,90,d}} \leq 1 \quad (6.53)$$

where:

$\tau_d$  is the design shear stress;

$f_{v,d}$  is the design shear strength;

$\sigma_{t,90,d}$  is the design tensile stress perpendicular to grain;

$k_{dis}$  and  $k_{vol}$  are given in (6).

(8) The greatest tensile stress perpendicular to the grain due to the bending moment should be calculated as follows:

$$\sigma_{t,90,d} = k_p \frac{6 M_{ap,d}}{b h_{ap}^2} \quad (6.54)$$

or, as an alternative to expression (6.54), as

$$\sigma_{t,90,d} = k_p \frac{6 M_{ap,d}}{b h_{ap}^2} - 0,6 \frac{p_d}{b} \quad (6.55)$$

where:

$p_d$  is the uniformly distributed load acting on the top of the beam over the apex area;

$b$  is the width of the beam;

$M_{ap,d}$  is the design moment at apex resulting in tensile stresses parallel to the inner curved edge;

with:

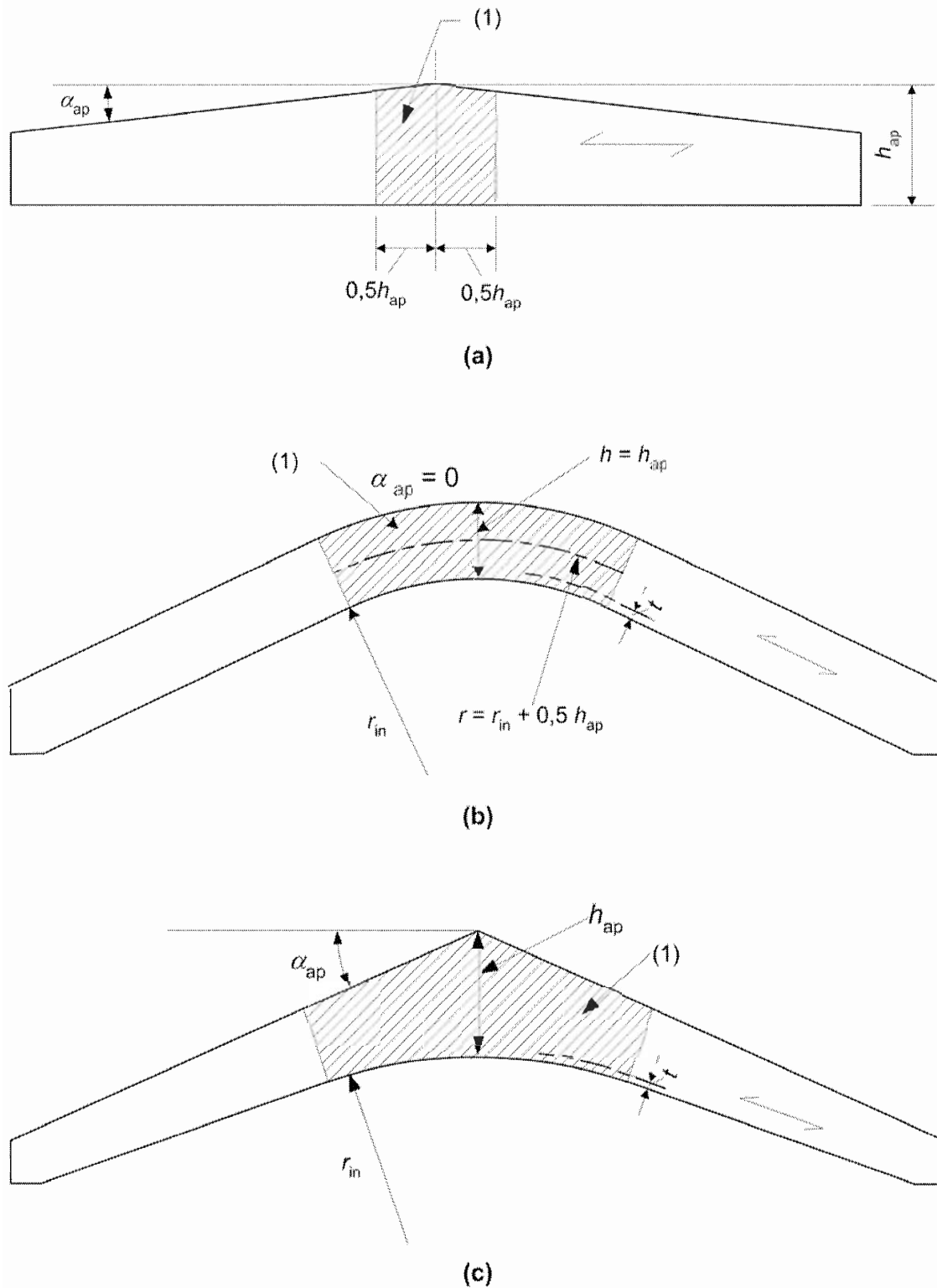
$$k_p = k_5 + k_6 \left( \frac{h_{ap}}{r} \right) + k_7 \left( \frac{h_{ap}}{r} \right)^2 \quad (6.56)$$

$$k_5 = 0,2 \tan \alpha_{ap} \quad (6.57)$$

$$k_6 = 0,25 - 1,5 \tan \alpha_{ap} + 2,6 \tan^2 \alpha_{ap} \quad (6.58)$$

$$k_7 = 2,1 \tan \alpha_{ap} - 4 \tan^2 \alpha_{ap} \quad (6.59)$$

Note: The recommended expression is (6.54). Information on the national choice between expressions (6.54) and (6.55) may be found in the National annex.



Key:  
(1) Apex Zone

NOTE: In curved and pitched cambered beams the apex zone extends over the curved parts of the beam.

**Figure 6.9 – Double tapered (a), curved (b) and pitched cambered (c) beams with the fibre direction parallel to the lower edge of the beam**

## 6.5 Notched members

### 6.5.1 General

(1)P The effects of stress concentrations at the notch shall be taken into account in the strength verification of members.

(2) The effect of stress concentrations may be disregarded in the following cases:

- tension or compression parallel to the grain;
- bending with tensile stresses at the notch if the taper is not steeper than  $1:i = 1:10$ , that is  $i \geq 10$ , see Figure 6.10a;
- bending with compressive stresses at the notch, see Figure 6.10b.

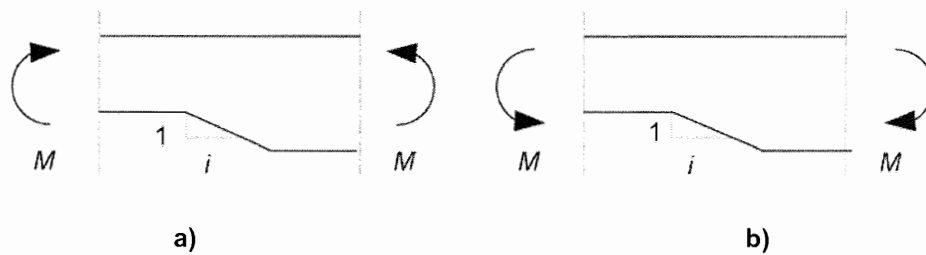


Figure 6.10 – Bending at a notch: a) with tensile stresses at the notch, b) with compressive stresses at the notch

### 6.5.2 Beams with a notch at the support

(1) For beams with rectangular cross-sections and where grain runs essentially parallel to the length of the member, the shear stresses at the notched support should be calculated using the effective (reduced) depth  $h_{ef}$  (see Figure 6.11).

(2) It should be verified that

$$\tau_d = \frac{1,5V}{bh_{ef}} \leq k_v f_{v,d} \quad (6.60)$$

where  $k_v$  is a reduction factor defined as follows:

- For beams notched at the opposite side to the support (see Figure 6.11b)

$$k_v = 1,0 \quad (6.61)$$

- For beams notched on the same side as the support (see Figure 6.11a)

$$k_v = \min \left\{ \begin{array}{l} 1 \\ k_n \left( 1 + \frac{1,1 i^{1,5}}{\sqrt{h}} \right) \\ \frac{1}{\sqrt{h} \left( \sqrt{\alpha(1-\alpha)} + 0,8 \frac{x}{h} \sqrt{\frac{1}{\alpha} - \alpha^2} \right)} \end{array} \right. \quad (6.62)$$

where:

- $i$  is the notch inclination (see Figure 6.11a);  
 $h$  is the beam depth in mm;  
 $x$  is the distance from the line of action of the support reaction to the corner of the notch, in mm;

$$\alpha = \frac{c_f h}{h}$$

$$k_n = \begin{cases} 4,5 & \text{for LVL} \\ 5 & \text{for solid timber} \\ 6,5 & \text{for glued laminated timber} \end{cases} \quad (6.63)$$

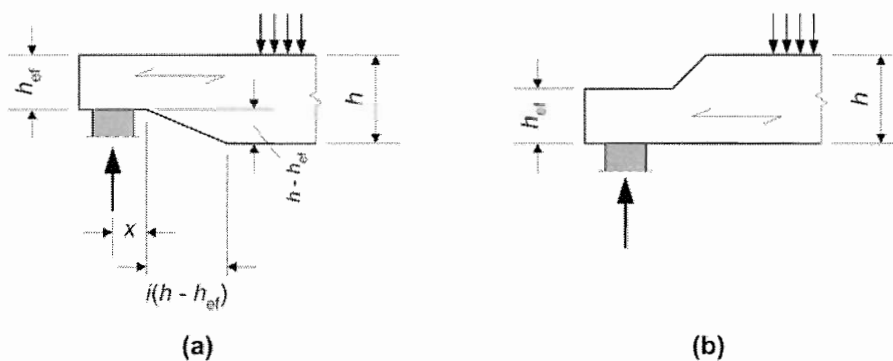


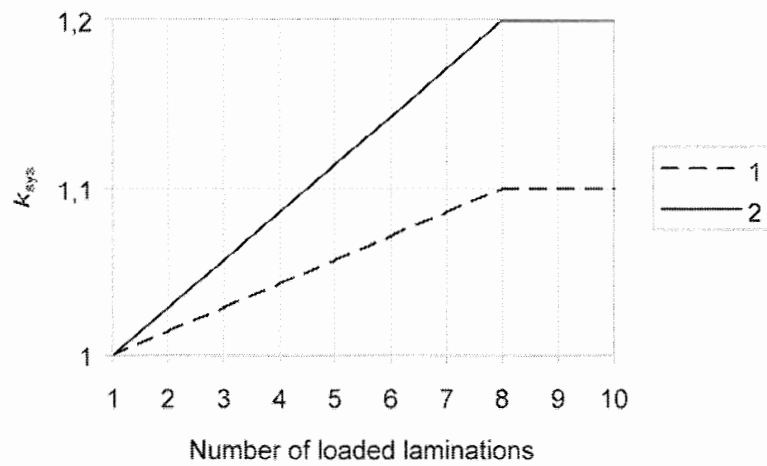
Figure 6.11 – End-notched beams

## 6.6 System strength

- (1) When several equally spaced similar members, components or assemblies are laterally connected by a continuous load distribution system, the member strength properties may be multiplied by a system strength factor  $k_{sys}$ .
- (2) Provided the continuous load-distribution system is capable of transferring the loads from one member to the neighbouring members, the factor  $k_{sys}$  should be 1,1.
- (3) The strength verification of the load distribution system should be carried out assuming the loads are of short-term duration.

NOTE: For roof trusses with a maximum centre to centre distance of 1,2 m it may be assumed that tiling battens, purlins or panels can transfer the load to the neighbouring trusses provided that these load-distribution members are continuous over at least two spans, and any joints are staggered.

- (4) For laminated timber decks or floors the values of  $k_{sys}$  given in Figure 6.12 should be used.



Key:

1 Nailed or screwed laminations

2 Laminations pre-stressed or glued together

Figure 6.12 – System strength factor  $k_{sys}$  for laminated deck plates of solid timber or glued laminated members

## Section 7 Serviceability limit states

### 7.1 Joint slip

(1) For joints made with dowel-type fasteners the slip modulus  $K_{ser}$  per shear plane per fastener under service load should be taken from Table 7.1 with  $\rho_m$  in kg/m<sup>3</sup> and  $d$  or  $d_c$  in mm. For the definition of  $d_c$ , see EN 13271.

NOTE: In EN 26891 the symbol used is  $k_s$  instead of  $K_{ser}$ .

**Table 7.1 – Values of  $K_{ser}$  for fasteners and connectors in N/mm in timber-to-timber and wood-based panel-to-timber connections**

Fastener type	$K_{ser}$
Dowels Bolts with or without clearance <sup>a</sup> Screws Nails (with pre-drilling)	$\rho_m^{1,5} d / 23$
Nails (without pre-drilling)	$\rho_m^{1,5} d^{0,8} / 30$
Staples	$\rho_m^{1,5} d^{0,8} / 80$
Split-ring connectors type A according to EN 912 Shear-plate connectors type B according to EN 912	$\rho_m d_c / 2$
Toothed-plate connectors:	
– Connectors types C1 to C9 according to EN 912	$1,5 \rho_m d_c / 4$
– Connectors type C10 and C11 according to EN 912	$\rho_m d_c / 2$
<sup>a</sup> The clearance should be added separately to the deformation.	

(2) If the mean densities  $\rho_{m,1}$  and  $\rho_{m,2}$  of the two jointed wood-based members are different then  $\rho_m$  in the above expressions should be taken as

$$\rho_m = \sqrt{\rho_{m,1} \rho_{m,2}} \quad (7.1)$$

(3) For steel-to-timber or concrete-to-timber connections,  $K_{ser}$  should be based on  $\rho_m$  for the timber member and may be multiplied by 2,0.

### 7.2 Limiting values for deflections of beams

(1) The components of deflection resulting from a combination of actions (see 2.2.3(5)) are shown in Figure 7.1, where the symbols are defined as follows, see 2.2.3:

- $w_c$  is the precamber (if applied);
- $w_{inst}$  is the instantaneous deflection;
- $w_{creep}$  is the creep deflection;
- $w_{fin}$  is the final deflection;
- $w_{net,fin}$  is the net final deflection.

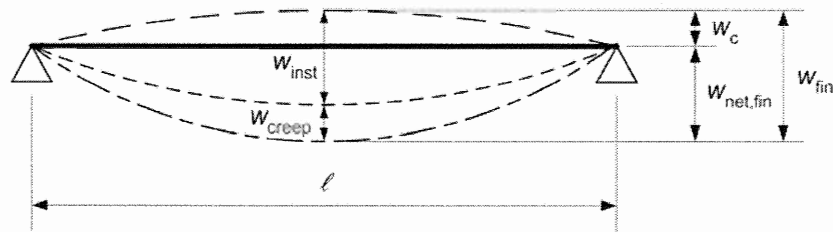


Figure 7.1 – Components of deflection

(2) The net deflection below a straight line between the supports,  $w_{\text{net,fin}}$ , should be taken as:

$$w_{\text{net,fin}} = w_{\text{inst}} + w_{\text{creep}} - w_{\text{c}} = w_{\text{fin}} - w_{\text{c}} \quad (7.2)$$

NOTE: The recommended range of limiting values of deflections for beams with span  $\ell$  is given in Table 7.2 depending upon the level of deformation deemed to be acceptable. Information on National choice may be found in the National annex.

Table 7.2 – Examples of limiting values for deflections of beams

	$w_{\text{inst}}$	$w_{\text{net,fin}}$	$w_{\text{fin}}$
Beam on two supports	$\ell/300$ to $\ell/500$	$\ell/250$ to $\ell/350$	$\ell/150$ to $\ell/300$
Cantilevering beams	$\ell/150$ to $\ell/250$	$\ell/125$ to $\ell/175$	$\ell/75$ to $\ell/150$

### 7.3 Vibrations

#### 7.3.1 General

(1)P It shall be ensured that the actions which can be reasonably anticipated on a member, component or structure, do not cause vibrations that can impair the function of the structure or cause unacceptable discomfort to the users.

(2) The vibration level should be estimated by measurements or by calculation taking into account the expected stiffness of the member, component or structure and the modal damping ratio.

(3) For floors, unless other values are proven to be more appropriate, a modal damping ratio of  $\zeta = 0,01$  (i.e 1 %) should be assumed.

#### 7.3.2 Vibrations from machinery

(1)P Vibrations caused by rotating machinery and other operational equipment shall be limited for the unfavourable combinations of permanent load and variable loads that can be expected.

(2) For floors, acceptable levels for continuous vibration should be taken from figure 5a in Appendix A of ISO 2631-2 with a multiplying factor of 1,0.

#### 7.3.3 Residential floors

(1) For residential floors with a fundamental frequency less than 8 Hz ( $f_1 \leq 8 \text{ Hz}$ ) a special investigation should be made.

(2) For residential floors with a fundamental frequency greater than 8 Hz ( $f_1 > 8 \text{ Hz}$ ) the following requirements should be satisfied:

$$\frac{w}{F} \leq a \quad \text{mm/kN} \quad (7.3)$$

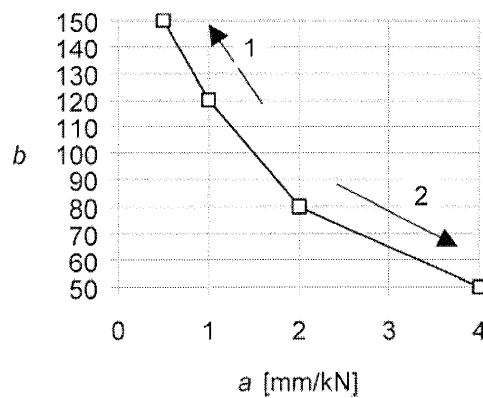
and

$$v \leq b(f_1 \zeta^{-1}) \quad \text{m/(Ns}^2\text{)} \quad (7.4)$$

where:

- $w$  is the maximum instantaneous vertical deflection caused by a vertical concentrated static force  $F$  applied at any point on the floor, taking account of load distribution;
- $v$  is the unit impulse velocity response, i.e. the maximum initial value of the vertical floor vibration velocity (in m/s) caused by an ideal unit impulse (1 Ns) applied at the point of the floor giving maximum response. Components above 40 Hz may be disregarded;
- $\zeta$  is the modal damping ratio.

NOTE: The recommended range of limiting values of  $a$  and  $b$  and the recommended relationship between  $a$  and  $b$  is given in Figure 7.2. Information on the National choice may be found in the National annex.



Key:  
1 Better performance  
2 Poorer performance

**Figure 7.2 — Recommended range of and relationship between  $a$  and  $b$**

(3) The calculations in 7.3.3(2) should be made under the assumption that the floor is unloaded, i.e., only the mass corresponding to the self-weight of the floor and other permanent actions.

(4) For a rectangular floor with overall dimensions  $\ell \times b$ , simply supported along all four edges and with timber beams having a span  $\ell$ , the fundamental frequency  $f_1$  may approximately be calculated as

$$f_1 = \frac{\pi}{2\ell^2} \sqrt{\frac{(EI)_\ell}{m}} \quad (7.5)$$

where:

- $m$  is the mass per unit area in kg/m²;
- $\ell$  is the floor span, in m;
- $(EI)_\ell$  is the equivalent plate bending stiffness of the floor about an axis perpendicular to the beam direction, in Nm²/m.

(5) For a rectangular floor with overall dimensions  $b \times \ell$ , simply supported along all four edges, the value  $v$  may, as an approximation, be taken as:

$$v = \frac{4(0,4 + 0,6 n_{40})}{mb\ell + 200} \quad (7.6)$$

where:

$v$  is the unit impulse velocity response, in  $\text{m}/(\text{Ns}^2)$ ;

$n_{40}$  is the number of first-order modes with natural frequencies up to 40 Hz;

$b$  is the floor width, in m;

$m$  is the mass, in  $\text{kg}/\text{m}^2$ ;

$\ell$  is the floor span, in m.

The value of  $n_{40}$  may be calculated from:

$$n_{40} = \left\{ \left[ \left( \frac{40}{f_1} \right)^2 - 1 \right] \left( \frac{b}{\ell} \right)^4 \frac{(EI)_\ell}{(EI)_b} \right\}^{0,25} \quad (7.7)$$

where  $(EI)_b$  is the equivalent plate bending stiffness, in  $\text{Nm}^2/\text{m}$ , of the floor about an axis parallel to the beams, where  $(EI)_b < (EI)_\ell$ .

## Section 8 Connections with metal fasteners

### 8.1 General

#### 8.1.1 Fastener requirements

(1)P Unless rules are given in this section, the characteristic load-carrying capacity, and the stiffness of the connections shall be determined from tests according to EN 1075, EN 1380, EN 1381, EN 26891 and EN 28970. If the relevant standards describe tension and compression tests, the tests for the determination of the characteristic load-carrying capacity shall be performed in tension.

#### 8.1.2 Multiple fastener connections

(1)P The arrangement and sizes of the fasteners in a connection, and the fastener spacings, edge and end distances shall be chosen so that the expected strength and stiffness can be obtained.

(2)P It shall be taken into account that the load-carrying capacity of a multiple fastener connection, consisting of fasteners of the same type and dimension, may be lower than the summation of the individual load-carrying capacities for each fastener.

(3) When a connection comprises different types of fasteners, or when the stiffness of the connections in respective shear planes of a multiple shear plane connection is different, their compatibility should be verified.

(4) For one row of fasteners parallel to the grain direction, the effective characteristic load-carrying capacity parallel to the row,  $F_{v,ef,Rk}$ , should be taken as:

$$F_{v,ef,Rk} = n_{ef} F_{v,Rk} \quad (8.1)$$

where:

$F_{v,ef,Rk}$  is the effective characteristic load-carrying capacity of one row of fasteners parallel to the grain;

$n_{ef}$  is the effective number of fasteners in line parallel to the grain;

$F_{v,Rk}$  is the characteristic load-carrying capacity of each fastener parallel to the grain.

NOTE: Values of  $n_{ef}$  for rows parallel to grain are given in 8.3.1.1(8) and 8.5.1.1(4).

(5) For a force acting at an angle to the direction of the row, it should be verified that the force component parallel to the row is less than or equal to the load-carrying capacity calculated according to expression (8.1).

#### 8.1.3 Multiple shear plane connections

(1) In multiple shear plane connections the resistance of each shear plane should be determined by assuming that each shear plane is part of a series of three-member connections.

**[A1]** (2) To be able to combine the resistance from individual shear planes in a multiple shear plane connection, the governing failure mode of the fasteners in the respective shear planes should be compatible with each other and should not consist of a combination of failure modes (a), (b), (g) and (h) from Figure 8.2 or modes (c), (f) and (j/l) from Figure 8.3 with the other failure modes. **[A1]**

#### 8.1.4 Connection forces at an angle to the grain

(1)P When a force in a connection acts at an angle to the grain, (see Figure 8.1), the possibility

of splitting caused by the tension force component,  $F_{Ed} \sin \alpha$ , perpendicular to the grain, shall be taken into account.

(2)P To take account of the possibility of splitting caused by the tension force component,  $F_{Ed} \sin \alpha$ , perpendicular to the grain, the following shall be satisfied:

$$F_{v,Ed} \leq F_{90,Rd} \quad (8.2)$$

with

$$F_{v,Ed} = \max \begin{cases} F_{v,Ed,1} \\ F_{v,Ed,2} \end{cases} \quad (8.3)$$

where:

$F_{90,Rd}$  is the design splitting capacity, calculated from the characteristic splitting capacity  $F_{90,Rk}$  according to 2.4.3;

$F_{v,Ed,1}$ ,  $F_{v,Ed,2}$  are the design shear forces on either side of the connection. (see Figure 8.1).

(3) For softwoods, the characteristic splitting capacity for the arrangement shown in Figure 8.1 should be taken as:

$$F_{90,Rk} = 14 b w \sqrt{\frac{h_e}{1 - \frac{h_e}{h}}} \quad (8.4)$$

where:

$$w = \begin{cases} \max \left\{ \left( \frac{w_{pl}}{100} \right)^{0,35} \right. & \text{for punched metal plate fasteners} \\ 1 & \text{for all other fasteners} \end{cases} \quad (8.5)$$

and:

$F_{90,Rk}$  is the characteristic splitting capacity, in N;

$w$  is a modification factor;

$h_e$  is the loaded edge distance to the centre of the most distant fastener or to the edge of the punched metal plate fastener, in mm;

$h$  is the timber member height, in mm;

$b$  is the member thickness, in mm;

$w_{pl}$  is the width of the punched metal plate fastener parallel to the grain, in mm.

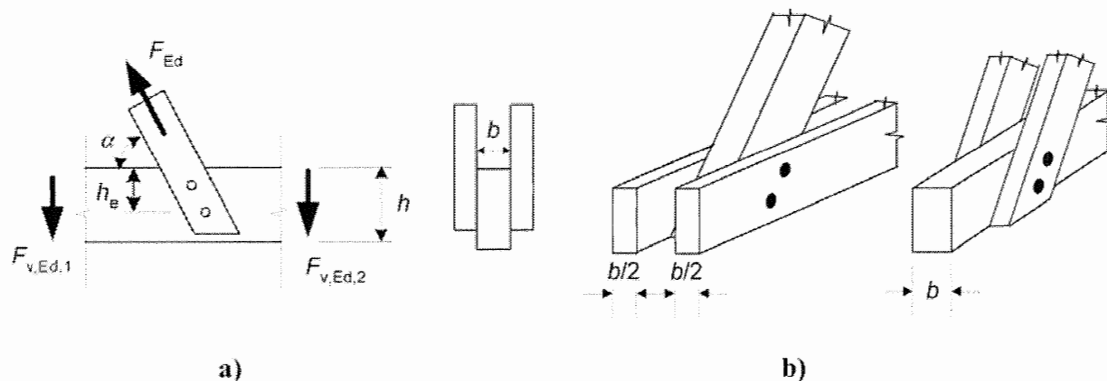


Figure 8.1 – Inclined force transmitted by a connection

### 8.1.5 Alternating connection forces

(1)P The characteristic load-carrying capacity of a connection shall be reduced if the connection is subject to alternating internal forces due to long-term or medium-term actions.

(2)The effect on connection strength of long-term or medium-term actions alternating between a tensile design force  $F_{t,Ed}$  and a compressive design force  $F_{c,Ed}$  should be taken into account by designing the connection for  $(F_{t,Ed} + 0,5F_{c,Ed})$  and  $(F_{c,Ed} + 0,5F_{t,Ed})$ .

## 8.2 Lateral load-carrying capacity of metal dowel-type fasteners

### 8.2.1 General

(1)P For the determination of the characteristic load-carrying capacity of connections with metal dowel-type fasteners the contributions of the yield strength, the embedment strength, and the withdrawal strength of the fastener shall be considered.

### 8.2.2 Timber-to-timber and panel-to-timber connections

(1) The characteristic load-carrying capacity for nails, staples, bolts, dowels and screws per shear plane per fastener, should be taken as the minimum value found from the following expressions:

– For fasteners in single shear

$$F_{v,Rk} = \min \left\{ \begin{array}{ll} f_{h,1,k} t_1 d & (a) \\ f_{h,2,k} t_2 d & (b) \\ \frac{f_{h,1,k} t_1 d}{1 + \beta} \left[ \sqrt{\beta + 2\beta^2 \left[ 1 + \frac{t_2}{t_1} + \left( \frac{t_2}{t_1} \right)^2 \right] + \beta^3 \left( \frac{t_2}{t_1} \right)^2} - \beta \left( 1 + \frac{t_2}{t_1} \right) \right] + \frac{F_{ax,Rk}}{4} & (c) \\ 1,05 \frac{f_{h,1,k} t_1 d}{2 + \beta} \left[ \sqrt{2\beta(1 + \beta) + \frac{4\beta(2 + \beta)M_{y,Rk}}{f_{h,1,k} d t_1^2}} - \beta \right] + \frac{F_{ax,Rk}}{4} & (d) \\ 1,05 \frac{f_{h,1,k} t_2 d}{1 + 2\beta} \left[ \sqrt{2\beta^2(1 + \beta) + \frac{4\beta(1 + 2\beta)M_{y,Rk}}{f_{h,1,k} d t_2^2}} - \beta \right] + \frac{F_{ax,Rk}}{4} & (e) \\ 1,15 \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2M_{y,Rk} f_{h,1,k} d} + \frac{F_{ax,Rk}}{4} & (f) \end{array} \right. \quad (8.6)$$

– For fasteners in double shear:

$$F_{v,Rk} = \min \left\{ \begin{array}{ll} f_{h,1,k} t_1 d & (g) \\ 0,5 f_{h,2,k} t_2 d & (h) \\ 1,05 \frac{f_{h,1,k} t_1 d}{2 + \beta} \left[ \sqrt{2\beta(1 + \beta) + \frac{4\beta(2 + \beta)M_{y,Rk}}{f_{h,1,k} d t_1^2}} - \beta \right] + \frac{F_{ax,Rk}}{4} & (j) \\ 1,15 \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2M_{y,Rk} f_{h,1,k} d} + \frac{F_{ax,Rk}}{4} & (k) \end{array} \right. \quad (8.7)$$

with

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} \quad (8.8)$$

where:

- $F_{v,Rk}$  is the characteristic load-carrying capacity per shear plane per fastener;
- $t_i$  is the timber or board thickness or penetration depth, with  $i$  either 1 or 2, see also 8.3 to 8.7 ;
- $f_{h,i,k}$  is the characteristic embedment strength in timber member  $i$ ;
- $d$  is the fastener diameter;
- $M_{y,Rk}$  is the characteristic fastener yield moment;
- $\beta$  is the ratio between the embedment strength of the members;
- $F_{ax,Rk}$  is the characteristic axial withdrawal capacity of the fastener, see (2).

NOTE: Plasticity of joints can be assured when relatively slender fasteners are used. In that case, failure modes (f) and (k) are governing.

(2) In the expressions (8.6) and (8.7), the first term on the right hand side is the load-carrying capacity according to the Johansen yield theory, whilst the second term  $F_{ax,Rk}/4$  is the contribution from the rope effect. The contribution to the load-carrying capacity due to the rope effect should be limited to following percentages of the Johansen part:

- Round nails 15 %
- Square and grooved nails 25 %
- Other nails 50 %
- Screws 100%
- Bolts 25 %
- Dowels 0 %

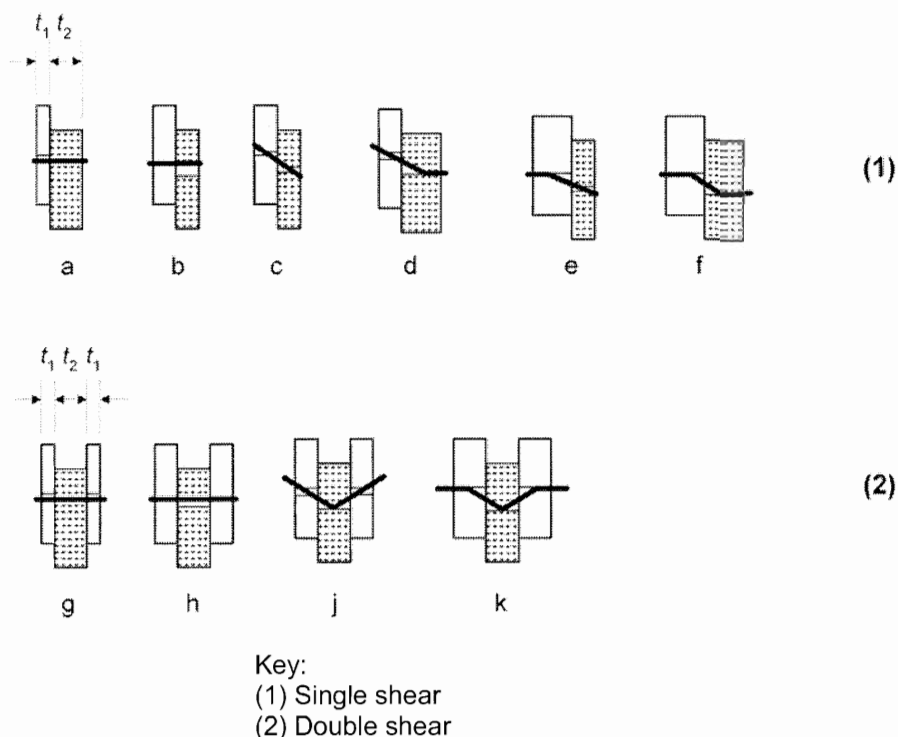
If  $F_{ax,Rk}$  is not known then the contribution from the rope effect should be taken as zero.

For single shear fasteners the characteristic withdrawal capacity,  $F_{ax,Rk}$ , is taken as the lower of the capacities in the two members. The different modes of failure are illustrated in Figure 8.2. For the withdrawal capacity,  $F_{ax,Rk}$ , of bolts the resistance provided by the washers may be taken into account, see 8.5.2(2).

(3) If no design rules are given below, the characteristic embedment strength  $f_{h,k}$  should be determined according to EN 383 and EN 14358.

(4) If no design rules are given below, the characteristic yield moment  $M_{y,Rk}$  should be determined

according to EN 409 and EN 14358.



NOTE: The letters correspond to the references of the expressions (8.6) and (8.7)

**Figure 8.2 – Failure modes for timber and panel connections.**

### 8.2.3 Steel-to-timber connections

(1) The characteristic load-carrying capacity of a steel-to-timber connection depends on the thickness of the steel plates. Steel plates of thickness less than or equal to  $0,5d$  are classified as thin plates and steel plates of thickness greater than or equal to  $d$  with the tolerance on hole diameters being less than  $0,1d$  are classified as thick plates. The characteristic load-carrying capacity of connections with steel plate thickness between a thin and a thick plate should be calculated by linear interpolation between the limiting thin and thick plate values.

(2)P The strength of the steel plate shall be checked.

(3) The characteristic load-carrying capacity for nails, bolts, dowels and screws per shear plane per fastener should be taken as the minimum value found from the following expressions:

– For a thin steel plate in single shear:

$$F_{v,Rk} = \min \left\{ \begin{array}{l} 0,4 f_{h,k} t_1 d \\ 1,15 \sqrt{2 M_{y,Rk} f_{h,k} d} + \frac{F_{ax,Rk}}{4} \end{array} \right. \quad \begin{array}{l} (a) \\ (b) \end{array} \quad (8.9)$$

– For a thick steel plate in single shear:

$$F_{v,Rk} = \min \begin{cases} f_{h,k} t_1 d & (c) \\ f_{h,k} t_1 d \left[ \sqrt{2 + \frac{4M_{y,Rk}}{f_{h,k} d t_1^2}} - 1 \right] + \frac{F_{ax,Rk}}{4} & (d) \\ 2,3 \sqrt{M_{y,Rk} f_{h,k} d} + \frac{F_{ax,Rk}}{4} & (e) \end{cases} \quad (8.10)$$

– For a steel plate of any thickness as the central member of a double shear connection:

$$F_{v,Rk} = \min \begin{cases} f_{h,l,k} t_1 d & (f) \\ f_{h,l,k} t_1 d \left[ \sqrt{2 + \frac{4M_{y,Rk}}{f_{h,l,k} d t_1^2}} - 1 \right] + \frac{F_{ax,Rk}}{4} & (g) \\ 2,3 \sqrt{M_{y,Rk} f_{h,l,k} d} + \frac{F_{ax,Rk}}{4} & (h) \end{cases} \quad (8.11)$$

– For thin steel plates as the outer members of a double shear connection:

$$F_{v,Rk} = \min \begin{cases} 0,5 f_{h,2,k} t_2 d & (j) \\ 1,15 \sqrt{2 M_{y,Rk} f_{h,2,k} d} + \frac{F_{ax,Rk}}{4} & (k) \end{cases} \quad (8.12)$$

– For thick steel plates as the outer members of a double shear connection:

$$F_{v,Rk} = \min \begin{cases} 0,5 f_{h,2,k} t_2 d & (l) \\ 2,3 \sqrt{M_{y,Rk} f_{h,2,k} d} + \frac{F_{ax,Rk}}{4} & (m) \end{cases} \quad (8.13)$$

where:

- $F_{v,Rk}$  is the characteristic load-carrying capacity per shear plane per fastener;
- $f_{h,k}$  is the characteristic embedment strength in the timber member;
- $t_1$  is the smaller of the thickness of the timber side member or the penetration depth;
- $t_2$  is the thickness of the timber middle member;
- $d$  is the fastener diameter;
- $M_{y,Rk}$  is the characteristic fastener yield moment;
- $F_{ax,Rk}$  is the characteristic withdrawal capacity of the fastener.

NOTE 1: The different failure modes are illustrated in Figure 8.3

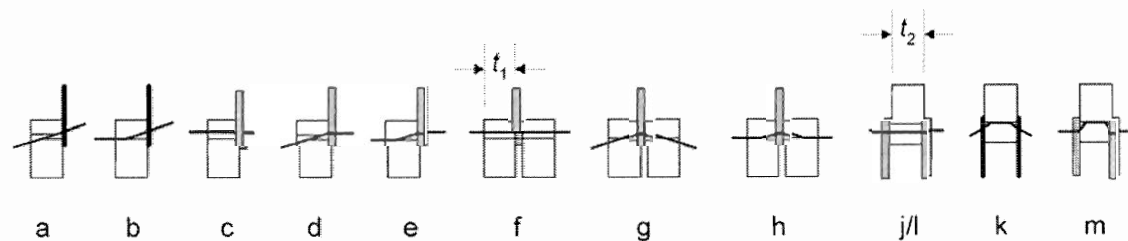


Figure 8.3 – Failure modes for steel-to-timber connections

(4) For the limitation of the rope effect  $F_{ax,Rk}$  8.2.2(2) applies.

(5)P It shall be taken into account that the load-carrying capacity of steel-to-timber connections with a loaded end may be reduced by failure along the perimeter of the fastener group.

NOTE: A method of determining the strength of the fastener group is given in Annex A (informative).

### 8.3 Nailed connections

#### 8.3.1 Laterally loaded nails

##### 8.3.1.1 General

(1) The symbols for the thicknesses in single and double shear connections (see Figure 8.4) are defined as follows:

$t_1$  is:

the headside thickness in a single shear connection;

the minimum of the head side timber thickness and the pointside penetration in a double shear connection;

$t_2$  is:

the pointside penetration in a single shear connection;

the central member thickness in a double shear connection.

**[A1]** (2) Timber should be pre-drilled when:

- the characteristic density of the timber is greater than 500 kg/m<sup>3</sup>;
- the diameter  $d$  of the nail exceeds 6 mm. **[A1]**

(3) For square and grooved nails, the nail diameter  $d$  should be taken as the side dimension.

(4) For smooth nails produced from wire with a minimum tensile strength of 600 N/mm<sup>2</sup>, the following characteristic values for yield moment should be used:

$$M_{y,Rk} = \begin{cases} 0,3 f_u d^{2,6} & \text{for round nails} \\ 0,45 f_u d^{2,6} & \text{for square and grooved nails} \end{cases} \quad (8.14)$$

where:

$M_{y,Rk}$  is the characteristic value for the yield moment, in Nmm;

$d$  is the nail diameter as defined in EN 14592, in mm;

$f_u$  is the tensile strength of the wire, in N/mm<sup>2</sup>.

(5) For nails with diameters up to 8 mm, the following characteristic embedment strengths in timber and LVL apply:

- without predrilled holes

$$f_{h,k} = 0,082 \rho_k d^{-0,3} \quad \text{N/mm}^2 \quad (8.15)$$

- with predrilled holes

$$f_{h,k} = 0,082 (1 - 0,01 d) \rho_k \quad \text{N/mm}^2 \quad (8.16)$$

where:

$\rho_k$  is the characteristic timber density, in kg/m<sup>3</sup>;

$d$  is the nail diameter, in mm.

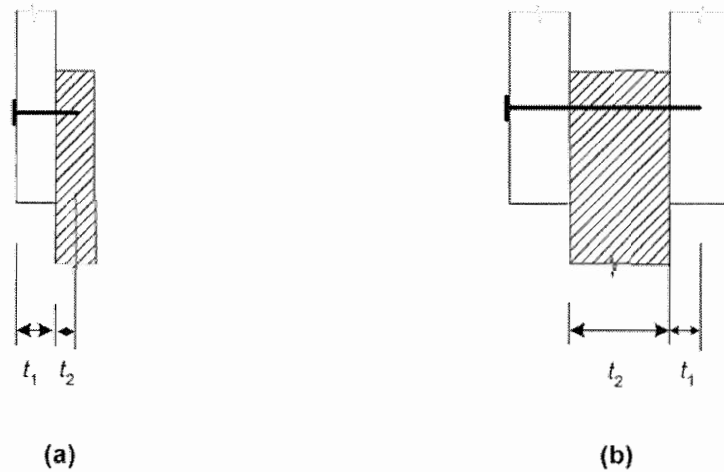


Figure 8.4 – Definitions of  $t_1$  and  $t_2$  (a) single shear connection, (b) double shear connection

(6) For nails with diameters greater than 8 mm the characteristic embedment strength values for bolts according to 8.5.1 apply.

(7) In a three-member connection, nails may overlap in the central member provided  $(t - t_2)$  is greater than  $4d$  (see Figure 8.5).

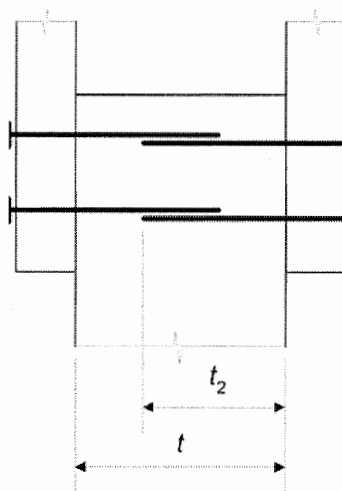


Figure 8.5 – Overlapping nails

(8) For one row of  $n$  nails parallel to the grain, unless the nails of that row are staggered perpendicular to grain by at least  $1d$  (see figure 8.6), the load-carrying capacity parallel to the grain (see 8.1.2(4)) should be calculated using the effective number of fasteners  $n_{ef}$ , where:

$$n_{ef} = n^{k_{ef}} \quad (8.17)$$

where:

$n_{ef}$  is the effective number of nails in the row;

$n$  is the number of nails in a row;

$k_{ef}$  is given in Table 8.1.

Table 8.1 – Values of  $k_{ef}$

Spacing <sup>a</sup>	$k_{ef}$	
	Not predrilled	Predrilled
$a_1 \geq 14d$	1,0	1,0
$a_1 = 10d$	0,85	0,85
$a_1 = 7d$	0,7	0,7
$a_1 = 4d$	-	0,5

<sup>a</sup> For intermediate spacings, linear interpolation of  $k_{ef}$  is permitted

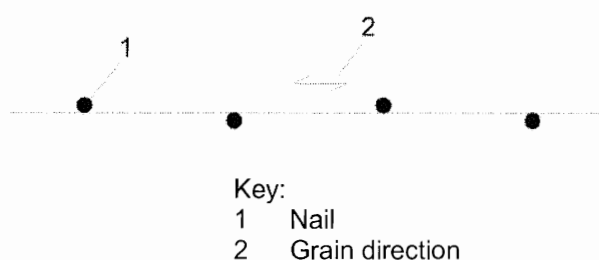


Figure 8.6 – Nails in a row parallel to grain staggered perpendicular to grain by  $d$

(9) There should be at least two nails in a connection.

(10) Requirements for structural detailing and control of nailed connections are given in 10.4.2.

### 8.3.1.2 Nailed timber-to-timber connections

(1) For smooth nails the pointside penetration length should be at least  $8d$ .

(2) For nails other than smooth nails, as defined in EN 14592, the pointside penetration length should be at least  $6d$ .

(3) Nails in end grain should not be considered capable of transmitting lateral forces.

(4) As an alternative to 8.3.1.2(3), for nails in end grain the following rules apply:

- In secondary structures smooth nails may be used. The design values of the load-carrying capacity should be taken as 1/3 of the values for nails installed at right angles to the grain;
- Nails other than smooth nails, as defined in EN 14592, may be used in structures other than secondary structures. The design values of the load-carrying capacity should be taken as 1/3 of the values for smooth nails of equivalent diameter installed at right angles to the grain, provided that:
  - the nails are only laterally loaded;
  - there are at least three nails per connection;
  - the pointside penetration is at least  $10d$ ;
  - the connection is not exposed to service class 3 conditions;
  - the prescribed spacings and edge distances given in Table 8.2 are satisfied.

Note 1: An example of a secondary structure is a fascia board nailed to rafters.

Note 2: The recommended application rule is given in 8.3.1.2(3). The National choice may be specified in the National annex.

(5) Minimum spacings and edge and end distances are given in Table 8.2, where (see Figure 8.7):

$a_1$  is the spacing of nails within one row parallel to grain;

$a_2$  is the spacing of rows of nails perpendicular to grain;

$a_{3,c}$  is the distance between nail and unloaded end;

$a_{3,t}$  is the distance between nail and loaded end;

$a_{4,c}$  is the distance between nail and unloaded edge;

$a_{4,t}$  is the distance between nail and loaded edge;

$\alpha$  is the angle between the force and the grain direction.

Table 8.2 – Minimum spacings and edge and end distances for nails

Spacing or distance (see Figure 8.7)	Angle $\alpha$	Minimum spacing or end/edge distance		
		without predrilled holes		with predrilled holes
		$\rho_k \leq 420 \text{ kg/m}^3$	$420 \text{ kg/m}^3 < \rho_k \leq 500 \text{ kg/m}^3$	
Spacing $a_1$ (parallel to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$d < 5 \text{ mm:}$ $(5+5  \cos \alpha ) d$ $d \geq 5 \text{ mm:}$ $(5+7  \cos \alpha ) d$	$(7+8  \cos \alpha ) d$	$(4+  \cos \alpha ) d$
Spacing $a_2$ (perpendicular to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$5d$	$7d$	$(3+  \sin \alpha ) d$
Distance $a_{3,t}$ (loaded end)	$-90^\circ \leq \alpha \leq 90^\circ$	$(10+ 5 \cos \alpha) d$	$(15+ 5 \cos \alpha) d$	$(7+ 5 \cos \alpha) d$
Distance $a_{3,c}$ (unloaded end)	$90^\circ \leq \alpha \leq 270^\circ$	$10d$	$15d$	$7d$
Distance $a_{4,t}$ (loaded edge)	$0^\circ \leq \alpha \leq 180^\circ$	$d < 5 \text{ mm:}$ $(5+2 \sin \alpha) d$ $d \geq 5 \text{ mm:}$ $(5+ 5 \sin \alpha) d$	$d < 5 \text{ mm:}$ $(7+2 \sin \alpha) d$ $d \geq 5 \text{ mm:}$ $(7+ 5 \sin \alpha) d$	$d < 5 \text{ mm:}$ $(3+ 2 \sin \alpha) d$ $d \geq 5 \text{ mm:}$ $(3+ 4 \sin \alpha) d$
Distance $a_{4,c}$ (unloaded edge)	$180^\circ \leq \alpha \leq 360^\circ$	$5d$	$7d$	$3d$

(6) Timber should be pre-drilled when the thickness of the timber members is smaller than

$$t = \max \left\{ \begin{array}{l} 7d \\ (13d - 30) \frac{\rho_k}{400} \end{array} \right. \quad (8.18)$$

where:

$t$  is the minimum thickness of timber member to avoid pre-drilling, in mm;

$\rho_k$  is the characteristic timber density in kg/m<sup>3</sup>;

$d$  is the nail diameter, in mm.

(7) Timber of species especially sensitive to splitting should be pre-drilled when the thickness of the timber members is smaller than

$$t = \max \left\{ \begin{array}{l} 14d \\ (13d - 30) \frac{\rho_k}{200} \end{array} \right. \quad (8.19)$$

Expression (8.19) may be replaced by expression (8.18) for edge distances given by:

$$a_4 \geq 10 d \quad \text{for } \rho_k \leq 420 \text{ kg/m}^3$$

$$a_4 \geq 14 d \quad \text{for } 420 \text{ kg/m}^3 \leq \rho_k \leq 500 \text{ kg/m}^3.$$

Note: Examples of species sensitive to splitting are fir (*abies alba*), Douglas fir (*pseudotsuga menziesii*) and spruce (*picea abies*). It is recommended to apply 8.3.1.2(7) for species fir (*abies alba*) and Douglas fir (*pseudotsuga menziesii*). The National choice may be specified in the National annex.

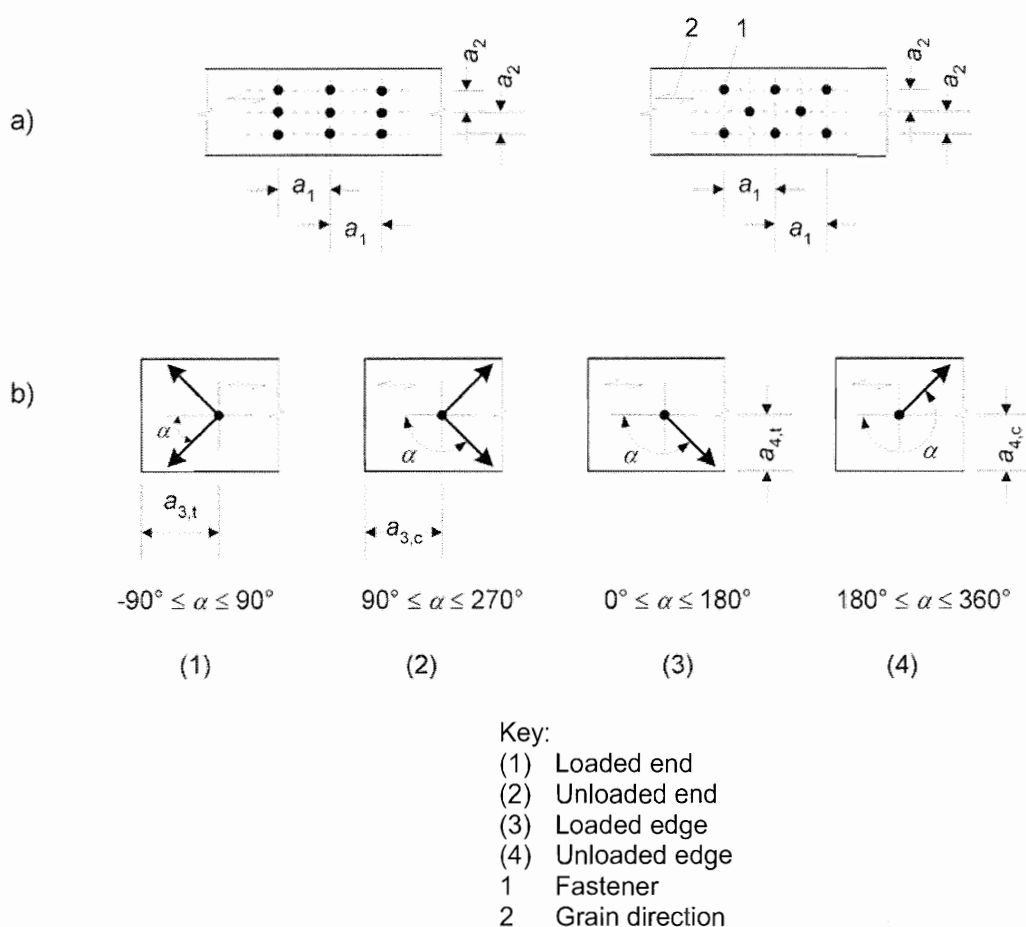


Figure 8.7 – Spacings and end and edge distances

(a) Spacing parallel to grain in a row and perpendicular to grain between rows, (b) Edge and end distances

### 8.3.1.3 Nailed panel-to-timber connections

(1) Minimum nail spacings for all nailed panel-to-timber connections are those given in Table 8.2, multiplied by a factor of 0,85. The end/edge distances for nails remain unchanged unless otherwise stated below.

(2) Minimum edge and end distances in plywood members should be taken as  $3d$  for an unloaded edge (or end) and  $(3 + 4 \sin \alpha)d$  for a loaded edge (or end), where  $\alpha$  is the angle between the direction of the load and the loaded edge (or end).

(3) For nails with a head diameter of at least  $2d$ , the characteristic embedment strengths are as follows:

– for plywood:

$$f_{h,k} = 0,11 \rho_k d^{-0,3} \quad (8.20)$$

where:

$f_{h,k}$  is the characteristic embedment strength, in  $\text{N/mm}^2$ ;

$\rho_k$  is the characteristic plywood density in  $\text{kg/m}^3$ ;

$d$  is the nail diameter, in mm;

– for hardboard in accordance with EN 622-2:

$$f_{h,k} = 30 d^{-0,3} t^{0,6} \quad (8.21)$$

where:

$f_{h,k}$  is the characteristic embedment strength, in  $\text{N/mm}^2$ ;

$d$  is the nail diameter, in mm;

$t$  is the panel thickness, in mm.

– for particleboard and OSB:

$$f_{h,k} = 65 d^{-0,7} t^{0,1} \quad (8.22)$$

where:

$f_{h,k}$  is the characteristic embedment strength, in  $\text{N/mm}^2$ ;

$d$  is the nail diameter, in mm;

$t$  is the panel thickness, in mm.

### 8.3.1.4 Nailed steel-to-timber connections

(1) The minimum edge and end distances for nails given in Table 8.2 apply. Minimum nail spacings are those given in Table 8.2, multiplied by a factor of 0,7.

### 8.3.2 Axially loaded nails

**[A1]** (1)P Nails used to resist permanent or long-term axial loading shall be threaded.

NOTE: The following definition of threaded nails is given in EN 14592: Nail that has its shank profiled or deformed over a part of its length of minimum  $4,5 d$  ( $4,5$  times the nominal diameter) and that has a characteristic withdrawal parameter  $f_{ax,k}$  greater than or equal to  $6 \text{ N/mm}^2$  when measured on timber with a characteristic density of  $350 \text{ kg/m}^3$  when conditioned to constant mass at  $20^\circ\text{C}$  and  $65\%$  relative humidity. **[A1]**

(2) For threaded nails, only the threaded part should be considered capable of transmitting axial load.

(3) Nails in end grain should be considered incapable of transmitting axial load.

(4) The characteristic withdrawal capacity of nails,  $F_{ax,Rk}$ , for nailing perpendicular to the grain (Figure 8.8 (a) and for slant nailing (Figure 8.8 (b)), should be taken as the smaller of the values

found from the following expressions:

– For nails other than smooth nails, as defined in EN 14592:

$$F_{ax,Rk} = \begin{cases} f_{ax,k} d t_{pen} & (a) \\ f_{head,k} d_h^2 & (b) \end{cases} \quad (8.23)$$

– For smooth nails:

$$F_{ax,Rk} = \begin{cases} f_{ax,k} d t_{pen} & (a) \\ f_{ax,k} d t + f_{head,k} d_h^2 & (b) \end{cases} \quad (8.24)$$

where:

- $f_{ax,k}$  is the characteristic pointside withdrawal strength;
- $f_{head,k}$  is the characteristic headside pull-through strength;
- $d$  is the nail diameter according to 8.3.1.1;
- $t_{pen}$  is the pointside penetration length or the length of the threaded part in the pointside member;
- $t$  is the thickness of the headside member;
- $d_h$  is the nail head diameter.

(5) The characteristic strengths  $f_{ax,k}$  and  $f_{head,k}$  should be determined by tests in accordance with EN 1382, EN 1383 and EN 14358 unless specified in the following.

(6) For smooth nails with a pointside penetration of at least  $12d$ , the characteristic values of the withdrawal and pull-through strengths should be found from the following expressions:

$$f_{ax,k} = 20 \times 10^{-6} \rho_k^2 \quad (8.25)$$

$$f_{head,k} = 70 \times 10^{-6} \rho_k^2 \quad (8.26)$$

where:

$\rho_k$  is the characteristic timber density in kg/m<sup>3</sup>;

(7) For smooth nails, the pointside penetration  $t_{pen}$  should be at least  $8d$ . For nails with a pointside penetration smaller than  $12d$  the withdrawal capacity should be multiplied by  $(t_{pen}/4d - 2)$ . For threaded nails, the pointside penetration should be at least  $6d$ . For nails with a pointside penetration smaller than  $8d$  the withdrawal capacity should be multiplied by  $(t_{pen}/2d - 3)$ .

(8) For structural timber which is installed at or near fibre saturation point, and which is likely to dry out under load, the values of  $f_{ax,k}$  and  $f_{head,k}$  should be multiplied by 2/3.

(9) The spacings, end and edge distances for laterally loaded nails apply to axially loaded nails.

**Ⓐ** (10) For slant nailing the distance to the loaded end should be at least  $10d$  (see Figure 8.8(b)). There should be at least two slant nails in a connection. **Ⓐ**

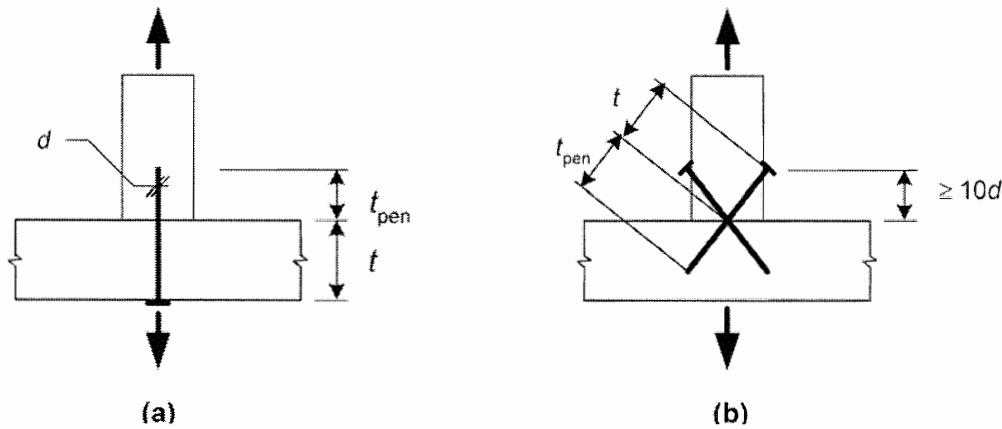


Figure 8.8 – (a) Nailing perpendicular to grain and (b) slant nailing

### 8.3.3 Combined laterally and axially loaded nails

(1) For connections subjected to a combination of axial load ( $F_{ax,Ed}$ ) and lateral load ( $F_{v,Ed}$ ) the following expressions should be satisfied:

– for smooth nails:

$$\frac{F_{ax,Ed}}{F_{ax,Rd}} + \frac{F_{v,Ed}}{F_{v,Rd}} \leq 1 \quad (8.27)$$

– for nails other than smooth nails, as defined in EN 14592:

$$\left( \frac{F_{ax,Ed}}{F_{ax,Rd}} \right)^2 + \left( \frac{F_{v,Ed}}{F_{v,Rd}} \right)^2 \leq 1 \quad (8.28)$$

where:

$F_{ax,Rd}$  and  $F_{v,Rd}$  are the design load-carrying capacities of the connection loaded with axial load or lateral load respectively.

### 8.4 Stapled connections

**[A1]** (1) The rules given in 8.3, except for 8.3.1.1(4) and (6) and 8.3.1.2(7), apply for round or nearly round or rectangular staples with bevelled or symmetrical pointed legs. **[A1]**

(2) For staples with rectangular cross-sections the diameter  $d$  should be taken as the square root of the product of both dimensions.

(3) The width  $b$  of the staple crown should be at least  $6d$ , and the pointside penetration length  $t_2$  should be at least  $14d$ , see Figure 8.9.

(4) There should be at least two staples in a connection.

(5) The lateral design load-carrying capacity per staple per shear plane should be considered as equivalent to that of two nails with the staple diameter, provided that the angle between the crown and the direction of the grain of the timber under the crown is greater than  $30^\circ$ , see Figure 8.10. If the angle between the crown and the direction of the grain under the crown is equal to or less than  $30^\circ$ , then the lateral design load-carrying capacity should be multiplied by a factor of 0,7.

(6) For staples produced from wire with a minimum tensile strength of  $800 \text{ N/mm}^2$ , the following characteristic yield moment per leg should be used:

$$M_{y,Rk} = 240 d^{2,6} \quad (8.29)$$

where:

$M_{y,Rk}$  is the characteristic yield moment, in Nmm;

$d$  is the staple leg diameter, in mm.

(7) For a row of  $n$  staples parallel to the grain, the load-carrying capacity in that direction should be calculated using the effective number of fasteners  $n_{ef}$  according to 8.3.1.1(8)

(8) Minimum staple spacings, edge and end distances are given in Table 8.3, and illustrated in Figure 8.10 where  $\theta$  is the angle between the staple crown and the grain direction.

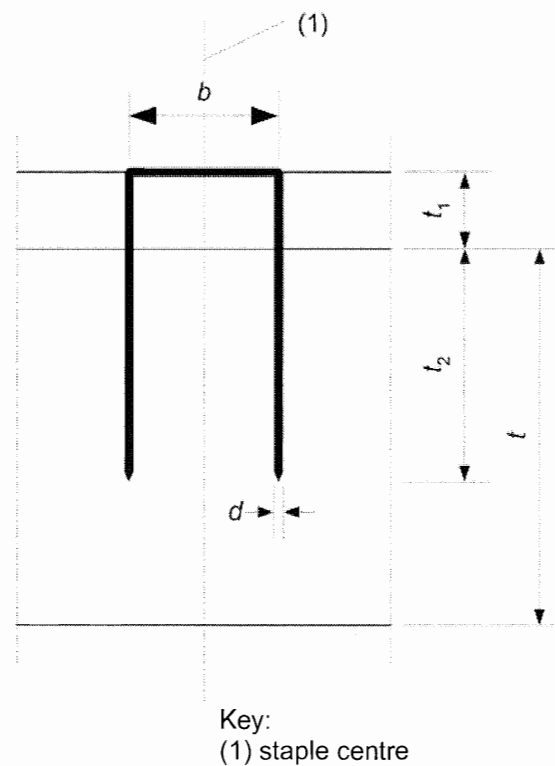


Figure 8.9 – Staple dimensions

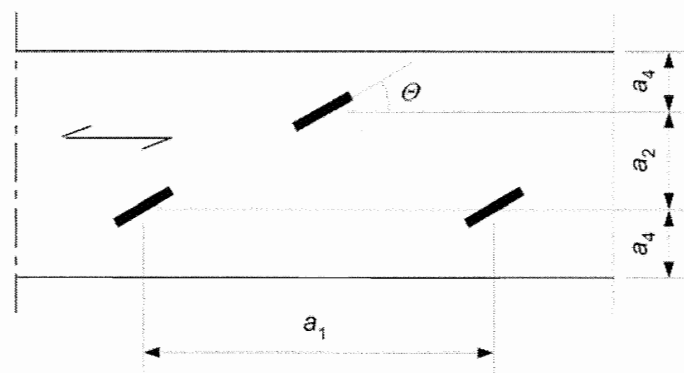


Figure 8.10 – Definition of spacing for staples

Table 8.3 – Minimum spacings and edge and end distances for staples

Spacing and edge/end distances (see Figure 8.7)	Angle	Minimum spacing or edge/end distance
$a_1$ (parallel to grain) for $\theta \geq 30^\circ$ for $\theta < 30^\circ$	$0^\circ \leq \alpha \leq 360^\circ$	$(10 + 5  \cos \alpha ) d$ $(15 + 5  \cos \alpha ) d$
$a_2$ (perpendicular to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$15 d$
$a_{3,l}$ (loaded end)	$-90^\circ \leq \alpha \leq 90^\circ$	$(15 + 5  \cos \alpha ) d$
$a_{3,e}$ (unloaded end)	$90^\circ \leq \alpha \leq 270^\circ$	$15 d$
$a_{4,l}$ (loaded edge)	$0^\circ \leq \alpha \leq 180^\circ$	$(15 + 5  \sin \alpha ) d$
$a_{4,e}$ (unloaded edge)	$180^\circ \leq \alpha \leq 360^\circ$	$10 d$

## 8.5 Bolted connections

### 8.5.1 Laterally loaded bolts

#### 8.5.1.1 General and bolted timber-to-timber connections

(1) For bolts the following characteristic value for the yield moment should be used:

$$M_{y,Rk} = 0,3 f_{u,k} d^{2,6} \quad (8.30)$$

where:

$M_{y,Rk}$  is the characteristic value for the yield moment, in Nmm;

$f_{u,k}$  is the characteristic tensile strength, in N/mm<sup>2</sup>;

$d$  is the bolt diameter, in mm.

(2) For bolts up to 30 mm diameter, the following characteristic embedment strength values in timber and LVL should be used, at an angle  $\alpha$  to the grain:

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \quad (8.31)$$

$$f_{h,0,k} = 0,082 (1 - 0,01 d) \rho_k \quad (8.32)$$

where:

$$k_{90} = \begin{cases} 1,35 + 0,015 d & \text{for softwoods} \\ 1,30 + 0,015 d & \text{for LVL} \\ 0,90 + 0,015 d & \text{for hardwoods} \end{cases} \quad (8.33)$$

and:

$f_{h,0,k}$  is the characteristic embedment strength parallel to grain, in N/mm<sup>2</sup>;

$\rho_k$  is the characteristic timber density, in kg/m<sup>3</sup>;

$\alpha$  is the angle of the load to the grain;

$d$  is the bolt diameter, in mm.

(3) Minimum spacings and edge and end distances should be taken from Table 8.4, with symbols illustrated in Figure 8.7.

Table 8.4 – Minimum values of spacing and edge and end distances for bolts

Spacing and end/edge distances (see Figure 8.7)	Angle	Minimum spacing or distance
$a_1$ (parallel to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$(4 +  \cos \alpha ) d$
$a_2$ (perpendicular to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$4 d$
$a_{3,t}$ (loaded end)	$-90^\circ \leq \alpha \leq 90^\circ$	$\max (7 d; 80 \text{ mm})$
$a_{3,c}$ (unloaded end)	$90^\circ \leq \alpha < 150^\circ$ $150^\circ \leq \alpha < 210^\circ$ $210^\circ \leq \alpha \leq 270^\circ$	$[A_1] (1 + 6 \sin \alpha) d$ $4 d$ $(1 + 6  \sin \alpha ) d [A_1]$
$a_{4,t}$ (loaded edge)	$0^\circ \leq \alpha \leq 180^\circ$	$\max [(2 + 2 \sin \alpha) d; 3d]$
$a_{4,c}$ (unloaded edge)	$180^\circ \leq \alpha \leq 360^\circ$	$3 d$

(4) For one row of  $n$  bolts parallel to the grain direction, the load-carrying capacity parallel to grain, see 8.1.2(4), should be calculated using the effective number of bolts  $n_{ef}$  where:

$$n_{ef} = \min \left\{ \begin{array}{l} n \\ n^{0,9} \sqrt[4]{\frac{a_1}{13d}} \end{array} \right. \quad (8.34)$$

where:

$a_1$  is the spacing between bolts in the grain direction;

$d$  is the bolt diameter

$n$  is the number of bolts in the row.

For loads perpendicular to grain, the effective number of fasteners should be taken as

$$n_{ef} = n \quad (8.35)$$

For angles  $0^\circ < \alpha < 90^\circ$  between load and grain direction,  $n_{ef}$  may be determined by linear interpolation between expressions (8.34) and (8.35).

(5) Requirements for minimum washer dimensions and thickness in relation to bolt diameter are given in 10.4.3

#### 8.5.1.2 Bolted panel-to-timber connections

(1) For plywood the following embedment strength, in  $\text{N/mm}^2$ , should be used at all angles to the face grain:

$$f_{h,k} = 0,11 (1 - 0,01 d) \rho_k \quad (8.36)$$

where:

$\rho_k$  is the characteristic plywood density, in  $\text{kg/m}^3$ ;

$d$  is the bolt diameter, in mm.

(2) For particleboard and OSB the following embedment strength value, in  $\text{N/mm}^2$ , should be used at all angles to the face grain:

$$f_{h,k} = 50 d^{-0,6} t^{0,2} \quad (8.37)$$

where:

$d$  is the bolt diameter, in mm;

$t$  is the panel thickness, in mm.

### 8.5.1.3 Bolted steel-to-timber connections

(1) The rules given in 8.2.3 apply.

### 8.5.2 Axially loaded bolts

(1) The axial load-bearing capacity and withdrawal capacity of a bolt should be taken as the lower value of:

- the bolt tensile capacity;
- the load-bearing capacity of either the washer or (for steel-to-timber connections) the steel plate.

(2) The bearing capacity of a washer should be calculated assuming a characteristic compressive strength on the contact area of  $3,0f_{c,90,k}$ .

(3) The bearing capacity per bolt of a steel plate should not exceed that of a circular washer with a diameter which is the minimum of:

- $12t$ , where  $t$  is the plate thickness;
- $4d$ , where  $d$  is the bolt diameter.

### 8.6 Dowelled connections

(1) The rules given in 8.5.1 except 8.5.1.1(3) apply.

(2) The dowel diameter should be greater than 6 mm and less than 30 mm.

(3) Minimum spacing and edge and end distances are given in Table 8.5, with symbols illustrated in Figure 8.7.

**Table 8.5 – Minimum spacings and edge and end distances for dowels**

Spacing and edge/end distances (see Figure 8.7)	Angle	Minimum spacing or edge/end distance
$a_1$ (parallel to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$(3 + 2  \cos \alpha ) d$
$a_2$ (perpendicular to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$3 d$
$a_{3,t}$ (loaded end)	$-90^\circ \leq \alpha \leq 90^\circ$	$\max(7 d; 80 \text{ mm})$
$a_{3,c}$ (unloaded end)	$90^\circ \leq \alpha < 150^\circ$	$\max(a_{3,t}  \sin \alpha  d; 3d)$
	$150^\circ \leq \alpha < 210^\circ$	$3 d$
	$210^\circ \leq \alpha \leq 270^\circ$	$\max(a_{3,t}  \sin \alpha  d; 3d)$
$a_{4,t}$ (loaded edge)	$0^\circ \leq \alpha \leq 180^\circ$	$\max([2 + 2 \sin \alpha] d; 3d)$
$a_{4,c}$ (unloaded edge)	$180^\circ \leq \alpha \leq 360^\circ$	$3 d$

(4) Requirements for dowel hole tolerances are given in 10.4.4.

## 8.7 Screwed connections

### 8.7.1 Laterally loaded screws

(1)P The effect of the threaded part of the screw shall be taken into account in determining the load-carrying capacity, by using an effective diameter  $d_{ef}$ .

(2) For smooth shank screws, where the outer thread diameter is equal to the shank diameter, the rules given in 8.2 apply, provided that:

- The effective diameter  $d_{ef}$  is taken as the smooth shank diameter;
- The smooth shank penetrates into the member containing the point of the screw by not less than  $4d$ .

(3) Where the conditions in (2) are not satisfied, the screw load-carrying capacity should be calculated using an effective diameter  $d_{ef}$  taken as 1,1 times the thread root diameter.

(4) For smooth shank screws with a diameter  $d > 6$  mm, the rules in 8.5.1 apply.

(5) For smooth shank screws with a diameter of 6 mm or less, the rules of 8.3.1 apply.

(6) Requirements for structural detailing and control of screwed joints are given in 10.4.5.

### 8.7.2 Axially loaded screws

**[A1]** (1)P For the verification of resistance of axially loaded screws, the following failure modes shall be taken into account:

- the withdrawal failure of the threaded part of the screw;
- the tear-off failure of the screw head of screws used in combination with steel plates, the tear-off resistance of the screw head should be greater than the tensile strength of the screw;
- the pull-through failure of the screw head;
- the tensile failure of the screw;
- the buckling failure of the screw when loaded in compression;
- failure along the circumference of a group of screws used in conjunction with steel plates (block shear or plug shear);

(2) Minimum spacings and end and edge distances for axially loaded screws, see figure 8.11a, should be taken from Table 8.6, provided the timber thickness  $t \geq 12d$ .

**Table 8.6 – Minimum spacings and end and edge distances for axially loaded screws**

Minimum screw spacing in a plane parallel to the grain	Minimum screw spacing perpendicular to a plane parallel to the grain	Minimum end distance of the centre of gravity of the threaded part of the screw in the member	Minimum edge distance of the centre of gravity of the threaded part of the screw in the member
$a_1$	$a_2$	$a_{1,CG}$	$a_{2,CG}$
$7d$	$5d$	$10d$	$4d$

**[A1]**



- $\boxed{A1}$   $F_{ax,\alpha,Rk}$  is the characteristic withdrawal capacity of the connection at an angle  $\alpha$  to the grain, in N;  
 $f_{ax,k}$  is the characteristic withdrawal strength perpendicular to the grain, in N/mm<sup>2</sup>;  
 $n_{ef}$  is the effective number of screws, see 8.7.2(8);  
 $\ell_{ef}$  is the penetration length of the threaded part, in mm;  
 $\rho_k$  is the characteristic density, in kg/m<sup>3</sup>;  
 $\alpha$  is the angle between the screw axis and the grain direction, with  $\alpha \geq 30^\circ$ .

NOTE: Failure modes in the steel or in the timber around the screw are brittle, i.e. with small ultimate deformation and therefore have a limited possibility for stress redistribution.

(5) Where the requirements with respect to the outer and inner thread diameter given in (4) are not satisfied, the characteristic withdrawal capacity,  $F_{ax,\alpha,Rk}$ , should be taken as:

$$F_{ax,\alpha,Rk} = \frac{n_{ef} f_{ax,k} d \ell_{ef}}{1,2 \cos^2 \alpha + \sin^2 \alpha} \left( \frac{\rho_k}{\rho_a} \right)^{0,8} \quad (8.40a)$$

where

- $f_{ax,k}$  is the characteristic withdrawal parameter perpendicular to the grain determined in accordance with EN 14592 for the associated density  $\rho_a$ ;  
 $\rho_a$  is the associated density for  $f_{ax,k}$ , in kg/m<sup>3</sup>  
and the other symbols are explained in (4).

(6) The characteristic pull-through resistance of connections with axially loaded screws should be taken as:

$$F_{ax,\alpha,Rk} = n_{ef} f_{head,k} d_h^2 \left( \frac{\rho_k}{\rho_a} \right)^{0,8} \quad (8.40b)$$

where:

- $F_{ax,\alpha,Rk}$  is the characteristic pull-through capacity of the connection at an angle  $\alpha$  to the grain in N, with  $\alpha \geq 30^\circ$ ;  
 $f_{head,k}$  is the characteristic pull-through parameter of the screw determined in accordance with EN 14592 for the associated density  $\rho_a$ ;  
 $d_h$  is the diameter of the screw head in mm  
and the other symbols are explained in (4).

(7) The characteristic tensile resistance of the connection (head tear-off or tensile capacity of shank),  $F_{t,Rk}$ , should be taken as:

$$F_{t,Rk} = n_{ef} f_{tens,k} \quad (8.40c)$$

where

- $f_{tens,k}$  is the characteristic tensile capacity of the screw determined in accordance with EN 14592;  
 $n_{ef}$  is the effective number of screws, see 8.7.2(8).

(8) For a connection with a group of screws loaded by a force component parallel to the shank, the effective number of screws is given by:

$$n_{ef} = n^{0,9} \quad (8.41) \quad \boxed{A1}$$

$\sqrt{A_1}$  where:

$n_{ef}$  is the effective number of screws;

$n$  is the number of screws acting together in a connection.  $\sqrt{A_1}$

### 8.7.3 Combined laterally and axially loaded screws

(1) For screwed connections subjected to a combination of axial load and lateral load, expression (8.28) should be satisfied.

## 8.8 Connections made with punched metal plate fasteners

### 8.8.1 General

(1)P Connections made with punched metal plate fasteners shall comprise punched metal plate fasteners of the same type, size and orientation, placed on each side of the timber members.

(2) The following rules apply only to punched metal plate fasteners with two orthogonal directions.

### 8.8.2 Plate geometry

(1) The symbols used to define the geometry of a punched metal plate fastener joint are given in Figure 8.11 and defined as follows:

x-direction main direction of plate;

y-direction perpendicular to the main plate direction;

$\alpha$  angle between the x-direction and the force (tension:  $0^\circ \leq \gamma < 90^\circ$ , compression:  $90^\circ \leq \gamma < 180^\circ$ );

$\beta$  angle between the grain-direction and the force;

$\gamma$  angle between the x-direction and the connection line;

$A_{ef}$  area of the total contact surface between the plate and the timber, reduced by 5 mm from the edges of the timber and by a distance in the grain direction from the end of timber equal to 6 times the fastener's nominal thickness;

$l$  dimension of the plate measured along the connection line.

### 8.8.3 Plate strength properties

(1)P The plate shall have characteristic values for the following properties, determined in accordance with EN 14545 from tests carried out in accordance with EN 1075:

$f_{a,0,0}$  the anchorage capacity per unit area for  $\alpha = 0^\circ$  and  $\beta = 0^\circ$ ;

$f_{a,90,90}$  the anchorage capacity per unit area for  $\alpha = 90^\circ$  and  $\beta = 90^\circ$ ;

$f_{t,0}$  the tension capacity per unit width of plate for  $\alpha = 0^\circ$ ;

$f_{c,0}$  the compression capacity per unit width of plate for  $\alpha = 0^\circ$ ;

$f_{v,0}$  the shear capacity per unit width of plate in the x-direction;

$f_{t,90}$  the tension capacity per unit width of plate for  $\alpha = 90^\circ$ ;

$f_{c,90}$  the compression capacity per unit width of plate for  $\alpha = 90^\circ$ ;

$f_{v,90}$  the shear capacity per unit width of plate in the y-direction;

$k_1, k_2, \alpha_0$  constants.

(2)P In order to calculate the design tension, compression and shear capacities of the plate the value of  $k_{mod}$  shall be taken as 1,0.

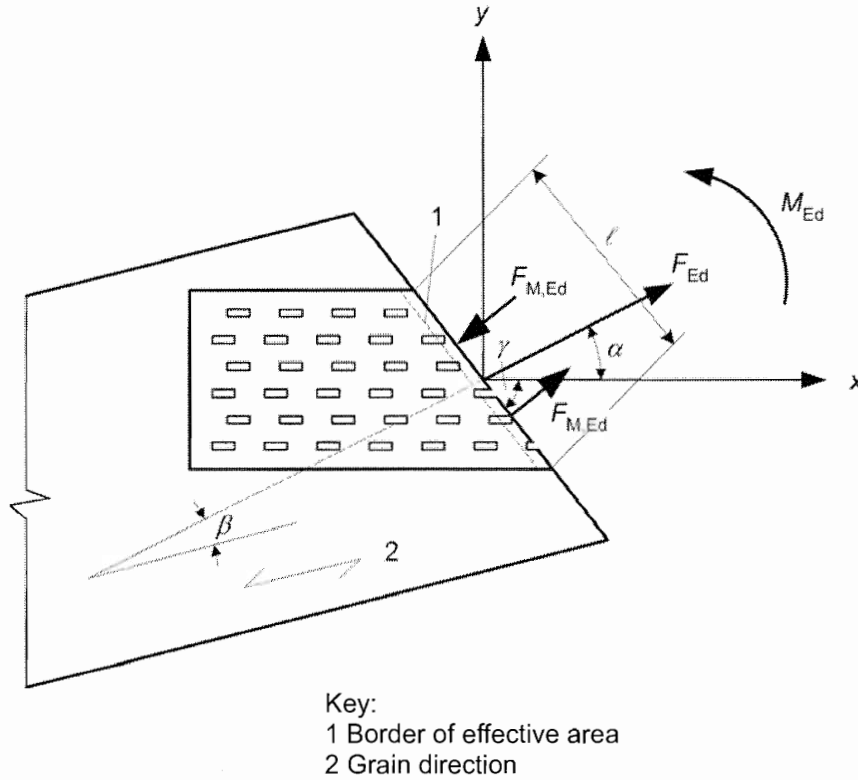


Figure 8.11 – Geometry of punched metal plate connection loaded by a force  $F_{Ed}$  and moment  $M_{Ed}$

#### 8.8.4 Plate anchorage strengths

(1) The characteristic anchorage strength per plate  $f_{a,\alpha,\beta,k}$  should either be derived from tests or calculated from:

$$f_{a,\alpha,\beta,k} = \max \begin{cases} f_{a,\alpha,0,k} - (f_{a,\alpha,0,k} - f_{a,90,90,k}) \frac{\beta}{45^\circ} & \text{for } \beta \leq 45^\circ, \text{ or} \\ f_{a,0,0,k} - (f_{a,0,0,k} - f_{a,90,90,k}) \sin(\max(\alpha, \beta)) & \end{cases} \quad (8.42)$$

$$f_{a,\alpha,\beta,k} = f_{a,0,0,k} - (f_{a,0,0,k} - f_{a,90,90,k}) \sin(\max(\alpha, \beta)) \quad \text{for } 45^\circ < \beta \leq 90^\circ \quad (8.43)$$

(2) The characteristic anchorage strength per plate parallel to grain should be taken as:

$$f_{a,\alpha,0,k} = \begin{cases} f_{a,0,0,k} + k_1 \alpha & \text{when } \alpha \leq \alpha_0 \\ f_{a,0,0,k} + k_1 \alpha_0 + k_2 (\alpha - \alpha_0) & \text{when } \alpha_0 < \alpha \leq 90^\circ \end{cases} \quad (8.44)$$

The constants  $k_1$ ,  $k_2$  and  $\alpha_0$  should be determined from anchorage tests in accordance with EN 1075 and derived in accordance with the procedure given in EN 14545 for the actual plate type.

#### 8.8.5 Connection strength verification

##### 8.8.5.1 Plate anchorage capacity

(1) The design anchorage stress  $\tau_{F,d}$  on a single punched metal plate fastener imposed by a

force  $F_{Ed}$  and the design anchorage stress  $\tau_{M,d}$  imposed from a moment  $M_{Ed}$ , should be taken as:

$$\tau_{F,d} = \frac{F_{A,Ed}}{A_{ef}} \quad (8.45)$$

$$\tau_{M,d} = \frac{M_{A,Ed}}{W_p} \quad (8.46)$$

with:

$$W_p = \int_{A_{ef}} r \, dA \quad (8.47)$$

where:

$F_{A,Ed}$  is the design force acting on a single plate at the centroid of the effective area (i.e. half of the total force in the timber member);

$M_{A,Ed}$  is the design moment acting on a single plate on the centroid of the effective area;

$dA$  is the segmental area of the punched metal plate fastener;

$r$  is the distance from the centre of gravity of the plate to the segmental plate area  $dA$ ;

$A_{ef}$  is the effective plate area.

(2) As an alternative to expression (8.47),  $W_p$  may be conservatively approximated from:

$$W_p = \frac{A_{ef} \, d}{4} \quad (8.48)$$

with:

$$d = \sqrt{\left(\frac{A_{ef}}{h_{ef}}\right)^2 + h_{ef}^2} \quad (8.49)$$

where:

$h_{ef}$  is the maximum height of the effective anchorage area perpendicular to the longest side.

(3) Contact pressure between timber members may be taken into account to reduce the value of  $F_{Ed}$  in compression provided that the gap between the members has an average value, which is not greater than 1,5 mm, and a maximum value of 3 mm. In such cases the connection should be designed for a minimum compressive design force of  $F_{A,Ed}/2$ .

(4) Contact pressure between the timber members in chord splices in compression may be taken into account by designing the single plate for a design force,  $F_{A,Ed}$ , and a design moment  $M_{A,Ed}$ , according to the following expressions:

$$F_{A,Ed} = \sqrt{\left(\frac{F_{Ed} \cos \beta}{2} - \frac{3|M_{Ed}|}{2h}\right)^2 + (F_{Ed} \sin \beta)^2} \quad (8.50)$$

$$M_{A,Ed} = \frac{M_{Ed}}{2} \quad (8.51)$$

where:

$F_{Ed}$  is the design axial force of the chord acting on a single plate (compression or zero);

$M_{Ed}$  is the design moment of the chord acting on a single plate;

$h$  is the height of the chord.

(5) The following expression should be satisfied:

$$\left( \frac{\tau_{F,d}}{f_{a,\alpha,\beta,d}} \right)^2 + \left( \frac{\tau_{M,d}}{f_{a,0,0,d}} \right)^2 \leq 1 \quad (8.52)$$

### 8.8.5.2 Plate capacity

(1) For each joint interface, the forces in the two main directions should be taken as:

$$F_{x,Ed} = F_{Ed} \cos \alpha \pm 2F_{M,Ed} \sin \gamma \quad (8.53)$$

$$F_{y,Ed} = F_{Ed} \sin \alpha \pm 2F_{M,Ed} \cos \gamma \quad (8.54)$$

where:

$F_{Ed}$  is the design force in a single plate (i.e. half of the total force in the timber member)

$F_{M,Ed}$  is the design force from the moment on a single plate ( $F_{M,Ed} = 2 M_{Ed} / \ell$ )

(2) The following expression should be satisfied:

$$\left( \frac{F_{x,Ed}}{F_{x,Rd}} \right)^2 + \left( \frac{F_{y,Ed}}{F_{y,Rd}} \right)^2 \leq 1 \quad (8.55)$$

where:

$F_{x,Ed}$  and  $F_{y,Ed}$  are the design forces acting in the x and y direction,

$F_{x,Rd}$  and  $F_{y,Rd}$  are the corresponding design values of the plate capacity. They are determined from the maximum of the characteristic capacities at sections parallel or perpendicular to the main axes, based upon the following expressions for the characteristic plate capacities in these directions

$$F_{x,Rk} = \max \left\{ \begin{array}{l} |f_{n,0,k} \ell \sin(\gamma - \gamma_0 \sin(2\gamma))| \\ |f_{v,0,k} \ell \cos \gamma| \end{array} \right. \quad (8.56)$$

$$F_{y,Rk} = \max \left\{ \begin{array}{l} |f_{n,90,k} \ell \cos \gamma| \\ |k f_{v,90,k} \ell \sin \gamma| \end{array} \right. \quad (8.57)$$

with

$$f_{n,0,k} = \begin{cases} f_{t,0,k} & \text{for } F_{x,Ed} > 0 \\ f_{c,0,k} & \text{for } F_{x,Ed} \leq 0 \end{cases} \quad (8.58)$$

$$f_{n,90,k} = \begin{cases} f_{t,90,k} & \text{for } F_{y,Ed} > 0 \\ f_{c,90,k} & \text{for } F_{y,Ed} \leq 0 \end{cases} \quad (8.59)$$

$$k = \begin{cases} 1 + k_v \sin(2\gamma) & \text{for } F_{x,Ed} > 0 \\ 1 & \text{for } F_{x,Ed} \leq 0 \end{cases} \quad (8.60)$$

where  $\gamma_0$  and  $k_v$  are constants determined from shear tests in accordance with EN 1075 and derived in accordance with the procedure given in EN 14545 for the actual plate type.

(3) If the plate covers more than two connection lines on the member then the forces in each straight part of the connection line should be determined such that equilibrium is fulfilled and

that expression (8.55) is satisfied in each straight part of the connection line. All critical sections should be taken into account.

### 8.9 Split ring and shear plate connectors

(1) For connections made with ring connectors of type A or shear plate connectors of type B according to EN 912 and EN 14545, and with a diameter not bigger than 200 mm, the characteristic load-carrying capacity parallel to grain,  $F_{v,0,Rk}$  per connector and per shear plane should be taken as:

$$F_{v,0,Rk} = \min \begin{cases} k_1 k_2 k_3 k_4 (35 d_c^{1.5}) & (a) \\ k_1 k_3 h_e (31,5 d_c) & (b) \end{cases} \quad (8.61)$$

where:

$F_{v,0,Rk}$  is the characteristic load-carrying capacity parallel to the grain, in N;

$d_c$  is the connector diameter, in mm;

$h_e$  is the embedment depth, in mm;

$k_i$  are modification factors, with  $i = 1$  to 4, defined below.

(2) The minimum thickness of the outer timber members should be  $2,25h_e$ , and of the inner timber member should be  $3,75h_e$ , where  $h_e$  is the embedment depth, see Figure 8.12.

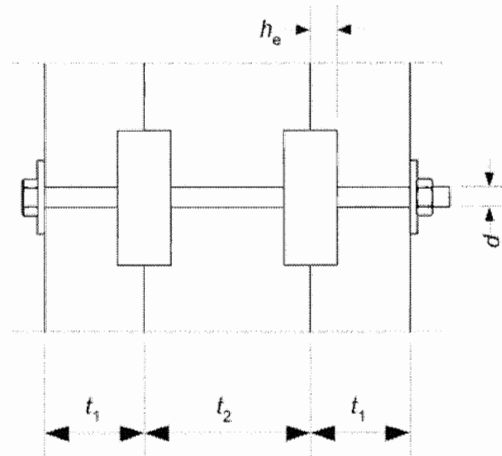


Figure 8.12 – Dimensions for connections with split ring and shear plate connectors

(3) The factor  $k_1$  should be taken as:

$$k_1 = \min \begin{cases} 1 \\ \frac{t_1}{3h_e} \\ \frac{t_2}{5h_e} \end{cases} \quad (8.62)$$

(4) The factor  $k_2$  applies to a loaded end ( $-30^\circ \leq \alpha \leq 30^\circ$ ) and should be taken as:

$$k_2 = \min \left\{ \begin{array}{l} k_a \\ \frac{a_{3,t}}{2d_c} \end{array} \right. \quad (8.63)$$

where:

$$k_a = \begin{cases} 1,25 & \text{for connections with one connector per shear plane} \\ 1,0 & \text{for connections with more than one connector per shear plane} \end{cases} \quad (8.64)$$

$a_{3,t}$  is given in Table 8.7.

For other values of  $\alpha$ ,  $k_2 = 1,0$ .

(5) The factor  $k_3$  should be taken as:

$$k_3 = \min \left\{ \begin{array}{l} 1,75 \\ \frac{\rho_k}{350} \end{array} \right. \quad (8.65)$$

where  $\rho_k$  is the characteristic density of the timber, in  $\text{kg/m}^3$ .

(6) The factor  $k_4$ , which depends on the materials connected, should be taken as:

$$k_4 = \begin{cases} 1,0 & \text{for timber-to-timber connections} \\ 1,1 & \text{for steel-to-timber connections} \end{cases} \quad (8.66)$$

(7) For connections with one connector per shear plane loaded in an unloaded end situation ( $150^\circ \leq \alpha \leq 210^\circ$ ), the condition (a) in expression (8.61) should be disregarded.

(8) For a force at an angle  $\alpha$  to the grain, the characteristic load-carrying capacity,  $F_{\alpha,Rk}$  per connector per shear plane should be calculated using the following expression:

$$F_{v,\alpha,Rk} = \frac{F_{v,0,Rk}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \quad (8.67)$$

with:

$$k_{90} = 1,3 + 0,001 d_c \quad (8.68)$$

where:

$F_{v,0,Rk}$  is the characteristic load-carrying capacity of the connector for a force parallel to grain according to expression (8.61);

$d_c$  is the connector diameter, in mm.

(9) Minimum spacing and edge and end distances are given in Table 8.7, with the symbols illustrated in Figure 8.7.

**Table 8.7 – Minimum spacings and edge and end distances for ring and shear plate connectors.**

Spacing and edge/end distances (see Figure 8.7)	Angle to grain	Minimum spacings and edge/end distances
$a_1$ (parallel to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$(1,2 + 0,8  \cos \alpha ) d_c$
$a_2$ (perpendicular to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$1,2 d_c$
$a_{3,t}$ (loaded end)	$-90^\circ \leq \alpha \leq 90^\circ$	$1,5 d_c$
$a_{3,c}$ (unloaded end)	$90^\circ \leq \alpha < 150^\circ$	$(0,4 + 1,6  \sin \alpha ) d_c$
	$150^\circ \leq \alpha < 210^\circ$	$1,2 d_c$
	$210^\circ \leq \alpha \leq 270^\circ$	$(0,4 + 1,6  \sin \alpha ) d_c$
$a_{4,t}$ (loaded edge)	$0^\circ \leq \alpha \leq 180^\circ$	$(0,6 + 0,2  \sin \alpha ) d_c$
$a_{4,c}$ (unloaded edge)	$180^\circ \leq \alpha \leq 360^\circ$	$0,6 d_c$

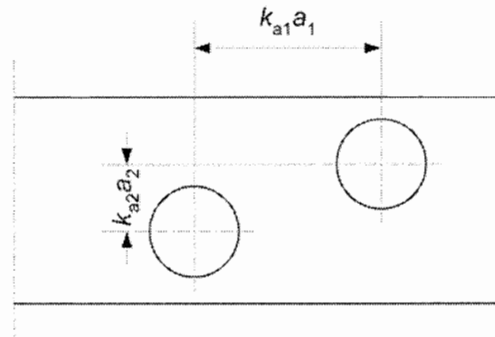
(10) When the connectors are staggered (see Figure 8.13), the minimum spacings parallel and perpendicular to the grain should comply with the following expression:

$$(k_{a1})^2 + (k_{a2})^2 \geq 1 \quad \text{with} \quad \begin{cases} 0 \leq k_{a1} \leq 1 \\ 0 \leq k_{a2} \leq 1 \end{cases} \quad (8.69)$$

where:

$k_{a1}$  is a reduction factor for the minimum distance  $a_1$  parallel to the grain;

$k_{a2}$  is a reduction factor for the minimum distance  $a_2$  perpendicular to the grain.



**Figure 8.13 – Reduced distances for connectors**

(11) The spacing parallel to grain,  $k_{a1} a_1$  may further be reduced by multiplication by a factor  $k_{s,red}$ , with  $0,5 \leq k_{s,red} \leq 1,0$ , provided that the load-carrying capacity is multiplied by a factor

$$k_{R,red} = 0,2 + 0,8 k_{s,red} \quad (8.70)$$

(12) For a row of connectors parallel to the grain, the load-carrying capacity in that direction should be calculated using the effective number of connectors  $n_{ef}$  where:

$$n_{ef} = 2 + \left(1 - \frac{n}{20}\right)(n - 2) \quad (8.71)$$

where:

$n_{ef}$  is the effective number of connectors;

$n$  is the number of connectors in a line parallel to grain.

(13) Connectors should be considered as positioned parallel to the grain where  $k_{a2} a_2 < 0,5 k_{a1} a_1$ .

### 8.10 Toothed-plate connectors

(1) The characteristic load-carrying capacity of connections made using toothed-plate connectors should be taken as the summation of the characteristic load-carrying capacity of the connectors themselves and the connecting bolts according to 8.5.

(2) The characteristic load-carrying capacity  $F_{v,Rk}$  per toothed-plate connector for connectors of type C according to EN 912 (single-sided: type C2, C4, C7, C9, C11; double sided: type C1, C3, C5, C6, C8, C10) and EN 14545 should be taken as:

$$F_{v,Rk} = \begin{cases} 18 k_1 k_2 k_3 d_c^{1,5} & \text{for types C1 to C9} \\ 25 k_1 k_2 k_3 d_c^{1,5} & \text{for types C10 and C11} \end{cases} \quad (8.72)$$

where:

$F_{v,Rk}$  is the characteristic load-carrying capacity per toothed-plate connector, in N.

$k_i$  are modification factors, with  $i = 1$  to 3, defined below.

$d_c$  is:

- the toothed-plate connector diameter for types C1, C2, C6, C7, C10 and C11, in mm;
- the toothed-plate connector side length for types C5, C8 and C9, in mm;
- the square root of the product of both side lengths for types C3 and C4, in mm.

(3) Clause 8.9(2) applies.

(4) The factor  $k_1$  should be taken as:

$$k_1 = \min \begin{cases} 1 \\ \frac{t_1}{3h_e} \\ \frac{t_2}{5h_e} \end{cases} \quad (8.73)$$

where:

$t_1$  is the side member thickness;

$t_2$  is the middle member thickness;

$h_e$  is the tooth penetration depth.  $\square$

(5) The factor  $k_2$  should be taken as:

– For types C1 to C9:

$$k_2 = \min \begin{cases} 1 \\ \frac{a_{3,i}}{1,5 d_c} \end{cases} \quad (8.74)$$

with

$$a_{3,t} = \max \begin{cases} 1,1 d_c \\ 7 d \\ 80 \text{ mm} \end{cases} \quad (8.75)$$

where:

$d$  is the bolt diameter, in mm;

$d_c$  is explained in (2) above.

- For types C10 and C11:

$$k_2 = \min \begin{cases} 1 \\ \frac{a_{3,t}}{2,0 d_c} \end{cases} \quad (8.76)$$

with

$$a_{3,t} = \max \begin{cases} 1,5 d_c \\ 7 d \\ 80 \text{ mm} \end{cases} \quad (8.77)$$

where:

$d$  is the bolt diameter in mm;

$d_c$  is explained in (2) above.

- (6) The factor  $k_3$  should be taken as:

$$k_3 = \min \begin{cases} 1,5 \\ \frac{\rho_k}{350} \end{cases} \quad (8.78)$$

where  $\rho_k$  is the characteristic density of the timber, in  $\text{kg/m}^3$ .

- (7) For toothed-plate connector types C1 to C9, minimum spacings and edge and end distances should be taken from Table 8.8, with the symbols illustrated in Figure 8.7.

- (8) For toothed-plate connector types C10 and C11, minimum spacing and edge and end distances should be taken from Table 8.9, with the symbols illustrated in Figure 8.7.

- (9) Where connectors of types C1, C2, C6 and C7 with circular shape are staggered, 8.9(10) applies.

- (10) For bolts used with toothed-plate connectors, 10.4.3 applies.

**Table 8.8 – Minimum spacings and edge and end distances for toothed-plate connector types C1 to C9.**

Spacings and edge/end distances (see Figure 8.7)	Angle to grain	Minimum spacings and edge/end distances
$a_1$ (parallel to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$(1,2 + 0,3  \cos \alpha ) d_c$
$a_2$ (perpendicular to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$1,2 d_c$
$a_{3,t}$ (loaded end)	$-90^\circ \leq \alpha \leq 90^\circ$	$2,0 d_c$
$a_{3,c}$ (unloaded end)	$90^\circ \leq \alpha < 150^\circ$	$(0,9 + 0,6  \sin \alpha ) d_c$
	$150^\circ \leq \alpha < 210^\circ$	$1,2 d_c$
	$210^\circ \leq \alpha \leq 270^\circ$	$(0,9 + 0,6  \sin \alpha ) d_c$
$a_{4,t}$ (loaded edge)	$0^\circ \leq \alpha \leq 180^\circ$	$(0,6 + 0,2  \sin \alpha ) d_c$
$a_{4,c}$ (unloaded edge)	$180^\circ \leq \alpha \leq 360^\circ$	$0,6 d_c$

**Table 8.9 – Minimum spacings and edge and end distances for toothed-plate connector types C10 and C11.**

Spacings and edge/end distances (see Figure 8.7)	Angle to grain	Minimum spacings and edge/end distances
$a_1$ (parallel to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$(1,2 + 0,8  \cos \alpha ) d_c$
$a_2$ (perpendicular to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$1,2 d_c$
$a_{3,t}$ (loaded end)	$-90^\circ \leq \alpha \leq 90^\circ$	$2,0 d_c$
$a_{3,c}$ (unloaded end)	$90^\circ \leq \alpha < 150^\circ$	$(0,4 + 1,6  \sin \alpha ) d_c$
	$150^\circ \leq \alpha < 210^\circ$	$1,2 d_c$
	$210^\circ \leq \alpha \leq 270^\circ$	$(0,4 + 1,6  \sin \alpha ) d_c$
$a_{4,t}$ (loaded edge)	$0^\circ \leq \alpha \leq 180^\circ$	$(0,6 + 0,2  \sin \alpha ) d_c$
$a_{4,c}$ (unloaded edge)	$180^\circ \leq \alpha \leq 360^\circ$	$0,6 d_c$

## Section 9 Components and assemblies

### 9.1 Components

#### 9.1.1 Glued thin-webbed beams

(1) If a linear variation of strain over the depth of the beam is assumed, the axial stresses in the wood-based flanges should satisfy the following expressions:

$$\sigma_{f,c,max,d} \leq f_{m,d} \quad (9.1)$$

$$\sigma_{f,t,max,d} \leq f_{m,d} \quad (9.2)$$

$$\sigma_{f,c,d} \leq k_c f_{c,0,d} \quad (9.3)$$

$$\sigma_{f,t,d} \leq f_{t,0,d} \quad (9.4)$$

where:

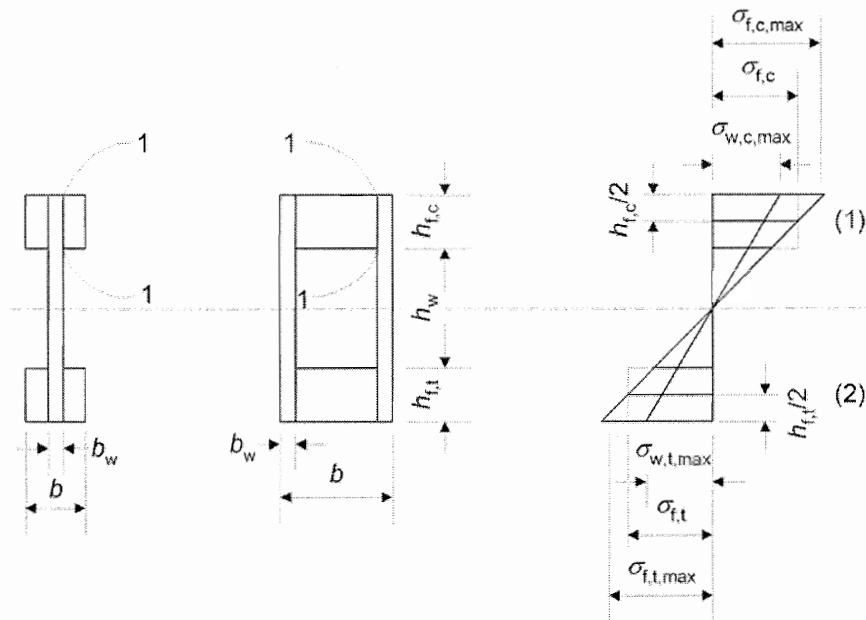
$\sigma_{f,c,max,d}$  is the extreme fibre flange design compressive stress;

$\sigma_{f,t,max,d}$  is the extreme fibre flange design tensile stress;

$\sigma_{f,c,d}$  is the mean flange design compressive stress;

$\sigma_{f,t,d}$  is the mean flange design tensile stress;

$k_c$  is a factor which takes into account lateral instability.



Key:  
(1) compression  
(2) tension

Figure 9.1 – Thin-webbed beams

(3) The factor  $k_c$  may be determined (conservatively, especially for box beams) according to 6.3.2 with

$$\lambda_z = \sqrt{12} \left( \frac{\ell_c}{b} \right) \quad (9.5)$$

where:

$\ell_c$  is the distance between the sections where lateral deflection of the compressive flange is prevented;

$b$  is given in Figure 9.1.

If a special investigation is made with respect to the lateral instability of the beam as a whole, it may be assumed that  $k_c = 1,0$ .

(4) The axial stresses in the webs should satisfy the following expressions:

$$\sigma_{w,c,d} \leq f_{c,w,d} \quad (9.6)$$

$$\sigma_{w,t,d} \leq f_{t,w,d} \quad (9.7)$$

where:

$\sigma_{w,c,d}$  and  $\sigma_{w,t,d}$  are the design compressive and tensile stresses in the webs;

$f_{c,w,d}$  and  $f_{t,w,d}$  are the design compressive and tensile bending strengths of the webs.

(5) Unless other values are given, the design in-plane bending strength of the webs should be taken as the design tensile or compressive strength.

(6)P It shall be verified that any glued splices have sufficient strength.

(7) Unless a detailed buckling analysis is made it should be verified that:

$$h_w \leq 70 b_w \quad (9.8)$$

and

$$F_{v,w,Ed} \leq \begin{cases} b_w h_w \left( 1 + \frac{0,5(h_{f,t} + h_{f,c})}{h_w} \right) f_{v,0,d} & \text{for } h_w \leq 35 b_w \\ 35 b_w^2 \left( 1 + \frac{0,5(h_{f,t} + h_{f,c})}{h_w} \right) f_{v,0,d} & \text{for } 35 b_w \leq h_w \leq 70 b_w \end{cases} \quad (9.9)$$

where:

$F_{v,w,Ed}$  is the design shear force acting on each web;

$h_w$  is the clear distance between flanges;

$h_{f,c}$  is the compressive flange depth;

$h_{f,t}$  is the tensile flange depth;

$b_w$  is the width of each web;

$f_{v,0,d}$  is the design panel shear strength.

(8) For webs of wood-based panels, it should, for sections 1-1 in Figure 9.1, be verified that:

$$\tau_{\text{mean,d}} \leq \begin{cases} f_{v,90,d} & \text{for } h_f \leq 4 b_{\text{ef}} \\ f_{v,90,d} \left( \frac{4b_{\text{ef}}}{h_f} \right)^{0,8} & \text{for } h_f > 4 b_{\text{ef}} \end{cases} \quad (9.10)$$

where:

$\tau_{\text{mean,d}}$  is the design shear stress at the sections 1-1, assuming a uniform stress distribution;

$f_{v,90,d}$  is the design planar (rolling) shear strength of the web;

$h_f$  is either  $h_{f,c}$  or  $h_{f,t}$ .

$$b_{\text{ef}} = \begin{cases} b_w & \text{for boxed beams} \\ b_w / 2 & \text{for I-beams} \end{cases} \quad (9.11)$$

### 9.1.2 Glued thin-flanged beams

(1) This clause assumes a linear variation of strain over the depth of the beam.

(2)P In the strength verification of glued thin-flanged beams, account shall be taken of the non-uniform distribution of stresses in the flanges due to shear lag and buckling.

(3) Unless a more detailed calculation is made, the assembly should be considered as a number of I-beams or U-beams (see Figure 9.2) with effective flange widths  $b_{\text{ef}}$ , as follows:

– For I-beams

$$b_{\text{ef}} = b_{\text{c,ef}} + b_w \quad (\text{or } b_{\text{t,ef}} + b_w) \quad (9.12)$$

– For U-beams

$$b_{\text{ef}} = 0,5b_{\text{c,ef}} + b_w \quad (\text{or } 0,5b_{\text{t,ef}} + b_w) \quad (9.13)$$

The values of  $b_{\text{c,ef}}$  and  $b_{\text{t,ef}}$  should not be greater than the maximum value calculated for shear lag from Table 9.1. In addition the value of  $b_{\text{c,ef}}$  should not be greater than the maximum value calculated for plate buckling from Table 9.1.

(4) Maximum effective flange widths due to the effects of shear lag and plate buckling should be taken from Table 9.1, where  $\ell$  is the span of the beam.

**Table 9.1 – Maximum effective flange widths due to the effects of shear lag and plate buckling**

Flange material	Shear lag	Plate buckling
Plywood, with grain direction in the outer plies:		
– Parallel to the webs	$0,1\ell$	$20h_f$
– Perpendicular to the webs	$0,1\ell$	$25h_f$
Oriented strand board	$0,15\ell$	$25h_f$
Particleboard or fibreboard with random fibre orientation	$0,2\ell$	$30h_f$

(5) Unless a detailed buckling investigation is made, the unrestrained flange width should not be

greater than twice the effective flange width due to plate buckling, from Table 9.1.

(6) For webs of wood-based panels, it should, for sections 1-1 of an I-shaped cross-section in Figure 9.2, be verified that:

$$\tau_{\text{mean,d}} \leq \begin{cases} f_{v,90,d} & \text{for } b_w \leq 8h_f \\ f_{v,90,d} \left( \frac{8h_f}{b_w} \right)^{0,8} & \text{for } b_w > 8h_f \end{cases} \quad (9.14)$$

where:

$\tau_{\text{mean,d}}$  is the design shear stress at the sections 1-1, assuming a uniform stress distribution;

$f_{v,90,d}$  is the design planar (rolling) shear strength of the flange.

For section 1-1 of a U-shaped cross-section, the same expressions should be verified, but with  $8h_f$  substituted by  $4h_f$ .

(7) The axial stresses in the flanges, based on the relevant effective flange width, should satisfy the following expressions:

$$\sigma_{f,c,d} \leq f_{f,c,d} \quad (9.15)$$

$$\sigma_{f,t,d} \leq f_{f,t,d} \quad (9.16)$$

where:

$\sigma_{f,c,d}$  is the mean flange design compressive stress;

$\sigma_{f,t,d}$  is the mean flange design tensile stress;

$f_{f,c,d}$  is the flange design compressive strength;

$f_{f,t,d}$  is the flange design tensile strength.

(8)P It shall be verified that any glued splices have sufficient strength.

(9) The axial stresses in the wood-based webs should satisfy the expressions (9.6) to (9.7) defined in 9.1.1

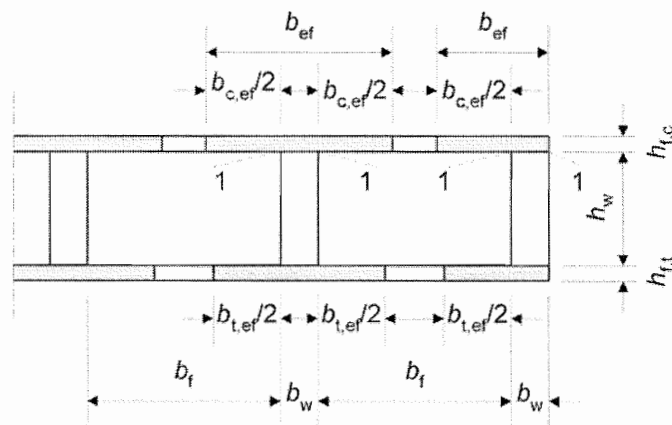


Figure 9.2 – Thin-flanged beam

### 9.1.3 Mechanically jointed beams

(1)P If the cross-section of a structural member is composed of several parts connected by mechanical fasteners, consideration shall be given to the influence of the slip occurring in the

joints.

(2) Calculations should be carried out assuming a linear relationship between force and slip.

(3) If the spacing of the fasteners varies in the longitudinal direction according to the shear force between  $s_{\min}$  and  $s_{\max}$  ( $\leq 4s_{\min}$ ), an effective spacing  $s_{\text{ef}}$  may be used as follows:

$$s_{\text{ef}} = 0,75 s_{\min} + 0,25 s_{\max} \quad (9.17)$$

NOTE: A method for the calculation of the load-carrying capacity of mechanically jointed beams is given in Annex B (Informative).

#### 9.1.4 Mechanically jointed and glued columns

(1)P Deformations due to slip in joints, to shear and bending in packs, gussets, shafts and flanges, and to axial forces in the lattice shall be taken into account in the strength verification.

NOTE: A method for the calculation of the load-carrying capacity of I- and box-columns, spaced columns and lattice columns is given in Annex C (Informative).

### 9.2 Assemblies

#### 9.2.1 Trusses

(1) For trusses which are loaded predominantly at the nodes, the sum of the combined bending and axial compressive stress ratios given in expressions (6.19) and (6.20) should be limited to 0,9.

(2) For members in compression, the effective column length for in-plane strength verification should generally be taken as the distance between two adjacent points of contraflexure.

(3) For fully triangulated trusses, the effective column length for members in compression should be taken as the bay length, see Figure 5.1, if:

- members are only one bay long, without rigid end connections,
- members are continuous over two or more bays and are not loaded laterally

(4) When a simplified analysis of a fully triangulated truss with punched metal plate fasteners according to clause 5.4.3 has been carried out, the following effective column lengths may be assumed (see Figure 9.3)

- for continuous members without significant end moments and where the bending stresses of the lateral load are at least 40 % of the compressive stresses:
  - in an outer bay: 0,8 times the bay length;
  - in an inner bay: 0,6 the bay length;
  - at a node: 0,6 times the largest adjacent bay length;
- for continuous members with significant end moments where the bending stresses of the lateral load are at least 40 % of the compressive stresses:
  - at the beam end with moment: 0,0 (i.e. no column effect);
  - in the penultimate bay: 1,0 times bay length;
  - remaining bays and nodes: as described above for continuous beams without significant end moments;
- for all other cases 1,0 times bay length.

For the strength verification of members in compression and connections, the calculated axial forces should be increased by 10 %.

(5) When a simplified analysis is carried out for trusses which are loaded at the nodes, the tensile and compressive stress ratios as well as the connection capacity should be limited to 70 %.

(6)P A check shall be made that the lateral (out-of-plane) stability of the truss members is adequate.

(7)P The joints shall be capable of transferring the forces which may occur during handling and erection.

(8) All joints should be capable of transferring a force  $F_{r,d}$  acting in any direction within the plane of the truss.  $F_{r,d}$  should be assumed to be of short-term duration, acting on timber in service class 2, with the value:

$$F_{r,d} = 1,0 + 0,1L \quad (9.18)$$

where:

$F_{r,d}$  is in kN;

$L$  is the overall length of the truss, in m.

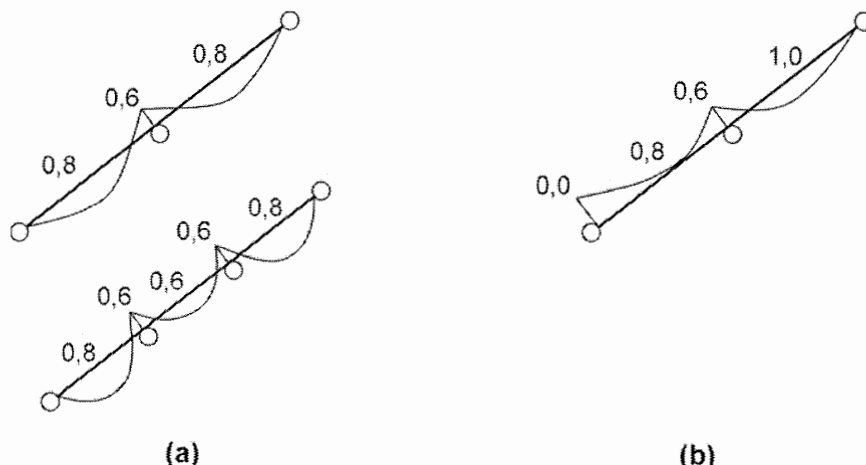


Figure 9.3 – Moment diagrams and effective lengths in compression (a) No significant end moments (b) Significant end moments

### 9.2.2 Trusses with punched metal plate fasteners

(1)P Trusses made with punched metal plate fasteners shall conform to the requirements of EN 14250.

(2) The requirements of 5.4.1 and 9.2.1 apply.

(3) For fully triangulated trusses where a small concentrated force (e.g. a man load) has a component perpendicular to the member of  $< 1,5\text{ kN}$ , and where  $\sigma_{c,d} < 0,4 f_{c,d}$ , and  $\sigma_{t,d} < 0,4 f_{t,d}$ , then the requirements of 6.2.3 and 6.2.4 may be replaced by:

$$\sigma_{m,d} \leq 0,75 f_{m,d} \quad (9.19)$$

(4) The minimum overlap of the punched metal plate on any timber member should be at least equal to 40 mm or one third of the height of the timber member, whichever is the greater.

(5) Punched metal plate fasteners used in chord splices should cover at least  $\frac{2}{3}$  of the required member height.

### 9.2.3 Roof and floor diaphragms

#### 9.2.3.1 General

(1) This section relates to simply supported diaphragms, such as floors or roofs, assembled from sheets of wood-based material fixed by mechanical fasteners to a timber frame.

(2) The load-carrying capacity of fasteners at sheet edges may be increased by a factor of 1,2 over the values given in Section 8.

#### 9.2.3.2 Simplified analysis of roof and floor diaphragms.

(1) For diaphragms with a uniformly distributed load (see Figure 9.4) the simplified method of analysis described in this section should be used provided that:

- the span  $\ell$  lies between  $2b$  and  $6b$ , where  $b$  is the diaphragm width;
- the critical ultimate design condition is failure in the fasteners (and not in the panels);
- the panels are fixed in accordance with the detailing rules in 10.8.1.

(2) Unless a more detailed analysis is made, the edge beams should be designed to resist the maximum bending moment in the diaphragm.

(3) The shear forces in the diaphragm should be assumed to be uniformly distributed over the width of the diaphragm.

(4) When the sheets are staggered, (see Figure 9.4), the nail spacings along the discontinuous panel edges may be increased by a factor of 1,5 (up to a maximum of 150 mm) without reduction of the load-carrying capacity.

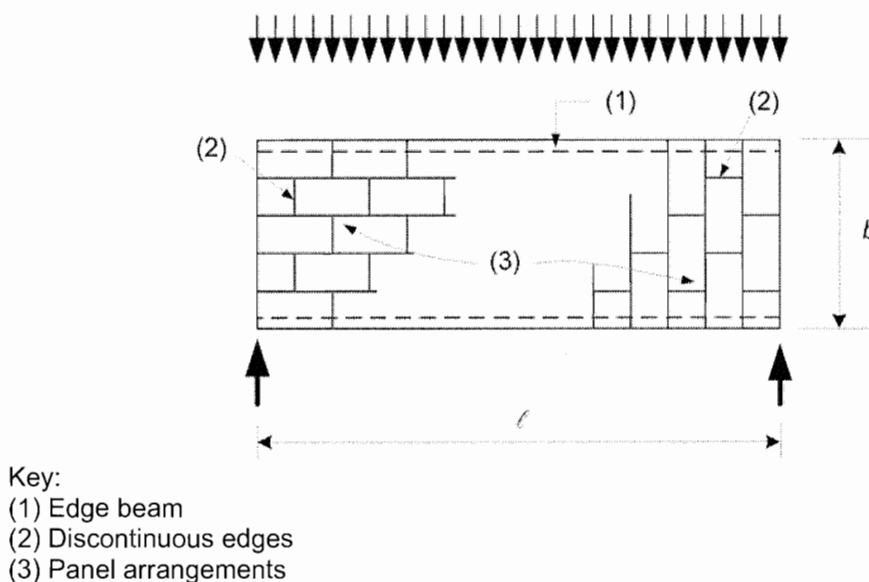


Figure 9.4 – Diaphragm loading and staggered panel arrangements

## 9.2.4 Wall diaphragms

### 9.2.4.1 General

- (1)P Wall diaphragms shall be designed to resist both horizontal and vertical actions imposed upon them.
- (2)P The wall shall be adequately restrained to avoid overturning and sliding.
- (3)P Wall diaphragms deemed to provide resistance to racking shall be stiffened in-plane by board materials, diagonal bracing or moment connections.
- (4)P The racking resistance of a wall shall be determined either by test according to EN 594 or by calculations, employing appropriate analytical methods or design models.
- (5)P The design of wall diaphragms shall take account of both the material construction and geometric make-up of the wall under consideration.
- (6)P The response of wall diaphragms to actions shall be assessed to ensure the construction remains within appropriate serviceability limits.
- (7) For wall diaphragms two alternative simplified methods of calculation are given in 9.2.4.2 and 9.2.4.3.

NOTE: The recommended procedure is method A given in 9.2.4.2. National choice may be given in the National annex.

### 9.2.4.2 Simplified analysis of wall diaphragms – Method A

- (1) The simplified method given in this subclause should only be applied to wall diaphragms with a tie-down at their end, that is the vertical member at the end is directly connected to the construction below.
- (2) The design load-carrying capacity  $F_{v,Rd}$  (the design racking resistance) under a force  $F_{v,Ed}$  acting at the top of a cantilevered panel secured against uplift (by vertical actions or by anchoring) should be determined using the following simplified method of analysis for walls made up of one or more panels, where each wall panel consists of a sheet fixed to one side of a timber frame, provided that:
- the spacing of fasteners is constant along the perimeter of every sheet;
  - the width of each sheet is at least  $h/4$ .
- (3) For a wall made up of several wall panels, the design racking load-carrying capacity of a wall should be calculated from

$$F_{v,Rd} = \sum F_{i,v,Rd} \quad (9.20)$$

where  $F_{i,v,Rd}$  is the design racking load-carrying capacity of the wall panel in accordance with 9.2.4.2(4) and 9.2.4.2(5).  $\overline{A_1}$

- (4) The design racking load-carrying capacity of each wall panel,  $F_{i,v,Rd}$ , against a force  $F_{i,v,Ed}$  according to Figure 9.5 should be calculated from

$$F_{i,v,Rd} = \frac{F_{f,Rd} b_i c_i}{s} \quad (9.21)$$

where:

$F_{f,Rd}$  is the lateral design capacity of an individual fastener;

$b_i$  is the wall panel width;

$s$  is the fastener spacing.

and

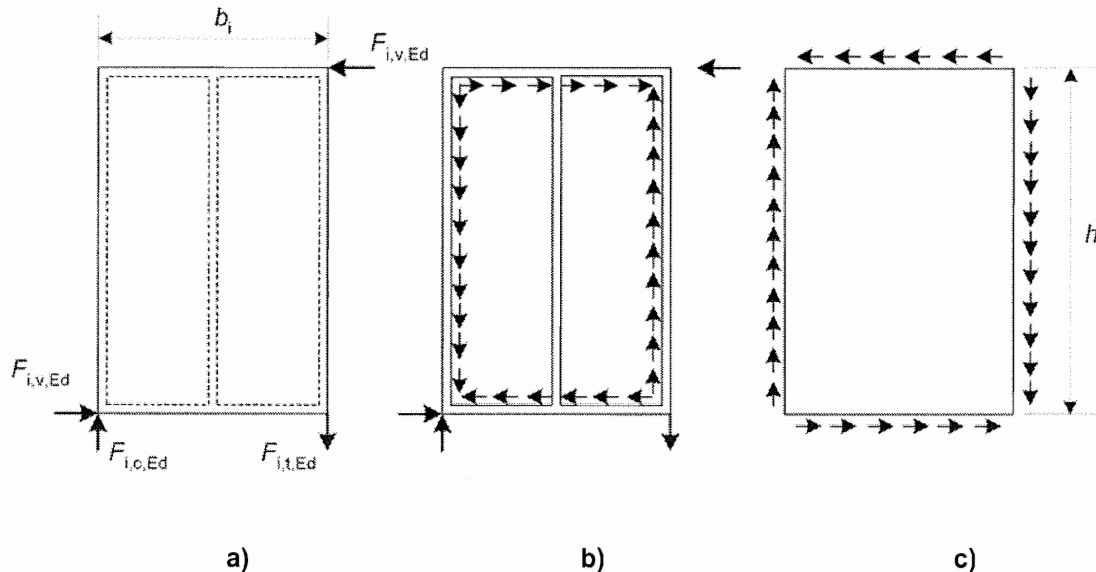
$$c_i = \begin{cases} 1 & \text{for } b_i \geq b_0 \\ \frac{b_i}{b_0} & \text{for } b_i < b_0 \end{cases} \quad (9.22)$$

where:

$$b_0 = h/2$$

$h$  is the height of the wall.

(5) For fasteners along the edges of an individual sheet, the design lateral load-carrying capacity should be increased by a factor of 1,2 over the corresponding values given in Section 8. In determining the fastener spacing in accordance with the requirements of Section 8, the edges should be assumed to be unloaded.



**Figure 9.5 – Forces acting on:**  
a) wall panel;  
b) framing;  
c) sheet

(6) Wall panels which contain a door or window opening should not be considered to contribute to the racking load-carrying capacity.

(7) For wall panels with sheets on both sides the following rules apply:

- if the sheets and fasteners are of the same type and dimension then the total racking load-carrying capacity of the wall should be taken as the sum of the racking load-carrying capacities of the individual sides
- if different types of sheets are used, 75 % of the racking load-carrying capacity of the weaker side may, unless some other value is shown to be valid, be taken into consideration if fasteners with similar slip moduli are used. In other cases not more than 50 % should be taken into consideration.

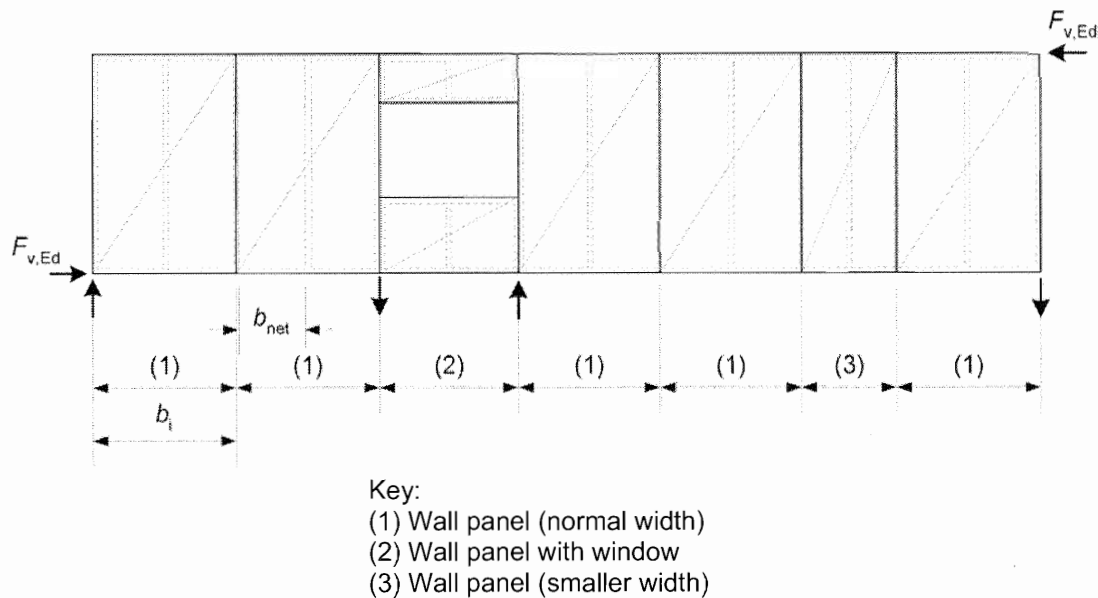
(8) The external forces  $F_{i,c,Ed}$  and  $F_{i,t,Ed}$  according to Figure 9.5 should be determined from

$$F_{i,c,Ed} = F_{i,t,Ed} = \frac{F_{i,v,Ed} h}{b_i} \quad (9.23)$$

where  $h$  is the height of the wall.

(9) These forces can either be transmitted to the sheets in the adjacent wall panel or transmitted to the construction situated above or below. When tensile forces are transmitted to the construction situated below, the panel should be anchored by stiff fasteners. Buckling of wall studs should be checked in accordance with 6.3.2. Where the ends of vertical members bear on horizontal framing members, the compression perpendicular to the grain stresses in the horizontal members should be assessed according to 6.1.5.

(10) The external forces which arise in wall panels containing door or window openings and in wall panels of smaller width, see Figure 9.6, can similarly be transmitted to the construction situated above or below.



**Figure 9.6 – Example of the assembly of wall panels containing a wall panel with a window opening and a wall panel of smaller width**

(11) Shear buckling of the sheet may be disregarded, provided that  $\frac{b_{net}}{t} \leq 100$

where:

$b_{net}$  is the clear distance between studs;

$t$  is the thickness of the sheet.

(12) In order that the centre stud may be considered to constitute a support for a sheet, the spacing of fasteners in the centre stud should not be greater than twice the spacing of the fasteners along the edges of the sheet.

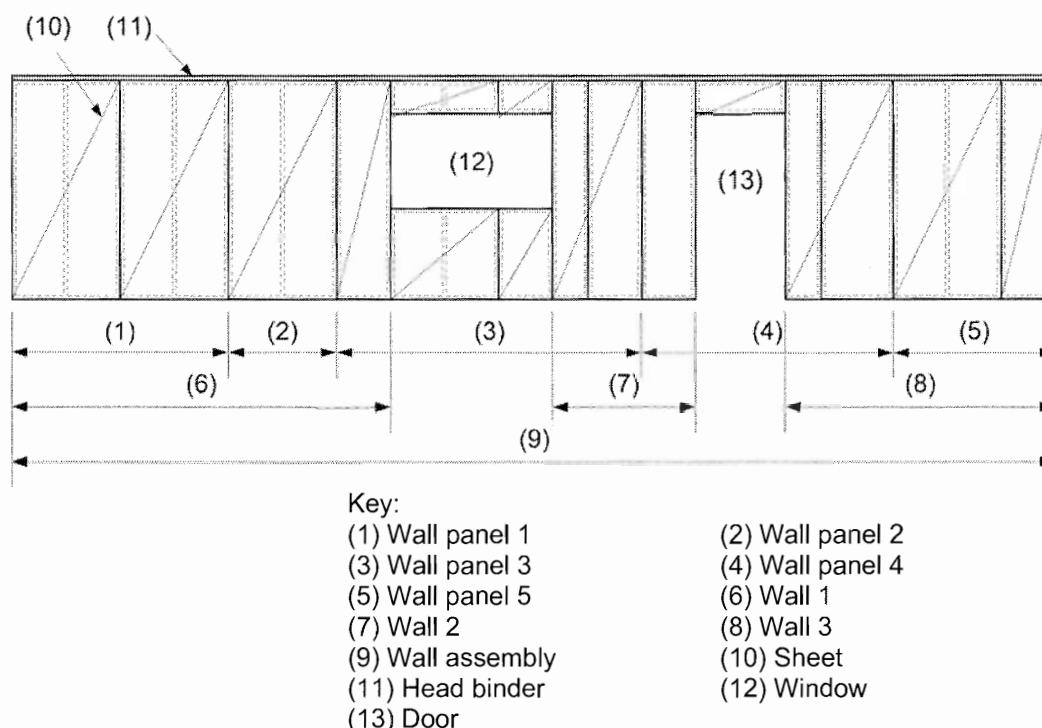
(13) Where each panel consists of a prefabricated wall element, the transfer of shear forces between the separate wall elements should be verified.

(14) In contact areas between vertical studs and horizontal timber members, compression stresses perpendicular to grain should be verified in the timber members.

### 9.2.4.3 Simplified analysis of wall diaphragms – Method B

#### 9.2.4.3.1 Construction of walls and panels to meet the requirements of the simplified analysis

(1) A wall assembly (see Figure 9.7) is comprised of one or more walls with each wall formed from one or more panels, the panels being made from sheets of wood-based panel products, such as those described in 3.5, fastened to a timber frame.



**Figure 9.7 – Example of wall assembly consisting of several wall panels**

(2) For a panel to contribute to the in-plane (racking) strength of a wall the width of the panel should be at least the panel height divided by 4. The fastening of the sheets to the timber frame should be by either nails or screws and the fasteners should be equally spaced around the perimeter of the sheet. Fasteners within the perimeter of a sheet should be spaced at not more than twice the perimeter fastener spacing.

(3) Where an opening is formed in a panel, the lengths of panel on each side of the opening should be considered as separate panels.

(4) Where panels are combined to form a wall:

- the tops of individual panels should be linked by a member or construction across the panel joints;
- the required vertical connection strength between two panels should be evaluated but should have a design strength of at least 2,5 kN/m;
- the panels when joined together to form a wall should be able to resist overturning and sliding forces by either anchorage to the supporting structure or the permanent actions applied to the wall or a combination of both effects.

#### 9.2.4.3.2 Design procedure

(1) The in-plane design shear (racking) strength  $F_{v,Rd}$  against a force  $F_{v,Ed}$  acting at the top of a cantilevered wall that is secured against uplift and sliding by vertical actions and/or anchorage, should be determined using the following simplified method for the wall construction defined in 9.2.4.3.1.

(2) For a wall assembly made up of several walls, the design racking strength of the wall assembly  $F_{v,Rd}$  should be calculated from

$$F_{v,Rd} = \sum F_{i,v,Rd} \quad (9.24)$$

where:

$F_{i,v,Rd}$  is the design racking strength of a wall in accordance with (3) below.

(3) The design racking strength of a wall  $i$ ,  $F_{i,v,Rd}$ , should be calculated from

$$F_{i,v,Rd} = \frac{F_{f,Rd} b_i}{s_0} k_d k_{i,q} k_s k_n \quad (9.25)$$

where:

$F_{f,Rd}$  is the lateral design capacity of an individual fastener;

$b_i$  is the wall length, in m;

$s_0$  is the basic fastener spacing, in m, see (4) below;

$k_d$  is the dimension factor for the wall, see (4) below;

$k_{i,q}$  is the uniformly distributed load factor for wall  $i$ , see (4) below;

$k_s$  is the fastener spacing factor, see (4) below;

$k_n$  is the sheathing material factor, see (4) below.

(4) The values of  $s_0$ ,  $k_d$ ,  $k_{i,q}$ ,  $k_s$  and  $k_n$  should be calculated as:

$$s_0 = \frac{9,7 d}{\rho_k} \quad (9.26)$$

where:

$s_0$  is the basic fastener spacing, in m;

$d$  is the fastener diameter, in mm;

$\rho_k$  is the characteristic density of the timber frame, in  $\text{kg/m}^3$ ;

$$k_d = \begin{cases} \frac{b_i}{h} & \text{for } \frac{b_i}{h} \leq 1,0 \\ \left(\frac{b_i}{h}\right)^{0,4} & \text{for } \frac{b_i}{h} > 1,0 \text{ and } b_i \leq 4,8 \text{ m} \\ \left(\frac{4,8}{h}\right)^{0,4} & \text{for } \frac{b_i}{h} > 1,0 \text{ and } b_i > 4,8 \text{ m} \end{cases} \quad (9.27)$$

where  $h$  is the height of the wall, in m;

$$k_{i,q} = 1 + \left(0,083 q_i - 0,0008 q_i^2\right) \left(\frac{2,4}{b_i}\right)^{0,4} \quad (9.28)$$

where  $q_i$  is the equivalent uniformly distributed vertical load acting on the wall, in  $\text{kN/m}$ , with  $q_i \geq 0$ , see (5) below;

$$- k_s = \frac{1}{0,86 \frac{s}{s_0} + 0,57} \quad (9.29)$$

where  $s$  is the spacing of the fasteners around the perimeter of the sheets;

$$- k_n = \begin{cases} 1,0 & \text{for sheathing on one side} \\ \frac{F_{i,v,Rd,max} + 0,5 F_{i,v,Rd,min}}{F_{i,v,Rd,max}} & \text{for sheathing on both sides} \end{cases} \quad (9.30)$$

where:

$F_{i,v,Rd,max}$  is the design racking strength of the stronger sheathing;

$F_{i,v,Rd,min}$  is the design racking strength of the weaker sheathing.

(5) The equivalent vertical load,  $q_i$ , used to calculate  $k_{i,q}$  should be determined using only permanent actions and any net effects of wind together with the equivalent actions arising from concentrated forces, including anchorage forces, acting on the panel. For the purposes of calculating  $k_{i,q}$ , concentrated vertical forces should be converted into an equivalent uniformly distributed load on the assumption that the wall is a rigid body e.g. for the load  $F_{i,vert,Ed}$  acting on the wall as shown in Figure 9.8

$$q_i = \frac{2 a F_{i,vert,Ed}}{b_i^2} \quad (9.31)$$

where:

$a$  is the horizontal distance from the force  $F$  to the leeward corner of the wall;

$b$  is the length of the wall.

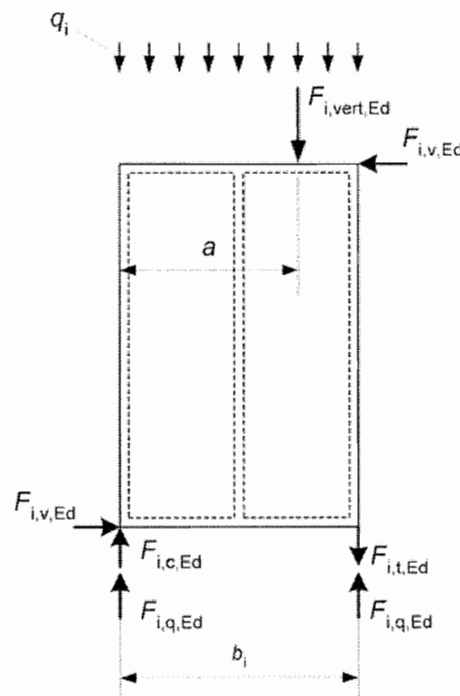


Figure 9.8 – Determination of equivalent vertical action  $q_i$  and reaction forces from vertical and horizontal actions

(6) The external forces  $F_{i,c,Ed}$  and  $F_{i,t,Ed}$  (see Figure 9.8) from the horizontal action  $F_{i,v,Ed}$  on wall  $i$  should be determined from

$$F_{i,c,Ed} = F_{i,t,Ed} = \frac{F_{i,v,Ed} h}{b_1} \quad (9.32)$$

where  $h$  is the height of the wall.

These external forces can be transmitted to either the adjacent panel through the vertical panel-to-panel connection or to the construction above or below the wall. When tensile forces are transmitted to the construction below, the panel should be anchored with stiff fasteners. Compression forces in the vertical members should be checked for buckling in accordance with 6.3.2. Where the ends of vertical members bear on horizontal framing members, the compression perpendicular to the grain stresses in the horizontal members should be assessed according to 6.1.5.

(7) The buckling of the sheets under the action of shear force  $F_{v,Ed}$  may be disregarded provided

$$\frac{b_{net}}{t} \leq 100 \quad (9.33)$$

where:

$b_{net}$  is the clear distance between vertical members of the timber frame;

$t$  is the thickness of the sheathing.

## **9.2.5 Bracing**

### **9.2.5.1 General**

(1)P Structures which are not otherwise adequately stiff shall be braced to prevent instability or excessive deflection.

(2)P The stress caused by geometrical and structural imperfections, and by induced deflections (including the contribution of any joint slip) shall be taken into account.

(3)P The bracing forces shall be determined on the basis of the most unfavourable combination of structural imperfections and induced deflections.

### **9.2.5.2 Single members in compression**

(1) For single elements in compression, requiring lateral support at intervals  $a$  (see Figure 9.9), the initial deviations from straightness between supports should be within  $a/500$  for glued laminated or LVL members, and  $a/300$  for other members.

(2) Each intermediate support should have a minimum spring stiffness  $C$

$$C = k_s \frac{N_d}{a} \quad (9.34)$$

where:

$k_s$  is a modification factor;

$N_d$  is the mean design compressive force in the element;

$a$  is the bay length (see Figure 9.9).

NOTE: For  $k_s$ , see note in 9.2.5.3(1)

(3) The design stabilizing force  $F_d$  at each support should be taken as:

$$F_d = \begin{cases} \frac{N_d}{k_{f,1}} & \text{for solid timber} \\ \frac{N_d}{k_{f,2}} & \text{for glued laminated timber and LVL} \end{cases} \quad (9.35)$$

where  $k_{f,1}$  and  $k_{f,2}$  are modification factors.

NOTE: For  $k_{f,1}$  and  $k_{f,2}$ , see note in 9.2.5.3(1)

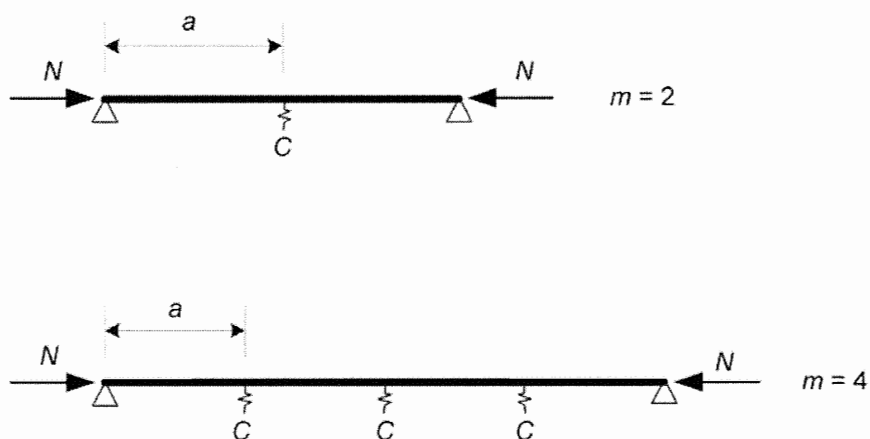


Figure 9.9 – Examples of single members in compression braced by lateral supports.

(4) The design stabilizing force  $F_d$  for the compressive edge of a rectangular beam should be determined in accordance with 9.2.5.2(3) where:

$$N_d = (1 - k_{crit}) \frac{M_d}{h} \quad (9.36)$$

The value of  $k_{crit}$  should be determined from 6.3.3(4) for the unbraced beam, and  $M_d$  is the maximum design moment acting on the beam of depth  $h$ .

### 9.2.5.3 Bracing of beam or truss systems

(1) For a series of  $n$  parallel members which require lateral supports at intermediate nodes A,B, etc. (see Figure 9.10) a bracing system should be provided, which, in addition to the effects of external horizontal load (e.g. wind), should be capable of resisting an internal stability load per unit length  $q$ , as follows:

$$q_d = k_\ell \frac{n N_d}{k_{f,3} \ell} \quad (9.37)$$

where:

$$k_\ell = \min \left\{ 1, \sqrt{\frac{15}{\ell}} \right\} \quad (9.38)$$

$N_d$  is the mean design compressive force in the member;



## Section 10 Structural detailing and control

### 10.1 General

(1)P The provisions given in this section are prerequisite requirements for the design rules given in this standard to apply.

### 10.2 Materials

(1) The deviation from straightness measured midway between the supports should, for columns and beams where lateral instability can occur, or members in frames, be limited to 1/500 times the length of glued laminated timber or LVL members and to 1/300 times the length of solid timber. The limitations on bow in most strength grading rules are inadequate for the selection of material for these members and particular attention should therefore be paid to their straightness.

(2) Timber and wood-based components and structural elements should not be unnecessarily exposed to climatic conditions more severe than those expected in the finished structure.

(3) Before being used in construction, timber should be dried as near as practicable to the moisture content appropriate to its climatic condition in the completed structure. If the effects of any shrinkage are not considered important, or if parts that are unacceptably damaged are replaced, higher moisture contents may be accepted during erection provided that it is ensured that the timber can dry to the desired moisture content.

### 10.3 Glued joints

(1) Where bond strength is a requirement for ultimate limit state design, the manufacture of glued joints should be subject to quality control, to ensure that the reliability and quality of the joint is in accordance with the technical specification.

(2) The adhesive manufacturer's recommendations with respect to mixing, environmental conditions for application and curing, moisture content of members and all factors relevant to the proper use of the adhesive should be followed.

(3) For adhesives which require a conditioning period after initial set, before attaining full strength, the application of load to the joint should be restricted for the necessary time.

### 10.4 Connections with mechanical fasteners

#### 10.4.1 General

(1)P Wane, splits, knots or other defects shall be limited in the region of the connection such that the load-carrying capacity of the connection is not reduced.

#### 10.4.2 Nails

(1) Unless otherwise specified, nails should be driven in at right angles to the grain and to such depth that the surfaces of the nail heads are flush with the timber surface.

(2) Unless otherwise specified, slant nailing should be carried out in accordance with Figure 8.8(b).

(3) The diameter of pre-drilled holes should not exceed  $0,8d$ , where  $d$  is the nail diameter.

#### 10.4.3 Bolts and washers

(1) Bolt holes in timber should have a diameter not more than 1 mm larger than the bolt. Bolt

holes in steel plates should have a diameter not more than 2 mm or  $0,1d$  (whichever is the greater) larger than the bolt diameter  $d$ .

(2) Washers with a side length or a diameter of at least  $3d$  and a thickness of at least  $0,3d$  should be used under the head and nut. Washers should have a full bearing area.

(3) Bolts and lag screws should be tightened so that the members fit closely, and they should be re-tightened if necessary when the timber has reached equilibrium moisture content to ensure that the load-carrying capacity and stiffness of the structure is maintained.

(4) The minimum diameter requirements given in Table 10.1 apply to bolts used with timber connectors, where:

$d_c$  is the connector diameter, in mm;

$d$  is the bolt diameter, in mm

$d_1$  is the diameter of centre hole of connector.

**Table 10.1 – Requirements for diameters of bolts used with timber connectors**

Type of connector EN 912	$d_c$	$d$ minimum	$d$ maximum
	mm	mm	mm
A1 – A6	$\leq 130$	12	24
A1, A4, A6	$> 130$	$0,1 d_c$	24
B		$d_1 - 1$	$d_1$

#### 10.4.4 Dowels


(1) The minimum dowel diameter should be 6 mm. The tolerances on the dowel diameter should be  $-0/+0,1$  mm. Pre-bored holes in the timber members should have a diameter not greater than the dowel.

#### 10.4.5 Screws

(1) For pre-drilling screws in softwoods with a smooth shank diameter  $d \leq 6$  mm, pre-drilling is not required. For all screws in hardwoods and for pre-drilling screws in softwoods with a diameter  $d > 6$  mm, pre-drilling is required, with the following requirements:

- The lead hole for the shank should have the same diameter as the shank and the same depth as the length of the shank
- The lead hole for the threaded portion should have a diameter of approximately 70 % of the shank diameter.

(2) For timber densities greater than  $500 \text{ kg/m}^3$ , the pre-drilling diameter should be determined by tests.

(3)P Where pre-drilling is applied to selfdrilling screws, the lead hole diameter shall not be greater than the inner thread diameter  $d_1$ . 

#### 10.5 Assembly

(1) The structure should be assembled in such a way that over-stressing of its members or connections is avoided. Members which are warped, split or badly fitting at the joints should be replaced.

## 10.6 Transportation and erection

(1) The over-stressing of members during storage, transportation or erection should be avoided. If the structure is loaded or supported in a different manner during construction than in the finished building the temporary condition should be considered as a relevant load case, including any possible dynamic actions. In the case of structural framework, e.g. framed arches, portal frames, special care should be taken to avoid distortion during hoisting from the horizontal to the vertical position.

## 10.7 Control

- (1) It is assumed that a control plan comprises:
- production and workmanship control off and on site;
  - control after completion of the structure.

NOTE 1: The control of the construction is assumed to include:

- preliminary tests, e.g. tests for suitability of materials and production methods;
- checking of materials and their identification e.g.:
  - for wood and wood-based materials: species, grade, marking, treatments and moisture content;
  - for glued constructions: adhesive type, production process, glue-line quality;
  - for fasteners: type, corrosive protection;
- transport, site storage and handling of materials;
- checking of correct dimensions and geometry;
- checking of assembly and erection;
- checking of structural details, e.g.:
  - number of nails, bolts etc.;
  - sizes of holes, correct pre-drilling;
  - spacings and distances to end and edge of members;
  - splitting;
- final checking of the result of the production process, e.g. by visual inspection or proof loading.

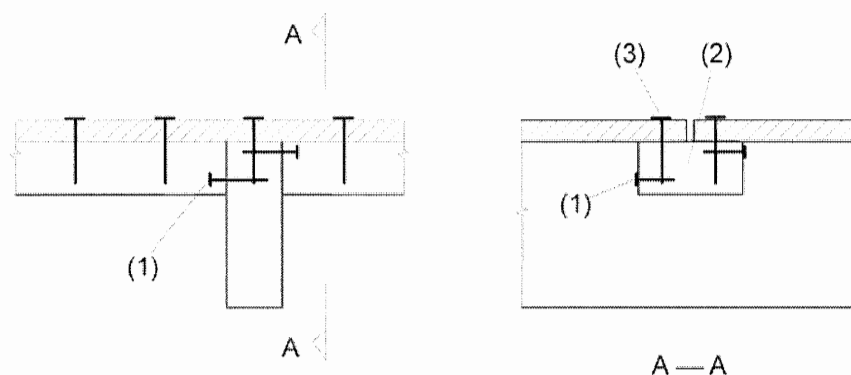
NOTE 2: A control program is assumed to specify the control measures (inspection maintenance) to be carried out in service where long-term compliance with the basic assumptions for the project is not adequately ensured.

NOTE 3: All the information required for the use in service and the maintenance of a structure is assumed to be made available to the person or authority who undertakes responsibility for the finished structure.

## 10.8 Special rules for diaphragm structures

### 10.8.1 Floor and roof diaphragms

(1) The simplified method of analysis given in 9.2.3.2 assumes that sheathing panels not supported by joists or rafters are connected to each other e.g. by means of battens as shown in Figure 10.1. Nails other than smooth nails, as defined in EN 14592, or screws should be used, with a maximum spacing along the edges of the sheathing panels of 150 mm. Elsewhere the maximum spacing should be 300 mm.



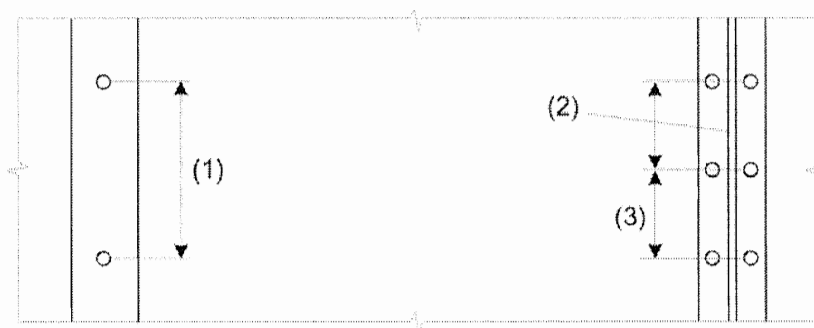
Key:

- (1) Batten slant nailed to joist or rafter
- (2) Batten
- (3) Sheathing nailed to batten

**Figure 10.1 – Example of connection of panels not supported by a joist or a rafter**

### 10.8.2 Wall diaphragms

(1) The simplified methods of analysis given in 9.2.4.2 and 9.2.4.3 assume that panel fixings have a maximum fastener spacing along the edges of 150 mm for nails, and 200 mm for screws. On internal studs the maximum spacing should be no more than twice the spacing along the edge or 300 mm, whichever is the lesser. See Figure 10.2.



Key:

- (1) Maximum nail spacing 300 mm to intermediate studs
- (2) Panel edge
- (3) Maximum nail spacing 150 mm

**Figure 10.2 – Panel fixings**

## 10.9 Special rules for trusses with punched metal plate fasteners

### 10.9.1 Fabrication

Note: Requirements for the fabrication of trusses are given in EN 14250.

### 10.9.2 Erection

(1) Trusses should be checked for straightness and vertical alignment prior to fixing the permanent bracing.

(2) When trusses are fabricated, the members should be free from distortion within the limits given in EN 14250. However, if members which have distorted during the period between fabrication and erection can be straightened without damage to the timber or the joints and maintained straight, the truss may be considered satisfactory for use.

(3) The maximum bow  $a_{\text{bow}}$  in any truss member after erection should be limited. Provided that it is adequately secured in the completed roof to prevent the bow from increasing, the permitted value of the maximum bow should be taken as  $a_{\text{bow,perm}}$ .

Note: The recommended range of  $a_{\text{bow,perm}}$  is 10 to 50 mm. The National choice may be given in the National annex.

(4) The maximum deviation  $a_{\text{dev}}$  of a truss from true vertical alignment after erection should be limited. The permitted value of the maximum deviation from true vertical alignment should be taken as  $a_{\text{dev,perm}}$ .

Note: The recommended range of  $a_{\text{dev,perm}}$  is 10 to 50 mm. The National choice may be given in the National annex.

## Annex A (Informative): Block shear and plug shear failure at multiple dowel-type steel-to-timber connections

(1) For steel-to-timber connections comprising multiple dowel-type fasteners subjected to a force component parallel to grain near the end of the timber member, the characteristic load-carrying capacity of fracture along the perimeter of the fastener area, as shown in Figure A.1 (block shear failure) and Figure A.2 (plug shear failure), should be taken as:

$$F_{bs,Rk} = \max \begin{cases} 1,5 A_{net,t} f_{t,0,k} \\ 0,7 A_{net,v} f_{v,k} \end{cases} \quad (A.1)$$

with

$$A_{net,t} = L_{net,t} t_1 \quad (A.2)$$

$$A_{net,v} = \begin{cases} L_{net,v} t_1 & \text{failure modes (c, f, j/l, k, m)} \\ \frac{L_{net,v}}{2} (L_{net,t} + 2t_{ef}) & \text{all other failure modes } \langle A_1 \rangle \end{cases} \quad (A.3)$$

and

$$L_{net,v} = \sum_i l_{v,i} \quad (A.4)$$

$$L_{net,t} = \sum_i l_{t,i} \quad (A.5)$$

– for thin steel plates (for failure modes given in brackets)

$$t_{ef} = \begin{cases} 0,4 t_1 & (a) \\ 1,4 \sqrt{\frac{M_{y,Rk}}{f_{h,k} d}} & (b) \end{cases} \quad (A.6)$$

– for thick steel plates (for failure modes given in brackets)

$$t_{ef} = \begin{cases} 2 \sqrt{\frac{M_{y,Rk}}{f_{h,k} d}} & (e)(h) \\ t_1 \left[ \sqrt{2 + \frac{M_{y,Rk}}{f_{h,k} d t_1^2}} - 1 \right] & (d)(g) \langle A_1 \rangle \end{cases} \quad (A.7)$$

where

$F_{bs,Rk}$  is the characteristic block shear or plug shear capacity;

$A_{net,t}$  is the net cross-sectional area perpendicular to the grain;

$A_{net,v}$  is the net shear area in the parallel to grain direction;

$L_{net,t}$  is the net width of the cross-section perpendicular to the grain;

$L_{net,v}$  is the total net length of the shear fracture area;

$\ell_{v,i}, \ell_{t,i}$  are defined in figure A.1;

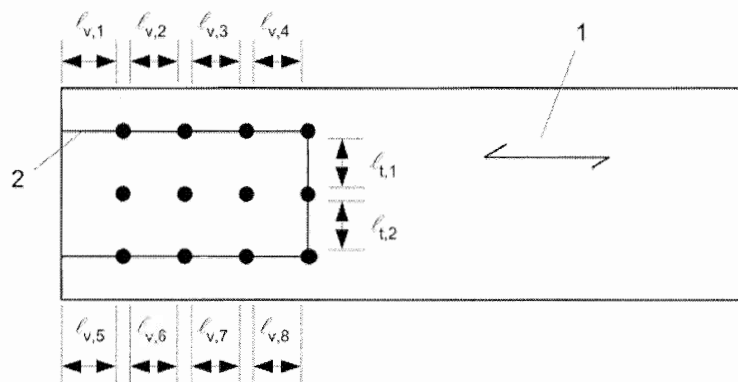
$t_{ef}$  is the effective depth depending of the failure mode of the fastener, see Figure 8.3;

$t_1$  is the timber member thickness or penetration depth of the fastener;

$M_{y,Rk}$  is the characteristic yield moment of the fastener;

- $d$  is the fastener diameter;  
 $f_{t,0,k}$  is the characteristic tensile strength of the timber member;  
 $f_{v,k}$  is the characteristic shear strength of the timber member;  
 $f_{h,k}$  is the characteristic embedding strength of the timber member.

NOTE: The failure modes associated with expressions (A.3), (A.6) and (A.7) are shown in Figure 8.3.



Key:

- 1 Grain direction
- 2 Fracture line

Figure A.1 – Example of block shear failure

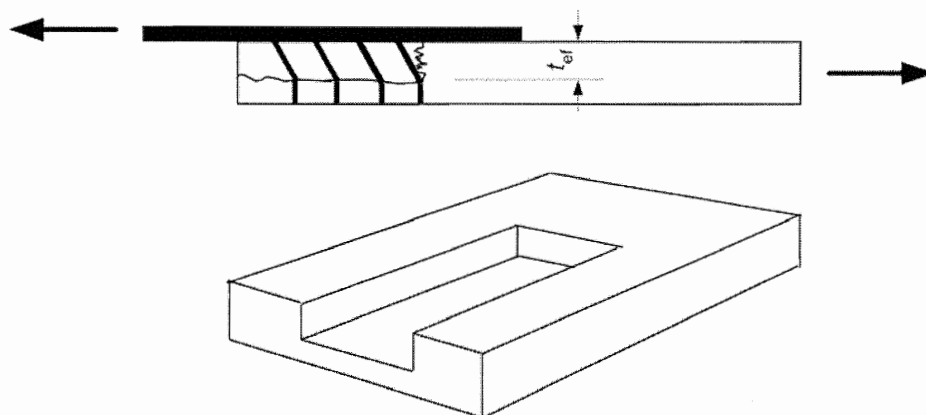


Figure A.2 – Example of plug shear failure

## Annex B (Informative): Mechanically jointed beams

### B.1 Simplified analysis

#### B.1.1 Cross-sections

(1) The cross-sections shown in Figure B.1 are considered in this annex.

#### B.1.2 Assumptions

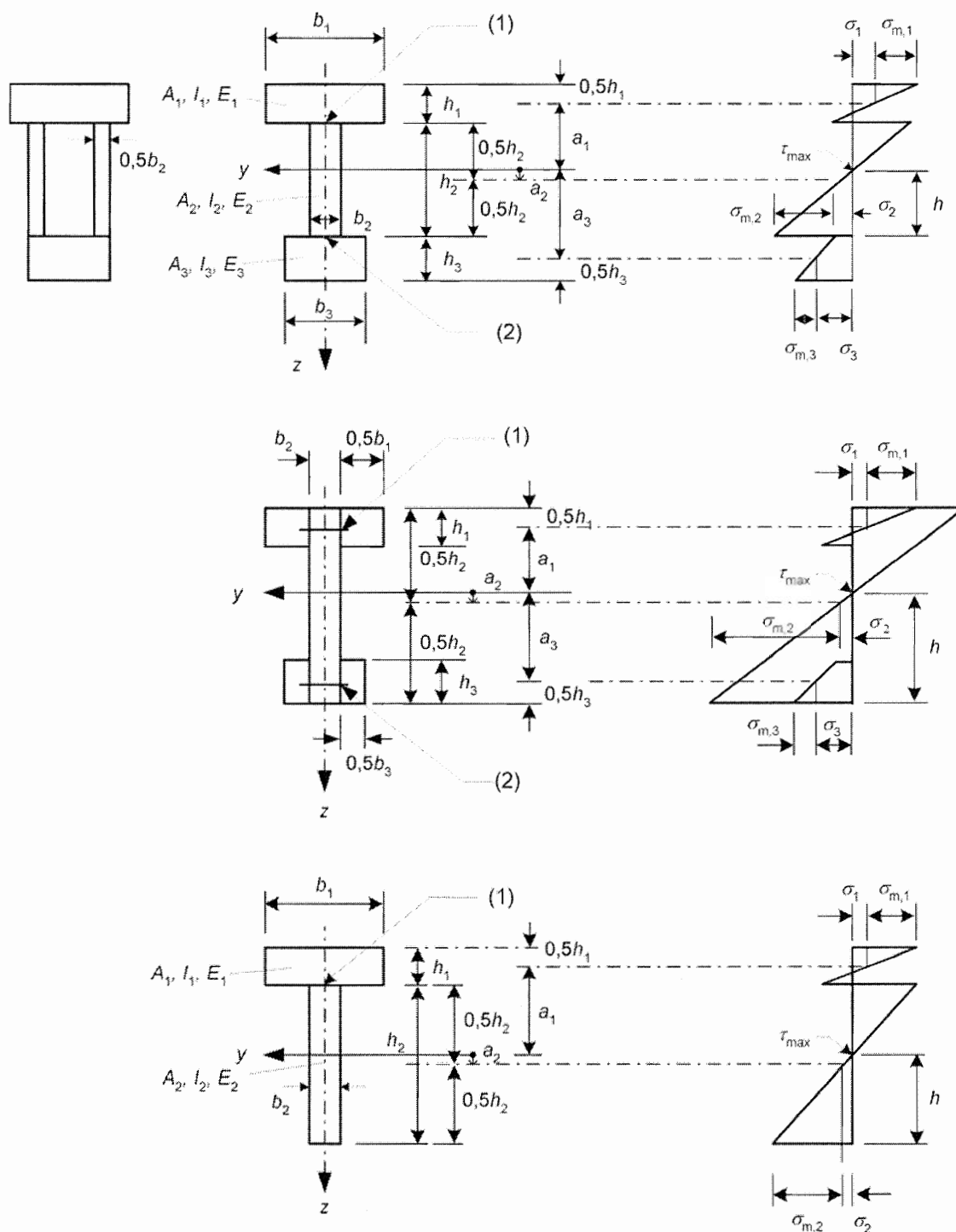
- (1) The design method is based on the theory of linear elasticity and the following assumptions:
- the beams are simply supported with a span  $\ell$ . For continuous beams the expressions may be used with  $\ell$  equal to 0,8 of the relevant span and for cantilevered beams with  $\ell$  equal to twice the cantilever length
  - the individual parts (of wood, wood-based panels) are either full length or made with glued end joints
  - the individual parts are connected to each other by mechanical fasteners with a slip modulus  $K$
  - the spacing  $s$  between the fasteners is constant or varies uniformly according to the shear force between  $s_{\min}$  and  $s_{\max}$ , with  $s_{\max} \leq 4 s_{\min}$
  - the load is acting in the z-direction giving a moment  $M = M(x)$  varying sinusoidally or parabolically and a shear force  $V = V(x)$ .

#### B.1.3 Spacings

(1) Where a flange consists of two parts jointed to a web or where a web consists of two parts (as in a box beam), the spacing  $s_i$  is determined by the sum of the fasteners per unit length in the two jointing planes.

#### B.1.4 Deflections resulting from bending moments

(1) Deflections are calculated by using an effective bending stiffness  $(EI)_{\text{ef}}$  determined in accordance with B.2.



Key:

(1) spacing:  $s_1$  slip modulus:  $K_1$  load:  $F_1$   
(2) spacing:  $s_3$  slip modulus:  $K_3$  load:  $F_3$

Figure B.1 – Cross-section (left) and distribution of bending stresses (right). All measurements are positive except for  $a_2$  which is taken as positive as shown.

## B.2 Effective bending stiffness

(1) The effective bending stiffness should be taken as:

$$(EI)_{\text{ef}} = \sum_{i=1}^3 (E_i I_i + \gamma_i E_i A_i a_i^2) \quad (\text{B.1})$$

using mean values of  $E$  and where:

$$A_i = b_i h_i \quad (\text{B.2})$$

$$I_i = \frac{b_i h_i^3}{12} \quad (\text{B.3})$$

$$\gamma_2 = 1 \quad (\text{B.4})$$

$$\gamma_i = \left[ 1 + \pi^2 E_i A_i s_i / (K_i I_i^2) \right]^{-1} \quad \text{for } i = 1 \text{ and } i = 3 \quad (\text{B.5})$$

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2) - \gamma_3 E_3 A_3 (h_2 + h_3)}{2 \sum_{i=1}^3 \gamma_i E_i A_i} \quad (\text{B.6})$$

where the symbols are defined in Figure B.1;

$K_i = K_{\text{ser},i}$  for the serviceability limit state calculations;

$K_i = K_{\text{u},i}$  for the ultimate limit state calculations.

For T-sections  $h_3 = 0$

## B.3 Normal stresses

(1) The normal stresses should be taken as:

$$\sigma_i = \frac{\gamma_i E_i a_i M}{(EI)_{\text{ef}}} \quad (\text{B.7})$$

$$\sigma_{\text{m},i} = \frac{0,5 E_i h_i M}{(EI)_{\text{ef}}} \quad (\text{B.8})$$

## B.4 Maximum shear stress

(1) The maximum shear stresses occur where the normal stresses are zero. The maximum shear stresses in the web member (part 2 in Figure B.1) should be taken as:

$$\tau_{2,\text{max}} = \frac{\gamma_3 E_3 A_3 a_3 + 0,5 E_2 b_2 h_2^2}{b_2 (EI)_{\text{ef}}} V \quad (\text{B.9})$$

## B.5 Fastener load

(1) The load on a fastener should be taken as:

$$F_i = \frac{\gamma_i E_i A_i a_i s_i}{(EI)_{\text{ef}}} V \quad (\text{B.10})$$

where:

$i = 1$  and  $3$ , respectively;

$s_i = s_i(x)$  is the spacing of the fasteners as defined in B.1.3(1).

## Annex C (Informative): Built-up columns

### C.1 General

#### C.1.1 Assumptions

(1) The following assumptions apply:

- the columns are simply supported with a length  $\ell$ ;
- the individual parts are full length;
- the load is an axial force  $F_c$  acting at the geometric centre of gravity, (see however C.2.3).

#### C.1.2 Load-carrying capacity

(1) For column deflection in the y-direction (see Figure C.1 and Figure C.3) the load-carrying capacity should be taken as the sum of the load-carrying capacities of the individual members.

(2) For column deflection in the z-direction (see Figure C.1 and Figure C.3) it should be verified that:

$$\sigma_{c,0,d} \leq k_c f_{c,0,d} \quad (C.1)$$

where:

$$\sigma_{c,0,d} = \frac{F_{c,d}}{A_{tot}} \quad (C.2)$$

where:

$A_{tot}$  is the total cross-sectional area;

$k_c$  is determined in accordance with 6.3.2 but with an effective slenderness ratio  $\lambda_{ef}$  determined in accordance with sections C.2 - C.4.

### C.2 Mechanically jointed columns

#### C.2.1 Effective slenderness ratio

(1) The effective slenderness ratio should be taken as:

$$\lambda_{ef} = \ell \sqrt{\frac{A_{tot}}{I_{ef}}} \quad (C.3)$$

with

$$I_{ef} = \frac{(EI)_{ef}}{E_{mean}} \quad (C.4)$$

where  $(EI)_{ef}$  is determined in accordance with Annex B (informative).

#### C.2.2 Load on fasteners

(1) The load on a fastener should be determined in accordance with Annex B (informative), where

$$V_d = \begin{cases} \frac{F_{c,d}}{120 k_c} & \text{for } \lambda_{ef} < 30 \\ \frac{F_{c,d} \lambda_{ef}}{3600 k_c} & \text{for } 30 \leq \lambda_{ef} < 60 \\ \frac{F_{c,d}}{60 k_c} & \text{for } 60 \leq \lambda_{ef} \end{cases} \quad (C.5)$$

### C.2.3 Combined loads

(1) In cases where small moments (e.g. from self weight) are acting in addition to axial load, 6.3.2(3) applies.

## C.3 Spaced columns with packs or gussets

### C.3.1 Assumptions

(1) Columns as shown in Figure C.1 are considered, i.e. columns comprising shafts spaced by packs or gussets. The joints may be either nailed or glued or bolted with suitable connectors.

(2) The following assumptions apply:

- the cross-section is composed of two, three or four identical shafts;
- the cross-sections are symmetrical about both axes;
- the number of unrestrained bays is at least three, i.e. the shafts are at least connected at the ends and at the third points;
- the free distance  $a$  between the shafts is not greater than three times the shaft thickness  $h$  for columns with packs and not greater than 6 times the shaft thickness for columns with gussets;
- the joints, packs and gussets are designed in accordance with C.3.3;  $\overline{A_1}$
- the pack length  $\ell_2$  satisfies the condition:  $\ell_2/a \geq 1,5$ ;
- there are at least four nails or two bolts with connectors in each shear plane. For nailed joints there are at least four nails in a row at each end in the longitudinal direction of the column;
- the gussets satisfies the condition:  $\ell_2/a \geq 2$ ;
- the columns are subjected to concentric axial loads.

(3) For columns with two shafts  $A_{tot}$  and  $I_{tot}$  should be calculated as

$$A_{tot} = 2A \quad (C.6)$$

$$I_{tot} = \frac{b \left[ (2h+a)^3 - a^3 \right]}{12} \quad (C.7)$$

(4) For columns with three shafts  $A_{tot}$  and  $I_{tot}$  should be calculated as

$$A_{tot} = 3A \quad (C.8)$$

$$I_{tot} = \frac{b \left[ (3h+2a)^3 - (h+2a)^3 + h^3 \right]}{12} \quad (C.9)$$

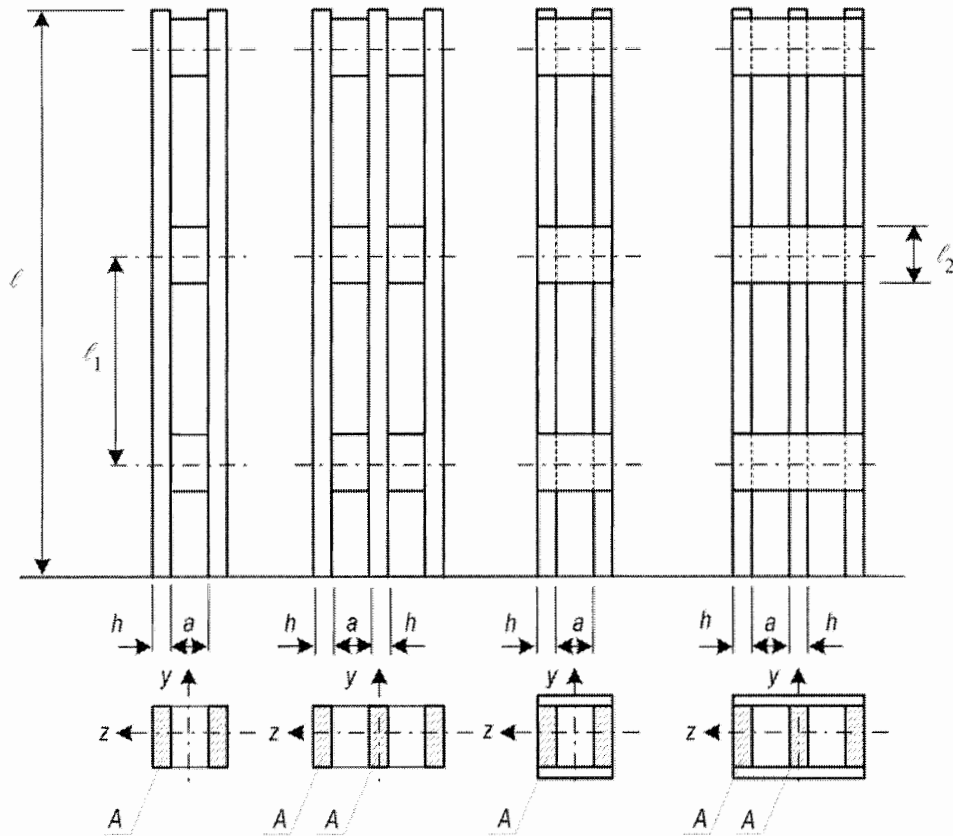


Figure C.1 – Spaced columns

### C.3.2 Axial load-carrying capacity

**A1** (1) For column deflection in the y-direction (see Figure C.1) the load-carrying capacity should be taken as the sum of the load-carrying capacities of the individual members. **A1**

(2) For column deflection in the z-direction C.1.2 applies with

$$\lambda_{\text{ef}} = \sqrt{\lambda^2 + \eta \frac{n}{2} \lambda_1^2} \quad (\text{C.10})$$

where:

$\lambda$  is the slenderness ratio for a solid column with the same length, the same area ( $A_{\text{tot}}$ ) and the same second moment of area ( $I_{\text{tot}}$ ), i.e.,

$$\lambda = \ell \sqrt{A_{\text{tot}} / I_{\text{tot}}} \quad (\text{C.11})$$

$\lambda_1$  is the slenderness ratio for the shafts and has to be set into expression (C.10) with a minimum value of at least 30, i.e.

$$\lambda_1 = \sqrt{12} \frac{\ell_1}{h} \quad (\text{C.12})$$

$n$  is the number of shafts;

$\eta$  is a factor given in Table C.1.

Table C.1 – The factor  $\eta$

	Packs			Gussets	
	Glued	Nailed	Bolted <sup>a</sup>	Glued	Nailed
Permanent/long-term loading	1	4	3,5	3	6
Medium/short-term loading	1	3	2,5	2	4,5
<sup>a</sup> with connectors					

### C.3.3 Load on fasteners, gussets or packs

(1) The load on the fasteners and the gussets or packs are as shown in Figure C.2 with  $V_d$  according to section C.2.2.

(2) The shear forces on the gussets or packs, see Figure C.2, should be calculated from:

$$T_d = \frac{V_d l_1}{a_1} \quad (\text{C.13})$$

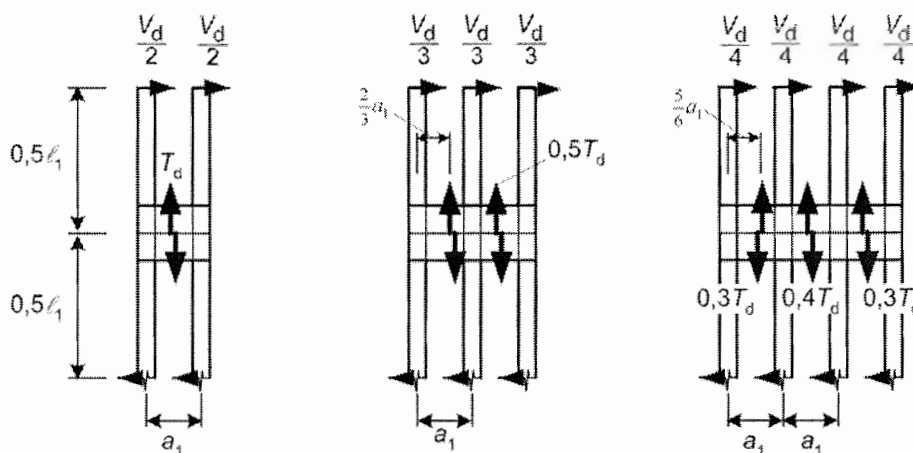


Figure C.2 – Shear force distribution and loads on gussets or packs

## C.4 Lattice columns with glued or nailed joints

### C.4.1 Assumptions

(1) Lattice columns with N- or V-lattice configurations and with glued or nailed joints are considered in this section, see Figure C.3.

(2) The following assumptions apply:

- the structure is symmetrical about the y- and z-axes of the cross-section. The lattice on the two sides may be staggered by a length of  $l_1/2$ , where  $l_1$  is the distance between the nodes;
- there are at least three bays;
- in nailed structures there are at least four nails per shear plane in each diagonal at each nodal point;

- each end is braced;
- the slenderness ratio of the individual flange corresponding to the node length  $\ell_1$  is not greater than 60;
- no local buckling occurs in the flanges corresponding to the column length  $\ell_1$ ;
- the number of nails in the verticals (of an N-truss) is greater than  $n \sin \theta$ , where  $n$  is the number of nails in the diagonals and  $\theta$  is the inclination of the diagonals.

#### **C.4.2 Load-carrying capacity**

(1) For column deflection in the y-direction (see Figure C.2), the load-carrying capacity should be taken as the sum of the load-carrying capacities of the individual flanges.

(2) For column deflection in the z-direction C.1.2 applies with

$$\lambda_{\text{ef}} = \max \left\{ \lambda_{\text{tot}} \sqrt{1 + \mu}, 1,05 \lambda_{\text{tot}} \right\} \quad (\text{C.14})$$

where:

$\lambda_{\text{tot}}$  is the slenderness ratio for a solid column with the same length, the same area and the same second moment of area, i.e.

$$\lambda_{\text{tot}} \approx \frac{2\ell}{h} \quad (\text{C.15})$$

$\mu$  takes the values given in (3) to (6) below.

(3) For a glued V-truss:

$$\mu = 4 \frac{e^2 A_f}{I_f} \left( \frac{h}{\ell} \right)^2 \quad (\text{C.16})$$

where(see Figure C.3):

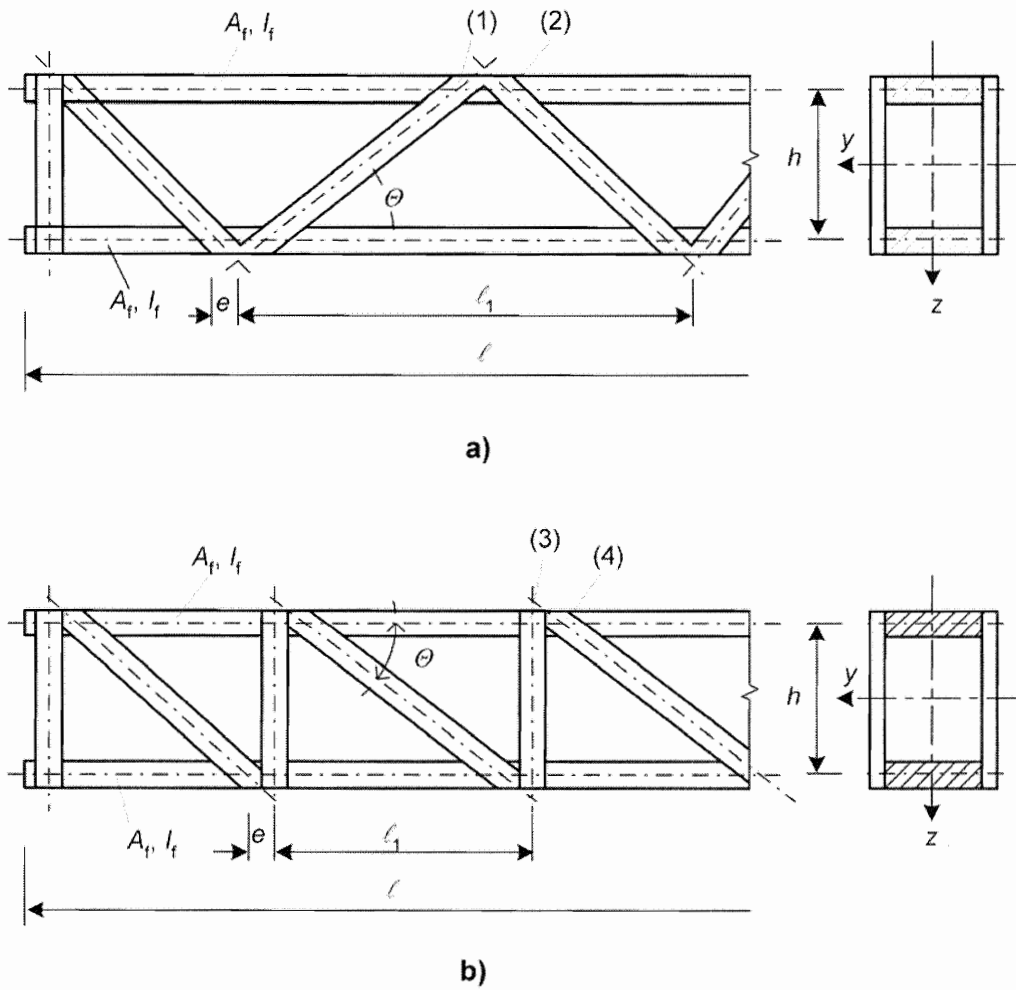
$e$  is the eccentricity of the joints;

$A_f$  is the area of the flange;

$I_f$  is the second moment of area of the flange;

$\ell$  is the span;

$h$  is the distance of the flanges.



Key:

- (1) number of nails:  $n$
- (2) number of nails:  $n$
- (3) number of nails:  $\geq n \sin \theta$
- (4) number of nails:  $n$

Figure C.3 – Lattice columns: (a) V-truss, (b) N-truss

(4) For a glued N-truss:

$$\mu = \frac{e^2 A_f}{I_f} \left( \frac{h}{\ell} \right)^2 \quad (\text{C.17})$$

(5) For a nailed V-truss:

$$\mu = 25 \frac{h E_{\text{mean}} A_f}{\ell^2 n K_u \sin 2\theta} \quad (\text{C.18})$$

where:

- $n$  is the number of nails in a diagonal. If a diagonal consists of two or more pieces,  $n$  is the sum of the nails (not the number of nails per shear plane);

$E_{\text{mean}}$  is the mean value of modulus of elasticity;

$K_u$  is the slip modulus of one nail in the ultimate limit state.

(6) For a nailed N-truss:

$$\mu = 50 \frac{h E_{\text{mean}} A_f}{\ell^2 n K_u \sin 2\theta} \quad (\text{C.19})$$

where:

$n$  is the number of nails in a diagonal. If a diagonal consists of two or more pieces,  $n$  is the sum of the nails (not the number of nails per shear plane);

$K_u$  is the slip modulus of one nail for the ultimate limit states.

#### **C.4.3 Shear forces**

(1) C.2.2 applies.

## **Annex D (Informative): Bibliography**

- EN 338      Structural timber – Strength classes
- EN 1194     Glued laminated timber – Strength classes and determination of characteristic values

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EN 1995-1-2 (2004) (English): Eurocode 5: Design of timber structures - Part 1-2: General - Structural fire design  
[Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]



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English version

## **Eurocode 5: Design of timber structures - Part 1-2: General - Structural fire design**

Eurocode 5: Conception et Calcul des structures en bois -  
Part 1-2: Généralités - Calcul des structures au feu

Eurocode 5: Entwurf, Berechnung und Bemessung von  
Holzbauten - Teil 1-2: Allgemeine Regeln - Bemessung für  
den Brandfall

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## **Foreword**

This European Standard EN 1995-1-2 has been prepared by Technical Committee CEN/TC250 "Structural Eurocodes", the Secretariat of which is held by BSI.

This European Standard shall be given the status of a National Standard, either by publication of an identical text or by endorsement, at the latest by May 2005, and conflicting national standards shall be withdrawn at the latest by March 2010.

This European Standard supersedes ENV 1995-1-2:1994.

CEN/TC250 is responsible for all Structural Eurocodes.

According to the CEN/CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxemburg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

## **Background of the Eurocode programme**

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980's.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement<sup>1</sup> between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to the CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990	Eurocode :	Basis of Structural Design
EN 1991	Eurocode 1:	Actions on structures
EN 1992	Eurocode 2:	Design of concrete structures
EN 1993	Eurocode 3:	Design of steel structures
EN 1994	Eurocode 4:	Design of composite steel and concrete structures
EN 1995	Eurocode 5:	Design of timber structures
EN 1996	Eurocode 6:	Design of masonry structures
EN 1997	Eurocode 7:	Geotechnical design

<sup>1</sup> Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

EN 1998	Eurocode 8:	Design of structures for earthquake resistance
EN 1999	Eurocode 9:	Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

### Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that EUROCODES serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs).

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents<sup>2</sup> referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards<sup>3</sup>. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

### National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National Foreword, and may be followed by a National Annex.

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, *i.e.*:

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic, etc.), e.g. snow map,
- the procedure to be used where alternative procedures are given in the Eurocode.

It may also contain

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<sup>2</sup> According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

<sup>3</sup> According to Art. 12 of the CPD the interpretative documents shall:  
give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary;  
indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc.;  
serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

- decisions on the application of informative annexes,
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

### **Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products**

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works<sup>4</sup>. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

### **Additional information specific to EN 1995-1-2**

EN 1995-1-2 describes the principles, requirements and rules for the structural design of buildings exposed to fire, including the following aspects.

#### *Safety requirements*

EN 1995-1-2 is intended for clients (e.g. for the formulation of their specific requirements), designers, contractors and relevant authorities.

The general objectives of fire protection are to limit risks with respect to the individual, society, neighbouring property, and where required, directly exposed property, in the case of fire.

Construction Products Directive 89/106/EEC gives the following essential requirement for the limitation of fire risks:

"The construction works must be designed and built in such a way, that in the event of an outbreak of fire

- the load-bearing resistance of the construction can be assumed for a specified period of time;
- the generation and spread of fire and smoke within the works is limited;
- the spread of fire to neighbouring construction works is limited;
- the occupants can leave the works or can be rescued by other means;
- the safety of rescue teams is taken into consideration".

According to the Interpretative Document "Safety in Case of Fire"<sup>5</sup> the essential requirement may be observed by following the various fire safety strategies prevailing in the Member States like conventional fire scenarios (nominal fires) or natural fire scenarios (parametric fires), including passive and/or active fire protection measures.

The fire parts of Structural Eurocodes deal with specific aspects of passive fire protection in terms of designing structures and parts thereof for adequate load-bearing resistance and for limiting fire spread as appropriate.

Required functions and levels of performance can be specified either in terms of nominal (standard) fire resistance rating, generally given in National fire regulations, or by referring to the fire safety engineering for assessing passive and active measures.

Supplementary requirements concerning, for example

- the possible installation and maintenance of sprinkler systems;
  - conditions on occupancy of building or fire compartment;
  - the use of approved insulation and coating materials, including their maintenance
- are not given in this document, because they are subject to specification by a competent authority.

<sup>4</sup> see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

<sup>5</sup> see clauses 2.2, 3.2(4) and 4.2.3.3

Numerical values for partial factors and other reliability elements are given as recommended values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and of quality management applies.

#### *Design procedure*

A full analytical procedure for structural fire design would take into account the behaviour of the structural system at elevated temperatures, the potential heat exposure and the beneficial effects of active fire protection systems, together with the uncertainties associated with these three features and the importance of the structure (consequences of failure).

At the present time it is possible to undertake a procedure for determining adequate performance which incorporates some, if not all, of these parameters, and to demonstrate that the structure, or its components, will give adequate performance in a real building fire. However, where the procedure is based on a nominal (standard) fire the classification system, which calls for specific periods of fire resistance, takes into account (though not explicitly), the features and uncertainties described above.

Options for the application of Part 1-2 of EN 1995 are illustrated in figure 1. The prescriptive and performance-based approaches are identified. The prescriptive approach uses nominal fires to generate thermal actions. The performance-based approach, using fire safety engineering, refers to thermal actions based on physical and chemical parameters.

For design according to this part, EN 1991-1-2 is required for the determination of thermal and mechanical actions acting on the structure.

#### *Design aids*

It is expected that design aids based on the calculation models given in EN 1995-1-2, will be prepared by interested external organisations.

The main text of EN 1995-1-2 includes most of the principal concepts and rules necessary for direct application of structural fire design to timber structures.

In an annex F (informative), guidance is given to help the user select the relevant procedures for the design of timber structures.

#### **National annex for EN 1995-1-2**

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1995-1-2 should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1995-1-2 through clauses:

- 2.1.3(2) Maximum temperature rise for separating function in parametric fire exposure;
- 2.3(1)P Partial factor for material properties;
- 2.3(2)P Partial factor for material properties;
- 2.4.2(3) Reduction factor for combination of actions;
- 4.2.1(1) Method for determining cross-sectional properties.

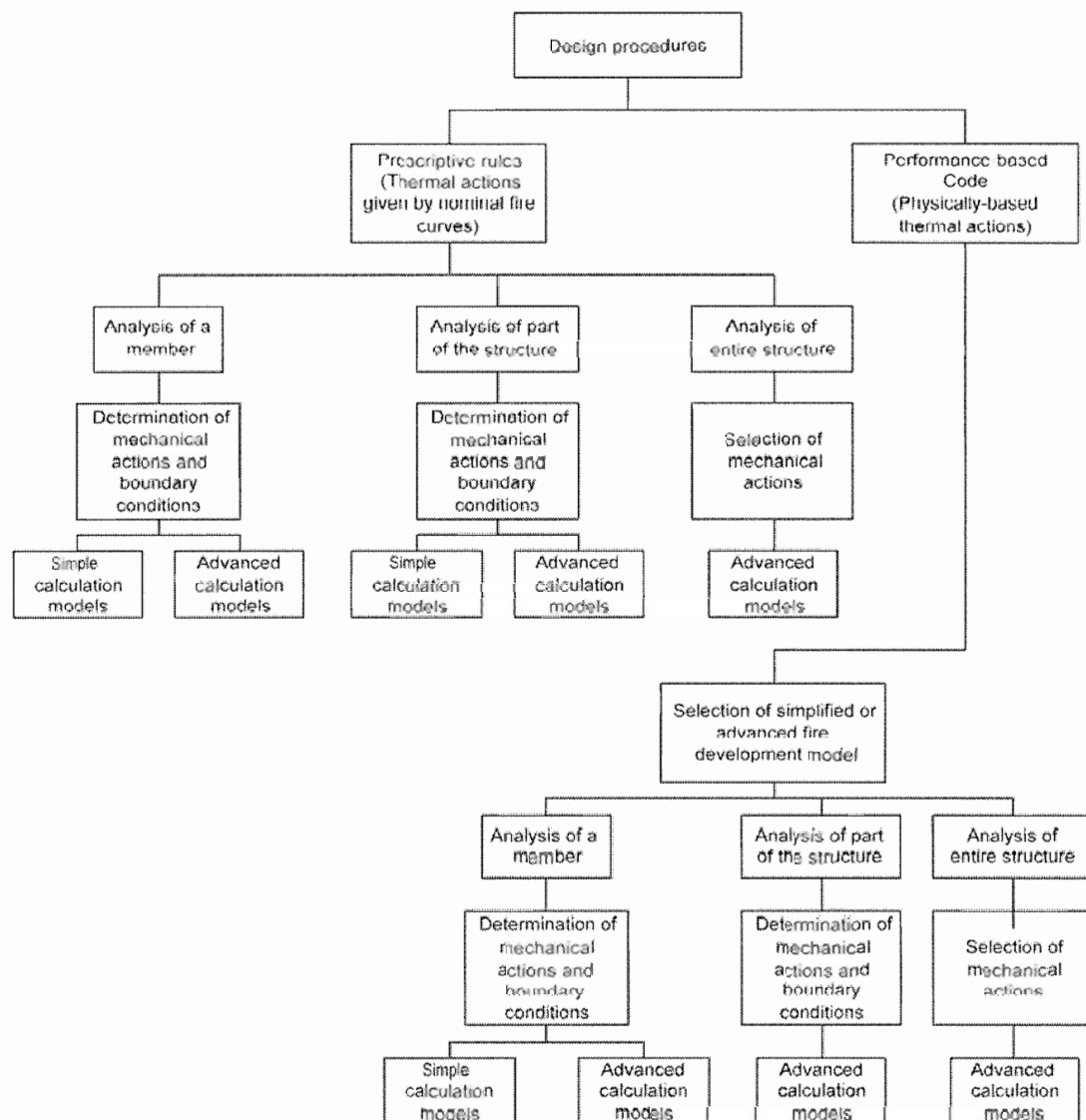


Figure 1 – Alternative design procedures

## **Section 1 General**

### **1.1 Scope**

#### **1.1.1 Scope of Eurocode 5**

(1)P Eurocode 5 applies to the design of buildings and civil engineering works in timber (solid timber, sawn, planed or in pole form, glued laminated timber or wood-based structural products, e.g. LVL) or wood-based panels jointed together with adhesives or mechanical fasteners. It complies with the principles and requirements for the safety and serviceability of structures and the basis of design and verification given in EN 1990:2002.

(2)P Eurocode 5 is only concerned with requirements for mechanical resistance, serviceability, durability and fire resistance of timber structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(3) Eurocode 5 is intended to be used in conjunction with:

EN 1990:2002 Eurocode - Basis of structural design"

EN 1991 "Actions on structures"

EN's for construction products relevant to timber structures

EN 1998 "Design of structures for earthquake resistance", when timber structures are built in seismic regions.

(4) Eurocode 5 is subdivided into various parts:

EN 1995-1 General

EN 1995-2 Bridges

(5) EN 1995-1 "General" comprises:

EN 1995-1-1 General – Common rules and rules for buildings

EN 1995-1-2 General – Structural Fire Design

(6) EN 1995-2 refers to the General rules in EN 1995-1-1. The clauses in EN 1995-2 supplement the clauses in EN 1995-1.

#### **1.1.2 Scope of EN 1995-1-2**

(1)P EN 1995-1-2 deals with the design of timber structures for the accidental situation of fire exposure and is intended to be used in conjunction with EN 1995-1-1 and EN 1991-1-2:2002. EN 1995-1-2 only identifies differences from, or supplements normal temperature design.

(2)P EN 1995-1-2 deals only with passive methods of fire protection. Active methods are not covered.

(3)P EN 1995-1-2 applies to building structures that are required to fulfil certain functions when exposed to fire, in terms of

- avoiding premature collapse of the structure (load-bearing function)
- limiting fire spread (flames, hot gases, excessive heat) beyond designated areas (separating function).

(4)P EN 1995-1-2 gives principles and application rules for designing structures for specified requirements in respect of the aforementioned functions and levels of performance.

(5)P EN 1995-1-2 applies to structures or parts of structures that are within the scope of EN 1995-1-1 and are designed accordingly.

(6)P The methods given in EN 1995-1-2 are applicable to all products covered by product standards made reference to in this Part.

## **1.2 Normative references**

(1)P This European Standard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

European Standards:

EN 300	Oriented strand boards (OSB) – Definition, classification and specifications
EN 301	Adhesives, phenolic and aminoplastic for load-bearing timber structures; classification and performance requirements
EN 309	Wood particleboards – Definition and classification
EN 313-1	Plywood – Classification and terminology. Part 1: Classification
EN 314-2	Plywood – Bonding quality. Part 2: Requirements
EN 316	Wood fibreboards – Definition, classification and symbols
AC2 EN 520	Gypsum plasterboards – Definitions, requirements and test methods AC2
EN 912	Timber fasteners – Specifications for connectors for timber
EN 1363-1	Fire resistance tests – Part 1: General requirements
EN 1365-1	Fire resistance tests for loadbearing elements – Part 1: Walls
EN 1365-2	Fire resistance tests for loadbearing elements – Part 2: Floors and roofs
EN 1990:2002	Eurocode: Basis of structural design
EN 1991-1-1:2002	Eurocode 1 Actions on structures Part 1-1: General actions – Densities, self-weight and imposed loads for buildings
EN 1991-1-2:2002	Eurocode 1: Actions on structures – Part 1-2: General actions – Actions on structures exposed to fire
EN 1993-1-2	Eurocode 3: Design of steel structures – Part 1-2: General – Structural fire design
EN 1995-1-1	Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings
EN 12369-1	Wood-based panels – Characteristic values for structural design – Part 1: OSB, particleboards and fibreboards
EN 13162	Thermal insulation products for buildings – factory-made mineral wool (MW) products – Specifications M/103
ENV 13381-7	Test methods for determining the contribution to the fire resistance of structural members – Part 7: Applied protection to timber members
EN 13986	Wood-based panels for use in construction - Characteristics, evaluation of conformity and marking
EN 14081-1	Timber structures – Strength graded structural timber with rectangular cross section – Part 1, General requirements
EN 14080	Timber structures – Glued laminated timber – Requirements
EN 14374	Timber structures – Structural laminated veneer lumber – Requirements

## **1.3 Assumptions**

(1) In addition to the general assumptions of EN 1990:2002 it is assumed that any passive fire protection systems taken into account in the design of the structure will be adequately maintained.

## **1.4 Distinction between principles and application rules**

(1)P The rules in EN 1990:2002 clause 1.4 apply.

## 1.5 Terms and definitions

(1)P The rules in EN 1990:2002 clause 1.5 and EN 1991-1-2 clause 1.5 apply.

(2)P The following terms and definitions are used in EN 1995-1-2 with the following meanings:

### 1.5.1

**Char-line:** Borderline between the char-layer and the residual cross-section.

### 1.5.2

**Effective cross-section:** Cross-section of member in a structural fire design based on the reduced cross-section method. It is obtained from the residual cross-section by removing the parts of the cross-section with assumed zero strength and stiffness.

### 1.5.3

**Failure time of protection:** Duration of protection of member against direct fire exposure; (e.g. when the fire protective cladding or other protection falls off the timber member, or when a structural member initially protecting the member fails due to collapse, or when the protection from another structural member is no longer effective due to excessive deformation).

### 1.5.4

**Fire protection material:** Any material or combination of materials applied to a structural member or element for the purpose of increasing its fire resistance.

### 1.5.5

**Normal temperature design:** Ultimate limit state design for ambient temperatures according to EN 1995-1-1.

### 1.5.6

**Protected members:** Members for which measures are taken to reduce the temperature rise in the member and to prevent or reduce charring due to fire.

### 1.5.7

**Residual cross-section:** Cross-section of the original member reduced by the charring depth.

## 1.6 Symbols

For the purpose of EN 1995-1-2, the following symbols apply:

*Latin upper case letters*

$A_r$	Area of the residual cross-section
$A_t$	Total area of floors, walls and ceilings that enclose the fire compartment
$A_v$	Total area of vertical openings of fire compartment
$E_d$	Design effect of actions
$E_{d,fi}$	Design modulus of elasticity in fire; design effect of actions for the fire situation
$F_{Ed,fi}$	Design effect of actions on a connection for the fire situation
$F_{R,0,2}$	20 % fractile of a resistance
$F_{Rk}$	Characteristic mechanical resistance of a connection at normal temperature without the effect of load duration and moisture ( $k_{mod} = 1$ )
$G_{d,fi}$	Design shear modulus in fire
$G_k$	Characteristic value of permanent action
$K_{fi}$	Slip modulus in the fire situation
$K_u$	Slip modulus for the ultimate limit state at normal temperature
$L$	Height of storey
$O$	Opening factor
$Q_{k,1}$	Characteristic value of leading variable action

$S_{05}$	5 % fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature
$S_{20}$	20 % fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature
$S_{d,fi}$	Design stiffness property (modulus of elasticity or shear modulus) in the fire situation
$W_{ef}$	Section modulus of effective cross-section
$W_r$	Section modulus of residual cross-section

*Latin lower case letters*

$a_0$	Parameter
$a_1$	Parameter
$a_2$	Distance
$a_3$	Distance
$a_{fi}$	Extra thickness of member for improved mechanical resistance of connections
$b$	Width; thermal absorptivity for the total enclosure
$b_0$	Parameter
$b_1$	Parameter
$c$	Specific heat
$d$	Diameter of fastener
$d_0$	Depth of layer with assumed zero strength and stiffness
$d_{char,0}$	Charring depth for one-dimensional charring
$d_{char,n}$	Notional charring depth
$d_{ef}$	Effective charring depth
$d_g$	Gap depth
$f_{20}$	20 % fractile strength at normal temperature
$f_{d,fi}$	Design strength in fire
$f_k$	Characteristic strength
$f_{v,k}$	Characteristic shear strength
$h_{eq}$	Weighted average of heights of all vertical openings in the fire compartment
$h_{ins}$	Insulation thickness
$h_p$	Fire protective panel thickness
$k$	Parameter
$k_p$	Density coefficient
$k_0$	Coefficient
$k_2$	Insulation coefficient
$k_3$	Post-protection coefficient
$k_{fi}$	Coefficient
$k_{flux}$	Heat flux coefficient for fasteners
$k_h$	Panel thickness coefficient
$k_j$	Joint coefficient
$k_{mod}$	Modification factor for duration of load and moisture content
$k_{mod,E,fi}$	Modification factor for modulus of elasticity in the fire situation
$k_{mod,fi}$	Modification factor for fire
$k_{mod,fm,fi}$	Modification factor for bending strength in the fire situation
$k_n$	Notional cross-section coefficient
$k_{pos}$	Position coefficient
$k_{\Theta}$	Temperature-dependent reduction factor for local strength or stiffness property
$l_a$	Penetration length of fastener into unburnt timber
$l_{a,min}$	Minimum anchorage length of fastener
$l_f$	Length of fastener
$l_p$	Span of the panel
$p$	Perimeter of the fire exposed residual cross-section
$q_{t,d}$	Design fire load density related to the total area of floors, walls and ceilings which enclose the fire compartment
$t$	Time of fire exposure
$t_0$	Time period with a constant charring rate

$t_1$	Thickness of the side member
$t_{ch}$	Time of start of charring of protected members (delay of start of charring due to protection)
$t_{d,fi}$	Time of the fire resistance of the unprotected connection
$t_f$	Failure time of protection
$t_{ins}$	Time of temperature increase on the unexposed side of the construction
$t_{ins,0,i}$	Basic insulation value of layer "i"
$t_{p,min}$	Minimum thickness of panel
$t_R$	Time of fire resistance with respect to the load-bearing function
$t_{req}$	Required time of fire resistance
$y$	Co-ordinate
$z$	Co-ordinate

*Greek upper case letters*

$\Gamma$	Factor accounting for the thermal properties of the boundaries of the compartment
$\Theta$	Temperature

*Greek lower case letters*

$\beta_0$	Design charring rate for one-dimensional charring under standard fire exposure
$\beta_n$	Design notional charring rate under standard fire exposure
$\beta_{par}$	Design charring rate during heating phase of parametric fire curve
$\eta$	Conversion factor for the reduction of the load-bearing capacity in fire
$\eta_f$	Conversion factor for slip modulus
$\gamma_{GA}$	Partial factor for permanent actions in accidental design situations
$\gamma_M$	Partial factor for a material property, also accounting for model uncertainties and dimensional variations
$\gamma_{M,fi}$	Partial factor for timber in fire
$\gamma_{Q,1}$	Partial factor for leading variable action
$\lambda$	Thermal conductivity
$\rho$	Density
$\rho_k$	Characteristic density
$\omega$	Moisture content
$\psi_{1,1}$	Combination factor for frequent value of a variable action
$\psi_{2,1}$	Combination factor for quasi-permanent value of a variable action
$\psi_{fi}$	Combination factor for frequent values of variable actions in the fire situation

## **Section 2 Basis of design**

### **2.1 Requirements**

#### **2.1.1 Basic requirements**

(1)P Where mechanical resistance in the case of fire is required, structures shall be designed and constructed in such a way that they maintain their load-bearing function during the relevant fire exposure.

(2)P Where fire compartmentation is required, the elements forming the boundaries of the fire compartment, including joints, shall be designed and constructed in such a way that they maintain their separating function during the relevant fire exposure. This shall include, when relevant, ensuring that:

- integrity failure does not occur;
- insulation failure does not occur;
- thermal radiation from the unexposed side is limited.

NOTE 1: See EN 1991-1-2:2002 for definitions.

NOTE 2: There is no risk of fire spread due to thermal radiation when an unexposed surface temperature is below 300°C.

(3)P Deformation criteria shall be applied where the means of protection, or the design criteria for separating elements, require that the deformation of the load-bearing structure is taken into account.

(4) Consideration of the deformation of the load-bearing structure is not necessary in the following cases, as relevant:

- the efficiency of the means of protection has been proved according to 3.4.3 or 5.2;
- the separating elements fulfil the requirements of a nominal fire exposure.

#### **2.1.2 Nominal fire exposure**

(1)P For standard fire exposure, elements shall comply with criteria R, E and I as follows:

- separating function only: integrity (criterion E) and, when requested, insulation (criterion I);
- load-bearing function only: mechanical resistance (criterion R);
- separating and load-bearing functions: criteria R, E and, when requested, I.

(2) Criterion R is assumed to be satisfied when the load-bearing function is maintained during the required time of fire exposure.

(3) Criterion I may be assumed to be satisfied where the average temperature rise over the whole of the non-exposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K.

#### **2.1.3 Parametric fire exposure**

(1) The load-bearing function should be maintained during the complete duration of the fire including the decay phase, or a specified period of time.

(2) For the verification of the separating function the following applies, assuming that the normal temperature is 20°C:

- the average temperature rise of the unexposed side of the construction should be limited to 140 K and the maximum temperature rise of the unexposed side should not exceed 180 K

during the heating phase until the maximum temperature in the fire compartment is reached;

- the average temperature rise of the unexposed side of the construction should be limited to  $\Delta\theta_1$  and the maximum temperature rise of the unexposed side should not exceed  $\Delta\theta_2$  during the decay phase.

NOTE: The recommended values for maximum temperature rise during the decay phase are  $\Delta\theta_1 = 200$  K and  $\Delta\theta_2 = 240$  K. Information on National choice may be found in the National annex.

## 2.2 Actions

(1)P Thermal and mechanical actions shall be taken from EN 1991-1-2:2002.

(2) For surfaces of wood, wood-based materials and gypsum plasterboard the emissivity coefficient should be taken as equal to 0,8.

## 2.3 Design values of material properties and resistances

(1)P For verification of mechanical resistance, the design values of strength and stiffness properties shall be determined from

$$f_{d,fi} = k_{mod,fi} \frac{f_{20}}{\gamma_{M,fi}} \quad (2.1)$$

$$S_{d,fi} = k_{mod,fi} \frac{S_{20}}{\gamma_{M,fi}} \quad (2.2)$$

where:

- $f_{d,fi}$  is the design strength in fire;
- $S_{d,fi}$  is the design stiffness property (modulus of elasticity  $E_{d,fi}$  or shear modulus  $G_{d,fi}$ ) in fire;
- $f_{20}$  is the 20 % fractile of a strength property at normal temperature;
- $S_{20}$  is the 20 % fractile of a stiffness property (modulus of elasticity or shear modulus ) at normal temperature;
- $k_{mod,fi}$  is the modification factor for fire;
- $\gamma_{M,fi}$  is the partial safety factor for timber in fire.

NOTE 1: The modification factor for fire takes into account the reduction in strength and stiffness properties at elevated temperatures. The modification factor for fire replaces the modification factor for normal temperature design  $k_{mod}$  given in EN 1995-1-1. Values of  $k_{mod,fi}$  are given in the relevant clauses.

NOTE 2: The recommended partial safety factor for material properties in fire is  $\gamma_{M,fi} = 1,0$ . Information on National choice may be found in the National annex..

(2)P The design value  $R_{d,t,fi}$  of a mechanical resistance (load-bearing capacity) shall be calculated as

$$R_{d,t,fi} = \eta \frac{R_{20}}{\gamma_{M,fi}} \quad (2.3)$$

where:

- $R_{d,t,fi}$  is the design value of a mechanical resistance in the fire situation at time  $t$ ;
- $R_{20}$  is the 20 % fractile value of a mechanical resistance at normal temperature without the effect of load duration and moisture ( $k_{mod} = 1$ );

$\eta$  is a conversion factor;

$\gamma_{M,fi}$  is the partial safety factor for timber in fire.

Note 1: See (1) above Note 2.

Note 2: Design resistances are applied for connections, see 6.2.2 and 6.4. For connections a conversion factor  $\eta$  is given in 6.2.2.1.

(3) The 20 % fractile of a strength or a stiffness property should be calculated as:

$$f_{20} = k_{fi} f_k \quad (2.4)$$

$$S_{20} = k_{fi} S_{05} \quad (2.5)$$

where:

$f_{20}$  is the 20 % fractile of a strength property at normal temperature;

$S_{20}$  is the 20 % fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature;

$S_{05}$  is the 5 % fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature

$k_{fi}$  is given in table 2.1.

**Table 2.1 — Values of  $k_{fi}$**

	$k_{fi}$
Solid timber	1,25
Glued-laminated timber	1,15
Wood-based panels	1,15
LVL	1,1
Connections with fasteners in shear with side members of wood and wood-based panels	1,15
Connections with fasteners in shear with side members of steel	1,05
Connections with axially loaded fasteners	1,05

(4) The 20 % fractile of a mechanical resistance,  $R_{20}$ , of a connection should be calculated as

$$R_{20} = k_{fi} R_k \quad (2.6)$$

where:

$k_{fi}$  is given in table 2.1.

$R_k$  is the characteristic mechanical resistance of a connection at normal temperature without the effect of load duration and moisture ( $k_{mod} = 1$ ).

(5) For design values of temperature-dependent thermal properties, see 3.2.

## **2.4 Verification methods**

### **2.4.1 General**

(1)P The model of the structural system adopted for design shall reflect the performance of the structure in the fire situation.

(2)P It shall be verified for the required duration of fire exposure  $t$ :

$$E_{d,fi} \leq R_{d,t,fi} \quad (2.7)$$

where

$E_{d,fi}$  is the design effect of actions for the fire situation, determined in accordance with EN 1991-1-2:2002, including effects of thermal expansions and deformations;

$R_{d,t,fi}$  is the corresponding design resistance in the fire situation.

(3) The structural analysis for the fire situation should be carried out in accordance with EN 1990:2002 subclause 5.1.4.

NOTE: For verifying standard fire resistance requirements, a member analysis is sufficient.

(4)P The effect of thermal expansions of materials other than timber shall be taken into account.

(5) Where application rules given in EN 1995-1-2 are valid only for the standard temperature-time curve, this is identified in the relevant clauses.

(6) As an alternative to design by calculation, fire design may be based on the results of fire tests, or on fire tests in combination with calculations, see EN 1990:2002 clause 5.2.

#### 2.4.2 Member analysis

(1) The effect of actions should be determined for time  $t = 0$  using combination factors  $\psi_{1,1}$  or  $\psi_{2,1}$  according to EN 1991-1-2:2002 clause 4.3.1.

(2) As a simplification to (1), the effect of actions  $E_{d,fi}$  may be obtained from the analysis for normal temperature as:

$$E_{d,fi} = \eta_{fi} E_d \quad (2.8)$$

where:

$E_d$  is the design effect of actions for normal temperature design for the fundamental combination of actions, see EN 1990:2002;

$\eta_{fi}$  is the reduction factor for the design load in the fire situation.

(3) The reduction factor  $\eta_{fi}$  for load combination (6.10) in EN 1990:2002 should be taken as

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \quad (2.9)$$

or, for load combinations (6.10a) and (6.10b) in EN 1990:2002, as the smallest value given by the following two expressions

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \quad (2.9a)$$

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\xi \gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \quad (2.9b)$$

where:

$Q_{k,1}$  is the characteristic value of the leading variable action;

$G_k$  is the characteristic value of the permanent action;

$\gamma_G$  is the partial factor for permanent actions;

$\gamma_{Q,1}$  is the partial factor for variable action 1;

$\psi_{fi}$  is the combination factor for frequent values of variable actions in the fire situation, given either by  $\psi_{1,1}$  or  $\psi_{2,1}$ , see EN 1991-1-1;  $\psi_{fi}$

$\xi$  is a reduction factor for unfavourable permanent actions  $G$ .

NOTE 1: An example of the variation of the reduction factor  $\eta_{fi}$  versus the load ratio  $Q_{k,1}/G_k$  for different values of the combination factor  $\psi_{fi}$  according to expression (2.9) is shown in figure 2.1 with the following assumptions:  $\gamma_{GA} = 1,0$ ,  $\gamma_G = 1,35$  and  $\gamma_Q = 1,5$ . Partial factors are specified in the relevant National annexes of EN 1990:2002. Expressions (2.9a) and (2.9b) give slightly higher values.

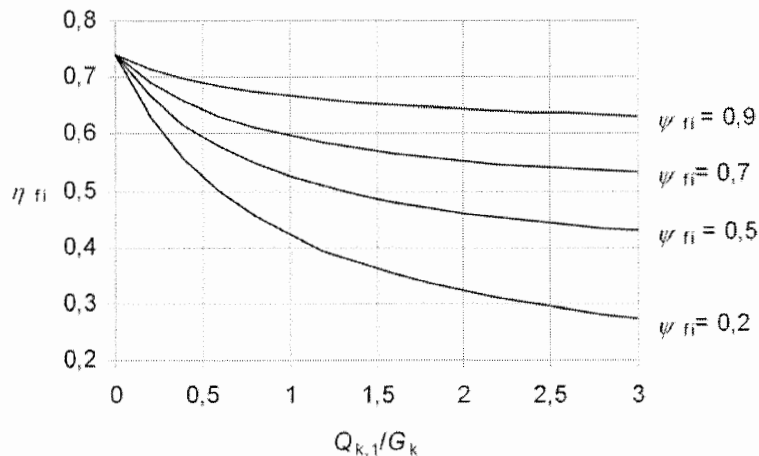


Figure 2.1 – Examples of reduction factor  $\eta_{fi}$  versus load ratio  $Q_{k,1}/G_k$  according to expression (2.9)

NOTE 2: As a simplification, the recommended value is  $\eta_{fi} = 0,6$ , except for imposed loads according to category E given in EN 1991-2-1:2002 (areas susceptible to accumulation of goods, including access areas) where the recommended value is  $\eta_{fi} = 0,7$ . Information on National choice may be found in the National annex.

NOTE 3: The National choice of load combinations between expression (2.9) and expressions (2.9a) and (2.9b) is made in EN 1991-1-2:2002.

(4) The boundary conditions at supports may be assumed to be constant with time.

### 2.4.3 Analysis of parts of the structure

(1) 2.4.2(1) applies.

(2) As an alternative to carrying out a structural analysis for the fire situation at time  $t = 0$ , the reactions at supports and internal forces and moments at boundaries of part of the structure may be obtained from structural analysis for normal temperature as given in 2.4.2.

(3) The part of the structure to be analysed should be specified on the basis of the potential thermal expansions and deformations such that their interaction with other parts of the structure can be approximated by time-independent support and boundary conditions during fire exposure.

(4)P Within the part of the structure to be analysed, the relevant failure mode in fire, the temperature-dependent material properties and member stiffnesses, effects of thermal expansions and deformations (indirect fire actions) shall be taken into account.

(5) The boundary conditions at supports and the forces and moments at boundaries of the part of the structure being considered may be assumed to be constant with time.

#### **2.4.4 Global structural analysis**

- (1)P A global structural analysis for the fire situation shall take into account:
- the relevant failure mode in fire exposure;
  - the temperature-dependent material properties and member stiffnesses;
  - effects of thermal expansions and deformations (indirect fire actions).

## **Section 3 Material properties**

### **3.1 General**

(1)P Unless given as design values, the values of material properties given in this section shall be treated as characteristic values.

(2)P The mechanical properties of timber at 20 °C shall be taken as those given in EN 1995-1-1 for normal temperature design.

### **3.2 Mechanical properties**

(1) Simplified methods for the reduction of the strength and stiffness parameters of the cross-section are given in 4.1 and 4.2.

NOTE 1: A simplified method for the reduction of the strength and stiffness parameters of timber frame members in wall and floor assemblies completely filled with insulation is given in annex C (informative).

NOTE 2: A simplified method for the reduction of the strength of timber members exposed to parametric fires is given in annex A (informative).

(2) For advanced calculation methods, a non-linear relationship between strain and compressive stress may be applied.

NOTE: Values of temperature-dependent mechanical properties are given in annex B (informative).

### **3.3 Thermal properties**

(1) Where fire design is based on a combination of tests and calculations, where possible, the thermal properties should be calibrated to the test results.

NOTE: For thermal analysis, design values of thermal conductivity and heat capacity of timber are given in annex B (informative).

### **3.4 Charring depth**

#### **3.4.1 General**

(1)P Charring shall be taken into account for all surfaces of wood and wood-based panels directly exposed to fire, and, where relevant, for surfaces initially protected from exposure to fire, but where charring of the wood occurs during the relevant time of fire exposure.

(2) The charring depth is the distance between the outer surface of the original member and the position of the char-line and should be calculated from the time of fire exposure and the relevant charring rate.

(3) The calculation of cross-sectional properties should be based on the actual charring depth including corner roundings. Alternatively a notional cross-section without corner roundings may be calculated based on the notional charring rate.

(4) The position of the char-line should be taken as the position of the 300-degree isotherm.

NOTE: This assumption is valid for most softwoods and hardwoods.

(5) It should be taken into account that the charring rates are normally different for

- surfaces unprotected throughout the time of fire exposure;
- initially protected surfaces prior to failure of the protection;

- initially protected surfaces when exposed to fire after failure of the protection.

(6) The rules of 3.4.2 and 3.4.3 apply to standard fire exposure.

NOTE: For parametric fire exposure, see annex A (informative).

### 3.4.2 Surfaces unprotected throughout the time of fire exposure

(1) The charring rate for one-dimensional charring, see figure 3.1, should be taken as constant with time. The design charring depth should be calculated as:

$$d_{\text{char},0} = \beta_0 t \quad (3.1)$$

where:

$d_{\text{char},0}$  is the design charring depth for one-dimensional charring;

$\beta_0$  is the one-dimensional design charring rate under standard fire exposure;

$t$  is the time of fire exposure.

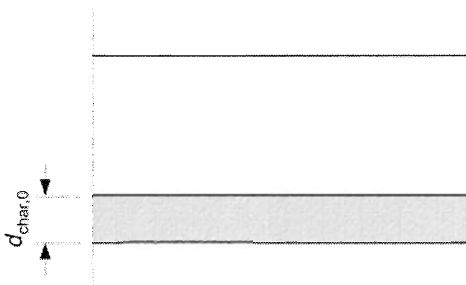


Figure 3.1 — One-dimensional charring of wide cross section (fire exposure on one side)

(2) The notional charring rate, the magnitude of which includes for the effect of corner roundings and fissures, see figure 3.2, should be taken as constant with time. The notional design charring depth should be calculated as

$$d_{\text{char},n} = \beta_n t \quad (3.2)$$

where:

$d_{\text{char},n}$  is the notional design charring depth, which incorporates the effect of corner roundings;

$\beta_n$  is the notional design charring rate, the magnitude of which includes for the effect of corner roundings and fissures.

(3) The one-dimensional design charring rate may be applied, provided that the increased charring near corners is taken into account, for cross-sections with an original minimum width,  $b_{\text{min}}$ , where

$$b_{\text{min}} = \begin{cases} 2 d_{\text{char},0} + 80 & \text{for } d_{\text{char},0} \geq 13 \text{ mm} \\ 8,15 d_{\text{char},0} & \text{for } d_{\text{char},0} < 13 \text{ mm} \end{cases} \quad (3.3)$$

When the smallest width of the cross section is smaller than  $b_{\text{min}}$ , notional design charring rates should be applied.

(4) For cross-sections calculated using one-dimensional design charring rates, the radius of the corner roundings should be taken equal to the charring depth  $d_{\text{char},0}$ .

Ⓐ (5) For surfaces of timber and wood-based materials, unprotected throughout the time of fire exposure, design charring rates  $\beta_0$  and  $\beta_n$  are given in table 3.1. Ⓐ

NOTE: For timber members in wall and floor assemblies where the cavities are completely filled with insulation, values for notional design charring rates  $\beta_n$  are given in annex C (informative).

(6) Design charring rates for solid hardwoods, except beech, with characteristic densities between 290 and 450 kg/m<sup>3</sup>, may be obtained by linear interpolation between the values of table 3.1. Charring rates of beech should be taken as given for solid softwood.

(7) Design charring rates for LVL, in accordance with EN 14374, are given in table 3.1.

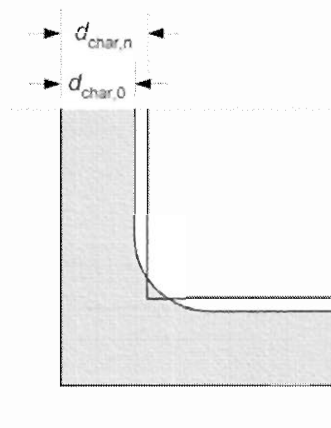


Figure 3.2 — Charring depth  $d_{char,0}$  for one-dimensional charring and notional charring depth  $d_{char,n}$

(8) Design charring rates for wood-based panels in accordance with EN 309, EN 313-1, EN 300 and EN 316, and wood panelling are given in Table 3.1. The values apply to a characteristic density of 450 kg/m<sup>3</sup> and a panel thickness of 20 mm.

(9) For other characteristic densities  $\rho_k$  and panel thicknesses  $h_p$  smaller than 20 mm, the charring rate should be calculated as

$$\beta_{0,\rho,t} = \beta_0 k_\rho k_h \quad (3.4)$$

with

$$k_\rho = \sqrt{\frac{450}{\rho_k}} \quad (3.5)$$

$$k_h = \sqrt{\frac{20}{h_p}} \quad (3.6)$$

where:

$\rho_k$  is the characteristic density, in kg/m<sup>3</sup>;

$h_p$  is the panel thickness, in millimetres.

NOTE: For wood-based panels characteristic densities are given in EN 12369.

**Table 3.1 – Design charring rates  $\beta_0$  and  $\beta_n$  of timber, LVL, wood panelling and wood-based panels**

	$\beta_0$ mm/min	$\beta_n$ mm/min
<b>a) Softwood and beech</b> Glued laminated timber with a characteristic density of $\geq 290 \text{ kg/m}^3$ Solid timber with a characteristic density of $\geq 290 \text{ kg/m}^3$	0,65 0,65	0,7 0,8
<b>b) Hardwood</b> Solid or glued laminated hardwood with a characteristic density of $\geq 290 \text{ kg/m}^3$ Solid or glued laminated hardwood with a characteristic density of $\geq 450 \text{ kg/m}^3$	0,65 0,50	0,7 0,55
<b>c) LVL</b> with a characteristic density of $\geq 480 \text{ kg/m}^3$	0,65	0,7
<b>d) Panels</b> Wood panelling Plywood Wood-based panels other than plywood	0,9 <sup>a</sup> 1,0 <sup>a</sup> 0,9 <sup>a</sup>	– – –
<sup>a</sup> The values apply to a characteristic density of $450 \text{ kg/m}^3$ and a panel thickness of 20 mm; see 3.4.2(9) for other thicknesses and densities.		

### 3.4.3 Surfaces of beams and columns initially protected from fire exposure

#### 3.4.3.1 General

(1) For surfaces protected by fire protective claddings, other protection materials or by other structural members, see figure 3.3, it should be taken into account that

- the start of charring is delayed until time  $t_{ch}$ ;
- charring may commence prior to failure of the fire protection, but at a lower rate than the charring rates shown in table 3.1 until failure time  $t_f$  of the fire protection;
- after failure time  $t_f$  of the fire protection, the charring rate is increased above the values shown in table 3.1 until the time  $t_a$  described below;
- at the time  $t_a$  when the charring depth equals either the charring depth of the same member without fire protection or 25 mm whichever is the lesser, the charring rate reverts to the value in table 3.1.

NOTE 1: Other fire protection available includes intumescent coatings and impregnation. Test methods are given in ENV 13381–7

NOTE 2: The protection provided by other structural members may be terminated due to

- failure or collapse of the protecting member;
- excessive deformation of the protecting member.

NOTE 3: The different stages of protection, the times of transition between stages and corresponding charring rates are illustrated in figures 3.4 to 3.6.

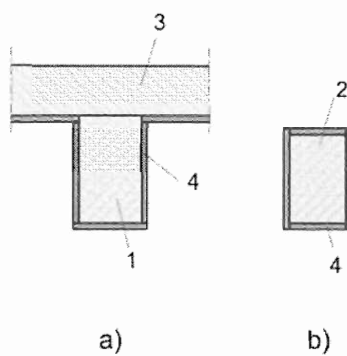
NOTE 4: Rules for assemblies with void cavities are given in annex D (informative).

(2) Unless rules are given below, the following should be assessed on the basis of tests:

- the time to the start of charring  $t_{ch}$  of the member;
- the time for failure of the fire protective cladding or other fire protection material  $t_f$ ;
- the charring rate before failure of the protection when  $t_f > t_{ch}$ .

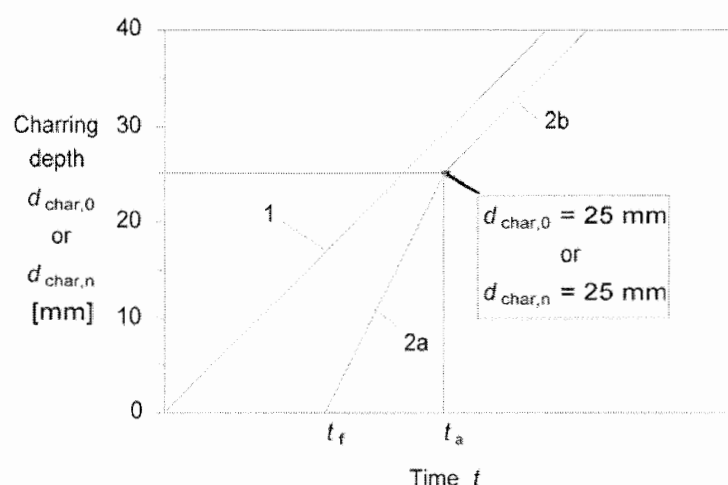
NOTE: Test methods are given in ENV 13381-7.

(3) The effect of unfilled gaps greater than 2 mm at joints in the cladding on the start of charring and, where relevant, on the charring rate before failure of the protection should be taken into account.



Key:  
1 beam  
2 column  
3 deck  
4 cladding

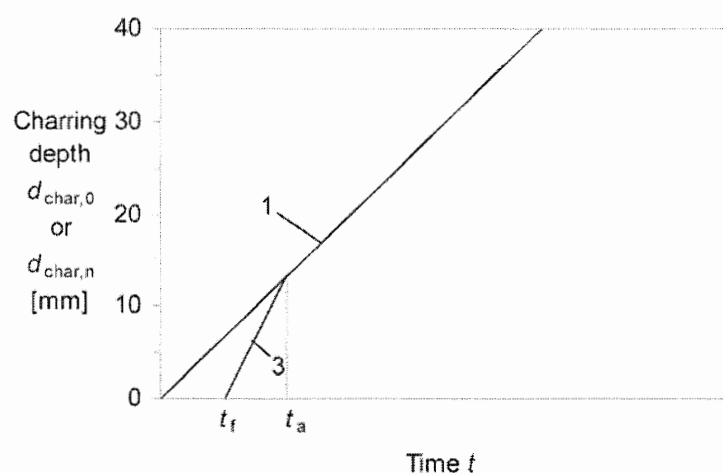
**Figure 3.3 — Examples of fire protective claddings to: a) beams, b) columns,**



Key:

- 1 Relationship for members unprotected throughout the time of fire exposure for charring rate  $\beta_h$  (or  $\beta_0$ )
- 2 Relationship for initially protected members after failure of the fire protection
- 2a After the fire protection has fallen off, charring starts at increased rate
- 2b After char depth exceeds 25 mm charring rate reduces to the rate shown in table 3.1

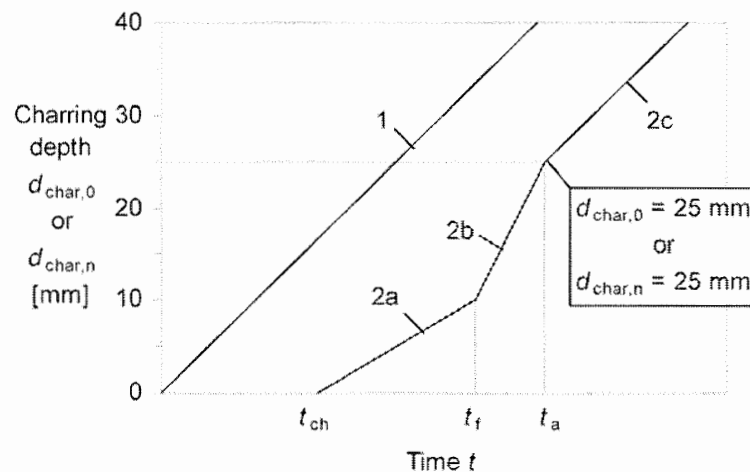
Figure 3.4 — Variation of charring depth with time when  $t_{ch} = t_f$  and the charring depth at time  $t_a$  is at least 25 mm



Key:

- 1 Relationship for members unprotected throughout the time of fire exposure for charring rate shown in table 3.1
- 3 Relationship for initially protected members with failure times of fire protection  $t_f$  and time limit  $t_a$  smaller than given by expression (3.8b)

Figure 3.5 — Variation of charring depth with time when  $t_{ch} = t_f$  and the charring depth at time  $t_a$  is less than 25 mm



Key:

- 1 Relationship for members unprotected throughout the time of fire exposure for charring rate  $\beta_h$  (or  $\beta_0$ )
- 2 Relationship for initially protected members where charring starts before failure of protection:
  - 2a Charring starts at  $t_{ch}$  at a reduced rate when protection is still in place
  - 2b After protection has fallen off, charring starts at increased rate
  - 2c After char depth exceeds 25 mm charring rate reduces to the rate shown in table 3.1

Figure 3.6 — Variation of charring depth with time when  $t_{ch} < t_f$

### 3.4.3.2 Charring rates

(1) For  $t_{ch} \leq t \leq t_f$  the charring rates of the timber member given in table 3.1 should be multiplied by a factor  $k_2$ .

(2) Where the timber member is protected by a single layer of gypsum plasterboard type F,  $k_2$  should be taken as

$$k_2 = 1 - 0,018 h_p \quad (3.7)$$

where  $h_p$  is the thickness of the layer, in millimetres.

Where the cladding consists of several layers of gypsum plasterboard type F,  $h_p$  should be taken as the thickness of the inner layer.

(3) Where the timber member is protected by rock fibre batts with a minimum thickness of 20 mm and a minimum density of  $26 \text{ kg/m}^3$  which remain coherent up to  $1000^\circ\text{C}$ ,  $k_2$  may be taken from table 3.2. For thicknesses between 20 and 45 mm, linear interpolation may be applied

Table 3.2 – Values of  $k_2$  for timber protected by rock fibre batts

Thickness $h_{ins}$ mm	$k_2$
20	1
$\geq 45$	0,6

(4) For the stage after failure of the protection given by  $t_f \leq t \leq t_a$ , the charring rates of table 3.1 should be multiplied by a factor  $k_3 = 2$ . For  $t \geq t_a$  the charring rates of table 3.1 should be applied without multiplication by  $k_3$ .

(5) The time limit  $t_a$ , see figure 3.4 and 3.5, should for  $t_{ch} = t_f$  be taken as

$$t_a = \min \begin{cases} 2 t_f & (a) \\ \frac{25}{k_3 \beta_n} + t_f & (b) \end{cases} \quad (3.8)$$

or for  $t_{ch} < t_f$  (see figure 3.6)

$$t_a = \frac{25 - (t_f - t_{ch}) k_2 \beta_n}{k_3 \beta_n} + t_f \quad (3.9)$$

where  $\beta_n$  is the notional design charring rate, in mm/min. Expressions (3.8) and (3.9) also apply to one-dimensional charring when  $\beta_n$  is replaced by  $\beta_0$ .

For the calculation of  $t_f$  see 3.4.3.4.

NOTE: Expression (3.8b) implies that a char-layer of 25 mm gives sufficient protection to reduce the charring rate to the values of table 3.1.

### 3.4.3.3 Start of charring

(1) For fire protective claddings consisting of one or several layers of wood-based panels or wood panelling, the time of start of charring  $t_{ch}$  of the protected timber member should be taken as

$$t_{ch} = \frac{h_p}{\beta_0} \quad (3.10)$$

where:

$h_p$  is the thickness of the panel, in case of several layers the total thickness of layers;

$t_{ch}$  is the time of start of charring;

(2) For claddings consisting of one layer of gypsum plasterboard of type A, F or H according to EN 520, at internal locations or at the perimeter adjacent to filled joints, or unfilled gaps with a width of 2 mm or less, the time of start of charring  $t_{ch}$  should be taken as

$$t_{ch} = 2,8 h_p - 14 \quad (3.11)$$

where:

$h_p$  is the thickness of the panel, in mm.

At locations adjacent to joints with unfilled gaps with a width of more than 2 mm, the time of start of charring  $t_{ch}$  should be calculated as

$$t_{ch} = 2,8 h_p - 23 \quad (3.12)$$

where:

$h_p$  is the thickness of the panel, in mm;

NOTE: Gypsum plasterboard type E, D, R and I according to EN 520 have equal or better thermal and mechanical properties than type A and H.

(3) For claddings consisting of two layers of gypsum plasterboard of type A or H, the time of start of charring  $t_{ch}$  should be determined according to expression (3.11) where the thickness  $h_p$

is taken as the thickness of the outer layer and 50 % of the thickness of the inner layer, provided that the spacing of fasteners in the inner layer is not greater than the spacing of fasteners in the outer layer.

(4) For claddings consisting of two layers of gypsum plasterboard of type F, the time of start of charring  $t_{ch}$  should be determined according to expression (3.11) where the thickness  $h_p$  is taken as the thickness of the outer layer and 80 % of the thickness of the inner layer, provided that the spacing of fasteners in the inner layer is not greater than the spacing of fasteners in the outer layer.

(5) For beams or columns protected by rock fibre batts as specified in 3.4.3.2(3), the time of start of charring  $t_{ch}$  should be taken as

$$t_{ch} = 0,07(h_{ins} - 20) \sqrt{\rho_{ins}} \quad (3.13)$$

where:

- $t_{ch}$  is the time of start of charring in minutes;
- $h_{ins}$  is the thickness of the insulation material in millimetres;
- $\rho_{ins}$  is the density of the insulating material in kg/m<sup>3</sup>.

#### **3.4.3.4 Failure times of fire protective claddings**

(1) Failure of fire protective claddings may occur due to

- charring or mechanical degradation of the material of the cladding;
- insufficient penetration length of fasteners into uncharred timber;
- inadequate spacing and distances of fasteners.

(2) For fire protective claddings of wood panelling and wood-based panels attached to beams or columns, the failure time should be determined according to the following:

$$t_f = t_{ch} \quad (3.14)$$

where  $t_{ch}$  is calculated according to expression (3.10).

(3) For gypsum plasterboard type A and H the failure time  $t_f$  should be taken as:

$$t_f = t_{ch} \quad (3.15)$$

where  $t_{ch}$  is calculated according to expression 3.4.3.3(3).

NOTE: In general, failure due to mechanical degradation is dependent on temperature and size of the panels and their orientation. Normally, vertical position is more favourable than horizontal.

(4) The penetration length  $l_a$  of fasteners into uncharred timber should be at least 10 mm. The required length of the fastener  $l_{f,req}$  should be calculated as

$$l_{f,req} = h_p + d_{char,0} + l_a \quad (3.16)$$

where:

- $h_p$  is the panel thickness;
- $d_{char,0}$  is the charring depth in the timber member;
- $l_a$  is the minimum penetration length of the fastener into uncharred timber.

Increased charring near corners should be taken into account, see 3.4.2(4).

### **3.5 Adhesives**

(1)P Adhesives for structural purposes shall produce joints of such strength and durability that the integrity of the bond is maintained in the assigned fire resistance period.

NOTE: For some adhesives, the softening temperature is considerably below the charring temperature of the wood.

(2) For bonding of wood to wood, wood to wood-based materials or wood-based materials to wood-based materials, adhesives of phenol-formaldehyde and aminoplastic type 1 adhesive according to EN 301 may be used. For plywood and LVL, adhesives according to EN 314 may be used.

## Section 4 Design procedures for mechanical resistance

### 4.1 General

(1) The rules of EN 1995-1-1 apply in conjunction with cross-sectional properties determined according to 4.2 and 4.3 and the additional rules for analysis given in 4.3. For advanced calculation methods, see 4.4.

### 4.2 Simplified rules for determining cross-sectional properties

#### 4.2.1 General

(1) The section properties should be determined by the rules given in either 4.2.2 or 4.2.3.

NOTE: The recommended procedure is the reduced cross-section method given in 4.2.2. Information on the National choice may be found in the National annex.

#### 4.2.2 Reduced cross-section method

(1) An effective cross-section should be calculated by reducing the initial cross-section by the effective charring depth  $d_{ef}$  (see figure 4.1)

$$d_{ef} = d_{char,n} + k_0 d_0 \quad (4.1)$$

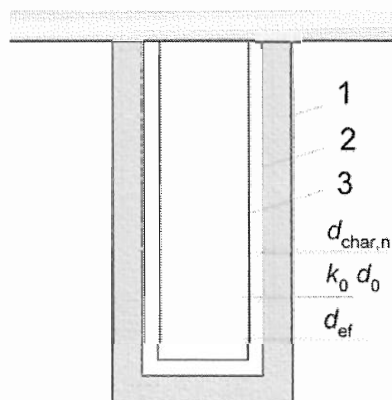
with:

$$d_0 = 7 \text{ mm}$$

$d_{char,n}$  is determined according to expression (3.2) or the rules given in 3.4.3.

$k_0$  is given in (2) and (3).

NOTE: It is assumed that material close to the char line in the layer of thickness  $k_0 d_0$  has zero strength and stiffness, while the strength and stiffness properties of the remaining cross-section are assumed to be unchanged.



Key

- 1 Initial surface of member
- 2 Border of residual cross-section
- 3 Border of effective cross-section

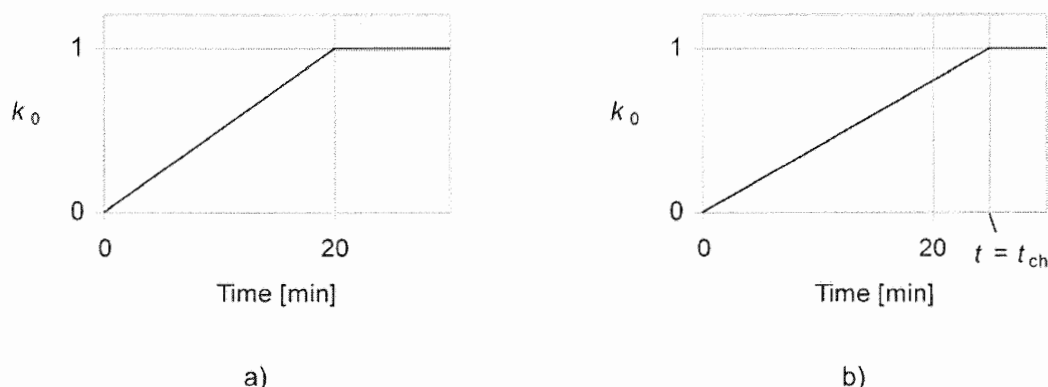
Figure 4.1 — Definition of residual cross-section and effective cross-section

(2) For unprotected surfaces,  $k_0$  should be determined from table 4.1.

**Table 4.1 — Determination of  $k_0$  for unprotected surfaces with  $t$  in minutes (see figure 4.2a)**

	$k_0$
$t < 20$ minutes	$t/20$
$t \geq 20$ minutes	1,0

(3) For protected surfaces with  $t_{ch} > 20$  minutes, it should be assumed that  $k_0$  varies linearly from 0 to 1 during the time interval from  $t = 0$  to  $t = t_{ch}$ , see figure 4.2b. For protected surfaces with  $t_{ch} \leq 20$  minutes table 4.1 applies.



**Figure 4.2 — Variation of  $k_0$ : a) for unprotected members and protected members where  $t_{ch} \leq 20$  minutes, b) for protected members where  $t_{ch} > 20$  minutes**

(4) For timber surfaces facing a void cavity in a floor or wall assembly (normally the wide sides of a stud or a joist), the following applies:

- Where the fire protective cladding consists of one or two layers of gypsum plasterboard type A, wood panelling or wood-based panels, at the time of failure  $t_f$  of the cladding,  $k_0$  should be taken as 0,3. Thereafter  $k_0$  should be assumed to increase linearly to unity during the following 15 minutes;
- Where the fire protective cladding consists of one or two layers of gypsum plasterboard type F, at the time of start of charring  $t_{ch}$ ,  $k_0$  should be taken as unity. For times  $t < t_{ch}$ , linear interpolation should be applied, see figure 4.2b.

(5) The design strength and stiffness properties of the effective cross-section should be calculated with  $k_{mod,fi} = 1,0$ .

### 4.2.3 Reduced properties method

(1) The following rules apply to rectangular cross-sections of softwood exposed to fire on three or four sides and round cross-sections exposed along their whole perimeter.

(2) The residual cross-section should be determined according to 3.4.

(3) For  $t \geq 20$  minutes, the modification factor for fire  $k_{mod,fi}$ , see 2.3 (1)P, should be taken as follows (see figure 4.3):

- for bending strength:

$$k_{mod,fi} = 1,0 - \frac{1}{200} \frac{p}{A_r} \quad (4.2)$$

- for compressive strength:

$$k_{\text{mod,fi}} = 1,0 - \frac{1}{125} \frac{p}{A_r} \quad (4.3)$$

– for tensile strength and modulus of elasticity:

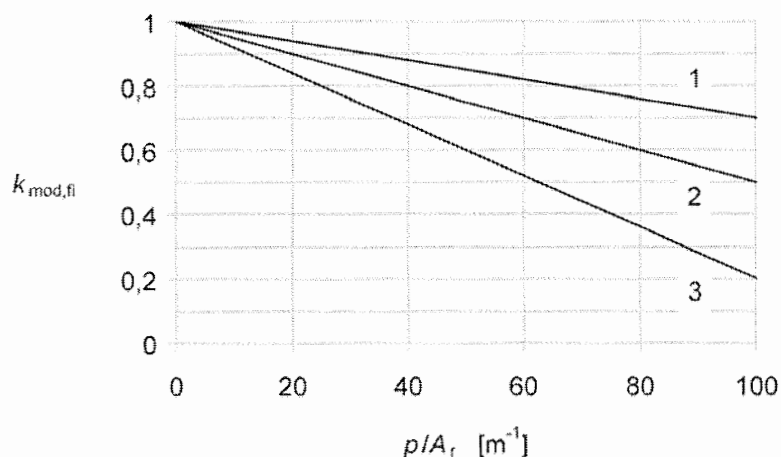
$$k_{\text{mod,fi}} = 1,0 - \frac{1}{330} \frac{p}{A_r} \quad (4.4)$$

where:

$p$  is the perimeter of the fire exposed residual cross-section, in metres;

$A_r$  is the area of the residual cross-section, in  $\text{m}^2$ .

(4) For unprotected and protected members, for time  $t = 0$  the modification factor for fire should be taken as  $k_{\text{mod,fi}} = 1$ . For unprotected members, for  $0 \leq t \leq 20$  minutes the modification factor may be determined by linear interpolation.



Key:

- 1 Tensile strength, Modulus of elasticity
- 2 Bending strength
- 3 Compressive strength

Figure 4.3 — Illustration of expressions (4.2)-(4.4)

### 4.3 Simplified rules for analysis of structural members and components

#### 4.3.1 General

- (1) Compression perpendicular to the grain may be disregarded.
- (2) Shear may be disregarded in rectangular and circular cross-sections. For notched beams it should be verified that the residual cross-section in the vicinity of the notch is at least 60 % of the cross-section required for normal temperature design.

#### 4.3.2 Beams

- (1) Where bracing fails during the relevant fire exposure, the lateral torsional stability of the beam should be considered without any lateral restraint from that bracing.

### 4.3.3 Columns

(1) Where bracing fails during the relevant fire exposure, the stability of the column should be considered without any lateral restraint from that bracing.

(2) More favourable boundary conditions than for normal temperature design may be assumed for a column in a fire compartment which is part of a continuous column in a non-sway frame. In intermediate storeys the column may be assumed as fixed at both ends, whilst in the top storey the column may be assumed as fixed at its lower end, see figure 4.4. The column length  $L$  should be taken as shown in figure 4.4.

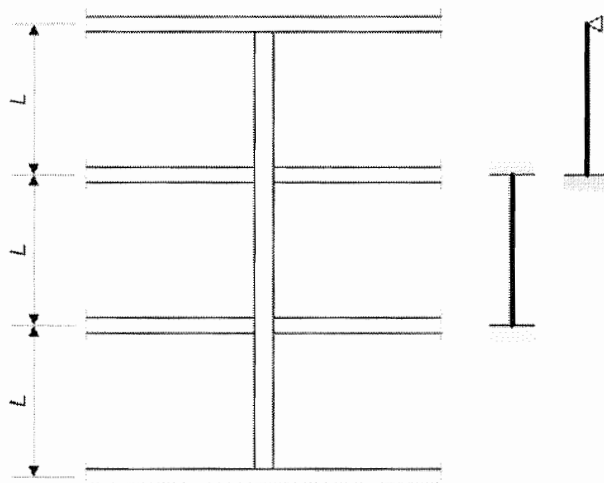


Figure 4.4 — Continuous column

### 4.3.4 Mechanically jointed members

(1)P For mechanically jointed members, the reduction in slip moduli in the fire situation shall be taken into account.

(2) The slip modulus  $K_{fi}$  for the fire situation should be determined as

$$K_{fi} = K_u \eta_f \quad (4.5)$$

where:

$K_{fi}$  is the slip modulus in the fire situation, in N/mm;

$K_u$  is the slip modulus at normal temperature for the ultimate limit state according to EN 1995-1-1 2.2.2(2), in N/mm;

$\eta_f$  is a conversion factor according to table 4.2.

Table 4.2 — Conversion factor  $\eta_f$

Nails and screws	0,2
Bolts; dowels; split ring, shear plate and toothed-plate connectors	0,67

#### **4.3.5 Bracings**

(1) Where members in compression or bending are designed taking into account the effect of bracing, it should be verified that the bracing does not fail during the required duration of the fire exposure.

(2) Bracing members made of timber or wood-based panels may be assumed not to fail if the residual thickness or cross-sectional area is 60 % of its initial value required for normal temperature design, and is fixed with nails, screws, dowels or bolts.

#### **4.4 Advanced calculation methods**

(1)P Advanced calculation methods for determination of the mechanical resistance and the separating function shall provide a realistic analysis of structures exposed to fire. They shall be based on fundamental physical behaviour in such a way as to lead to a reliable approximation of the expected behaviour of the relevant structural component under fire conditions.

NOTE: Guidance is given in annex B (informative).

## Section 5 Design procedures for wall and floor assemblies

### 5.1 General

(1) The rules in this subclause apply to load-bearing (R) constructions, separating (EI) constructions, and load-bearing and separating (REI) constructions. For the separating function the rules only apply for standard fire resistances not exceeding 60 minutes.

### 5.2 Analysis of load-bearing function

AC2 (1)P Non-separating load-bearing constructions shall be designed for fire exposure on both sides at the same time. AC2

NOTE 1: For wall and floor assemblies with cavities completely filled with insulation a design method is given in annex C (informative).

NOTE 2: For wall and floor assemblies with void cavities, design rules are given in annex D (informative).

### 5.3 Analysis of separating function

(1) The analysis should take into account the contributions of different material components and their position in the assembly.

NOTE: A design method is given in annex E (informative).

## Section 6 Connections

### 6.1 General

(1) This section applies to connections between members under standard fire exposure, and unless stated otherwise, for fire resistances not exceeding 60 minutes. Rules are given for connections made with nails, bolts, dowels, screws, split-ring connectors, shear-plate connectors and toothed-plate connectors.

(2) The rules of 6.2 and 6.3 apply to laterally loaded symmetrical three-member connections. Clause 6.4 deals with axially loaded screws.

### 6.2 Connections with side members of wood

#### 6.2.1 Simplified rules

##### 6.2.1.1 Unprotected connections

(1) The fire resistance of unprotected wood-to-wood connections where spacings, edge and end distances and side member dimensions comply with the minimum requirements given in EN 1995-1-1 section 8, may be taken from table 6.1.

**Table 6.1 —Fire resistances of unprotected connections with side members of wood**

	Time of fire resistance $t_{d,fi}$ min	Provisions <sup>a</sup>
Nails	15	$d \geq 2,8$ mm
Screws	15	$d \geq 3,5$ mm
Bolts	15	$t_1 \geq 45$ mm
Dowels	20	$t_1 \geq 45$ mm
Connectors according to EN 912	15	$t_1 \geq 45$ mm
<sup>a</sup> $d$ is the diameter of the fastener and $t_1$ is the thickness of the side member		

(2) For connections with dowels, nails or screws with non-projecting heads, fire resistance periods  $t_{d,fi}$  greater than those given in table 6.1, but not exceeding 30 minutes, may be achieved by increasing the following dimensions by  $a_{fi}$ :

- the thickness of side members,
- the width of the side members,
- the end and edge distance to fasteners.

where:

$$a_{fi} = \beta_n k_{flux} (t_{req} - t_{d,fi}) \quad (6.1)$$

$\beta_n$  is the charring rate according to table 3.1;

$k_{flux}$  is a coefficient taking into account increased heat flux through the fastener;

$t_{req}$  is the required standard fire resistance period;

$t_{d,fi}$  is the fire resistance period of the unprotected connection given in table 6.1.

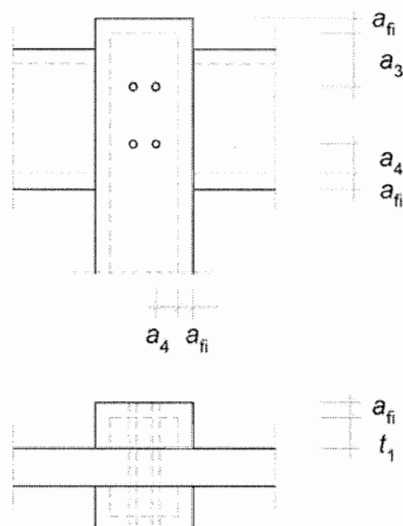


Figure 6.1 — Extra thickness and extra end and edge distances of connections

(3) The factor  $k_{flux}$  should be taken as  $k_{flux} = 1,5$ .

#### 6.2.1.2 Protected connections

(1) When the connection is protected by the addition of wood panelling, wood-based panels or gypsum plasterboard type A or H, the time until start of charring should satisfy

$$t_{ch} \geq t_{req} - 0,5 t_{d,fi} \quad (6.2)$$

where:

$t_{ch}$  is the time until start of charring according to 3.4.3.3;

$t_{req}$  is the required standard fire resistance period;

$t_{d,fi}$  is the fire resistance of the unprotected connection given in table 6.1.

(2) When the connection is protected by the addition of gypsum plasterboard type F, the time until start of charring should satisfy

$$t_{ch} \geq t_{req} - 1,2 t_{d,fi} \quad (6.3)$$

(3) For connections where the fasteners are protected by glued-in timber plugs, the length of the plugs should be determined according to expression (6.1), see figure 6.2.

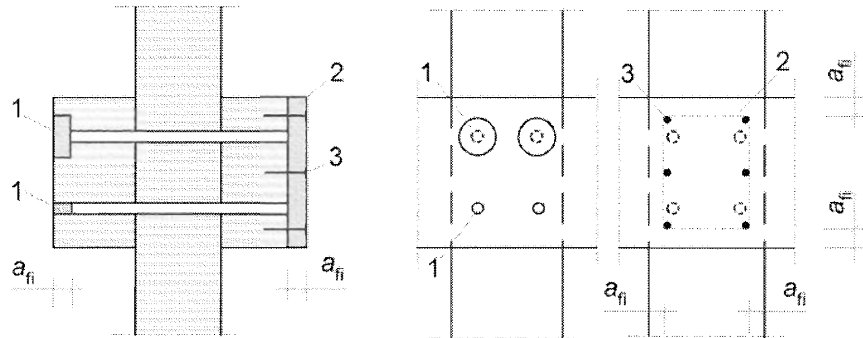
(4) The fixings of the additional protection should prevent its premature failure. Additional protection provided by wood-based panels or gypsum plasterboard should remain in place until charring of the member starts ( $t = t_{ch}$ ). Additional protection provided by gypsum plasterboard type F should remain in place during the required fire resistance period ( $t = t_{req}$ ).

(5) In bolted connections the bolt heads should be protected by a protection of thickness  $a_{fi}$ , see figure 6.3.

(6) The following rules apply for the fixing of additional protection by nails or screws:

- the distance between fasteners should be not more than 100 mm along the board edges and not more than 300 mm for internal fastenings;
- the edge distance of fasteners should be equal or greater than  $a_{fi}$  calculated using expression (6.1), see figure 6.2.

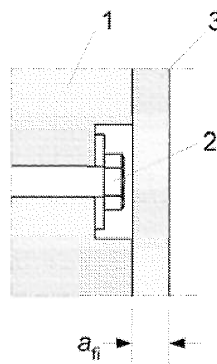
(7) The penetration depth of fasteners fixing of the additional protection made of wood, wood-based panels or gypsum plasterboard type A or H should be at least  $6d$  where  $d$  is the diameter of the fastener. For gypsum plasterboard type F, the penetration length into unburnt wood (that is beyond the char-line) should be at least 10 mm, see figure 7.1b.



Key:

- 1 Glued-in plugs
- 2 Additional protection using panels
- 3 Fastener fixing panels providing additional protection

**Figure 6.2 — Examples of additional protection from glued-in plugs or from wood-based panels or gypsum plasterboard (the protection of edges of side and middle members is not shown)**



Key:

- 1 Member
- 2 Bolt head
- 3 Member providing protection

**Figure 6.3 — Example of protection to a bolt head**

### 6.2.1.3 Additional rules for connections with internal steel plates

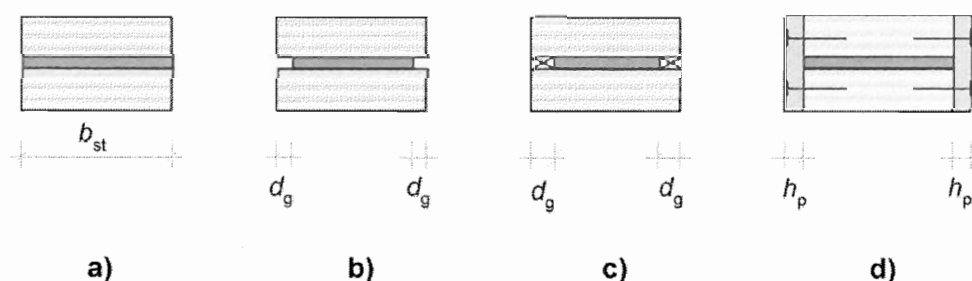
(1) For joints with internal steel plates of a thickness equal or greater than 2 mm, and which do not project beyond the timber surface, the width  $b_{st}$  of the steel plates should observe the conditions given in table 6.2.

**Table 6.2 — Widths of steel plates with unprotected edges**

		$b_{st}$
Unprotected edges in general	R 30	$\geq 200$ mm
	R 60	$\geq 280$ mm
Unprotected edges on one or two sides	R 30	$\geq 120$ mm
	R 60	$\geq 280$ mm

(2) Steel plates narrower than the timber member may be considered as protected in the following cases (see figure 6.4):

- For plates with a thickness of not greater than 3 mm where the gap depth  $d_g$  is greater than 20 mm for a fire resistance period of 30 minutes and greater than 60 mm for a fire resistance period of 60 minutes;
- For joints with glued-in strips or protective wood-based boards where the depth of the glued-in strip,  $d_g$ , or the panel thickness,  $h_p$ , is greater than 10 mm for a fire resistance period of 30 minutes and greater than 30 mm for a fire resistance period of 60 minutes.



**Figure 6.4 — Protection of edges of steel plates (fasteners not shown): a) unprotected, b) protected by gaps, c) protected by glued-in strips, d) protected by panels**

## 6.2.2 Reduced load method

### 6.2.2.1 Unprotected connections

**[AC2]** (1) The rules for bolts and dowels are valid where the thickness of the side plate is equal or greater than  $t_1$ , in mm: **[AC2]**

$$t_1 = \max \begin{cases} 50 \\ 50 + 1,25(d - 12) \end{cases} \quad (6.4)$$

where  $d$  is the diameter of bolt or dowel, in mm.

(2) For standard fire exposure, the characteristic load-carrying capacity of a connection with fasteners in shear should be calculated as

$$F_{v,Rk,fi} = \eta F_{v,Rk} \quad (6.5)$$

with

$$\eta = e^{-k t_{d,fi}} \quad (6.6)$$

where:

$F_{v,Rk}$  is the characteristic lateral load-carrying capacity of the connection with fasteners in shear at normal temperature, see EN 1995-1-1 section 8;

- $\eta$  is a conversion factor;  
 $k$  is a parameter given in table 6.3;  
 $t_{d,fi}$  is the design fire resistance of the unprotected connection, in minutes.

NOTE: The design load-bearing capacity is calculated corresponding to 2.3 (2)P.

**AC2** (3) The design fire resistance of the unprotected connection loaded by the design effect of actions in the fire situation, see 2.4.1, should be taken as:

$$t_{d,fi} = -\frac{1}{k} \ln \frac{\eta_{fi} \eta_0 k_{mod} \gamma_{M,fi}}{\gamma_M k_{fi}} \quad (6.7)$$

where:

- $k$  is a parameter given in table 6.3;  
 $\eta_{fi}$  is the reduction factor for the design load in the fire situation, see 2.4.2 (2);  
 $\eta_0$  is the degree of utilisation at normal temperature;  
 $k_{mod}$  is the modification factor from EN 1995-1-1, subclause 3.1.3;  
 $\gamma_M$  is the partial factor for the connection, see EN 1995-1-1, subclause 2.4.1;  
 $k_{fi}$  is a value according to 2.3 (4);  
 $\gamma_{M,fi}$  is the partial safety factor for timber in fire, see 2.3(1). **AC2**

**Table 6.3 — Parameter  $k$**

Connection with	$k$	Maximum period of validity for parameter $k$ in an unprotected connection min
Nails and screws	0,08	20
Bolts wood-to-wood with $d \geq 12$ mm	0,065	30
Bolts steel-to-wood with $d \geq 12$ mm	0,085	30
Dowels wood-to-wood <sup>a</sup> with $d \geq 12$ mm	0,04	40
Dowels steel-to-wood <sup>a</sup> with $d \geq 12$ mm	0,085	30
Connectors in accordance with EN 912	0,065	30
<sup>a</sup> The values for dowels are dependent on the presence of one bolt for every four dowels		

- (4) For dowels projecting more than 5 mm, values of  $k$  should be taken as for bolts.  
(5) For connections made of both bolts and dowels, the load-bearing capacity of the connection should be taken as the sum of the load-bearing capacities of the respective fasteners.  
(6) For connections with nails or screws with non-projecting heads, for fire resistances greater than given by expression (6.7) but not more than 30 minutes, the side member thickness and end and edge distances should be increased by  $a_{fi}$  (see figure 6.1) which should be taken as:

$$a_{fi} = \beta_n (t_{req} - t_{d,fi}) \quad (6.8)$$

where:

- $\beta_n$  is the notional charring rate according to table 3.1;  
 $t_{req}$  is the required standard fire resistance;

$t_{d,fi}$  is the fire resistance of the unprotected connection loaded by the design effect of actions in the fire situation, see 2.4.1.

### 6.2.2.2 Protected connections

(1) Subclause 6.2.1.2 applies, except that  $t_{d,fi}$  should be calculated according to expression (6.7).

(2) As an alternative method of protecting end and side surfaces of members, the end and edge distances may be increased by  $a_{fi}$  according to expression (6.1). For fire resistances greater than 30 minutes, however, the end distances should be increased by  $2a_{fi}$ . This increase in end distance also applies for butted central members in a connection.

## 6.3 Connections with external steel plates

### 6.3.1 Unprotected connections

(1) The load-bearing capacity of the external steel plates should be determined according to the rules given in EN 1993-1-2.

(2) For the calculation of the section factor of the steel plates according to EN 1993-1-2, it may be assumed that steel surfaces in close contact with wood are not exposed to fire.

### 6.3.2 Protected connections

(1) Steel plates used as side members may be considered as protected if they are totally covered, including at edges of plate, by timber or wood-based panels with a minimum thickness of  $a_{fi}$  according to expression (6.1) with  $t_{d,fi} = 5$  min.

(2) The effect of other fire protections should be calculated according to EN 1993-1-2.

## 6.4 Simplified rules for axially loaded screws

(1) For axially loaded screws that are protected from direct fire exposure, the following rules apply.

(2) The design resistance of the screws should be calculated according to expression (2.3).

(3) For connections where the distances  $a_2$  and  $a_3$  of the fastener satisfy expressions (6.9) and (6.10), see figure 6.5, the conversion factor  $\eta$  for the reduction in the axial resistance of the screw in the fire situation should be calculated using expression (6.11):

$$a_2 \geq a_1 + 40 \quad (6.9)$$

$$a_3 \geq a_1 + 20 \quad (6.10)$$

where  $a_1$ ,  $a_2$  and  $a_3$  are the distances, in millimetres.

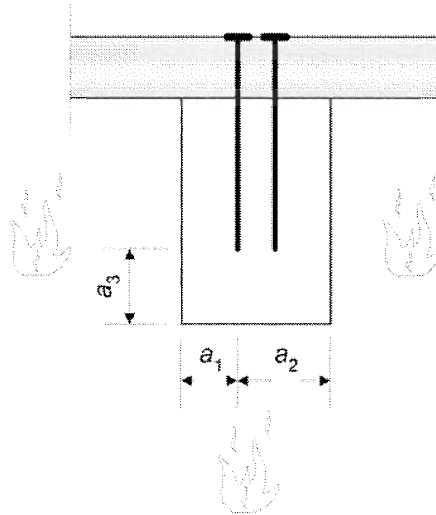
$$\eta = \begin{cases} 0 & \text{for } a_1 \leq 0,6 t_{d,fi} & (a) \\ \frac{0,44 a_1 - 0,264 t_{d,fi}}{0,2 t_{d,fi} + 5} & \text{for } 0,6 t_{d,fi} \leq a_1 \leq 0,8 t_{d,fi} + 5 & (b) \\ \frac{0,56 a_1 - 0,36 \cdot t_{d,fi} + 7,32}{0,2 t_{d,fi} + 23} & \text{for } 0,8 t_{d,fi} + 5 \leq a_1 \leq t_{d,fi} + 28 & (c) \\ 1,0 & \text{for } a_1 \geq t_{d,fi} + 28 & (d) \end{cases} \quad (6.11)$$

where:

$a_1$  is the side cover in mm, see figure 6.5;

$t_{d,fi}$  is the required fire resistance period, in minutes.

(4) The conversion factor  $\eta$  for fasteners with edge distances  $a_2 = a_1$  and  $a_3 \geq a_1 + 20$  mm should be calculated according to expression (6.11) where  $t_{d,fi}$  is replaced by  $1,25 t_{d,fi}$ .



**Figure 6.5 — Cross-section and definition of distances**

## Section 7 Detailing

### 7.1 Walls and floors

#### 7.1.1 Dimensions and spacings

(1) The spacing of wall studs and floor joists should not be greater than 625 mm.

(2) For walls, individual panels should have a minimum thickness of

$$t_{p,min} = \max \left\{ \begin{array}{l} l_p \\ 70 \\ 8 \end{array} \right. \quad (7.1)$$

where:

$t_{p,min}$  is the minimum thickness of panel in millimetres;

$l_p$  is the span of the panel (spacing between timber frame members or battens) in millimetres.

(3) Wood-based panels in constructions with a single layer on each side should have a characteristic density of at least 350 kg/m<sup>3</sup>.

#### 7.1.2 Detailing of panel connections

(1) Panels should be fixed to the timber frame or battens.

(2) For wood-based panels and wood panelling, the maximum spacing of nails and screws around the perimeter should be 150 mm and 250 mm respectively. The minimum penetration length should be eight times the fastener diameter for load-bearing panels and six times the fastener diameter for non-load-bearing panels.

(3) For gypsum plasterboard of types A and H, it is sufficient to observe the rules for normal temperature design with respect to penetration length, spacings and edge distances. For screws, however, the perimeter and internal spacing should not be greater than 200 mm and 300 mm respectively.

(4) For gypsum plasterboard type F panels, the penetration length  $l_a$  of fasteners into the residual cross-section should not be less than 10 mm, see figure 7.1.

(5) Panel edges should be tightly jointed with a maximum gap of 1 mm. They should be fixed to the timber member or battens on at least two opposite edges.

(6) For multiple layers the panel joints should be staggered by at least 60 mm. Each panel should be fixed individually.

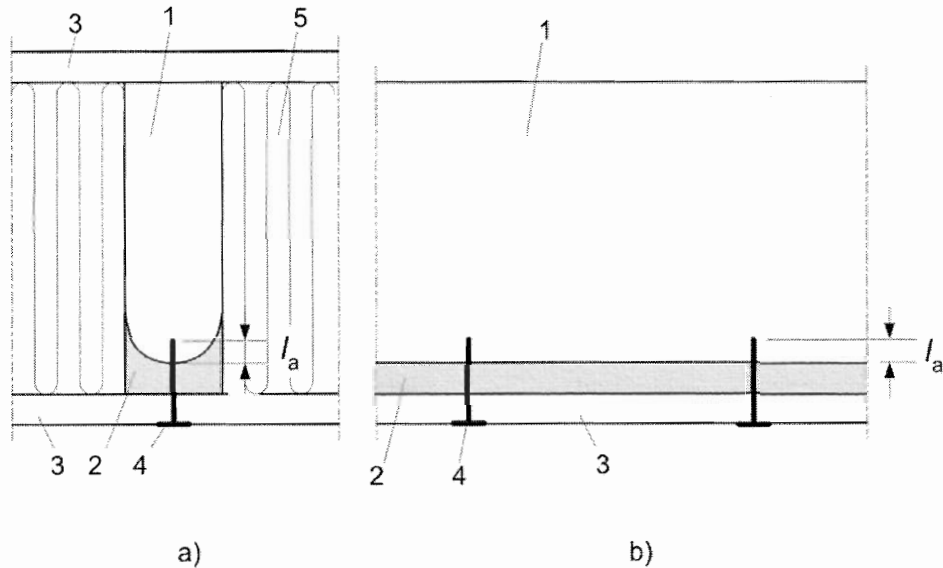
#### 7.1.3 Insulation

(1) Insulating layers or boards that are taken into account in the calculation should be tightly fitted and fixed to the timber frame such that premature failure or slumping is prevented.

### 7.2 Other elements

(1) Fire protective wood-based panels or wood panelling protecting members such as beams and columns should be fixed by nails or screws to the member according to figure 7.2. Panels should be fixed to the member itself and not to another panel. For claddings consisting of multiple layers of panels each layer should be fixed individually, and joints should be staggered by at least 60 mm. The spacing of fasteners should not be greater than 200 mm or 17 times the

panel thickness  $h_p$ , whichever is the smallest. With reference to fastener length, 7.1.2(1)-(2) applies, see figure 7.1 b. The edge distance should not be greater than 3 times the panel thickness  $h_p$  and not be smaller than 1,5 times the panel thickness or 15 mm, whichever is the smallest.



Key:

- 1 Unburnt timber
- 2 Char layer
- 3 Panel
- 4 Fastener
- 5 Insulation

Figure 7.1 — Timber members protected by gypsum plasterboard — Examples of penetration length of fastener into unburnt timber: a) Timber frame assembly with insulation in cavity, b) Wide timber member in general

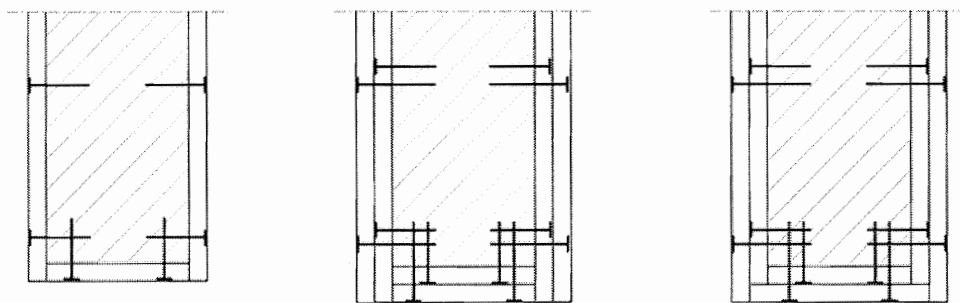


Figure 7.2 — Examples of fixing of fire protective panels to beams or columns

## Annex A (Informative) Parametric fire exposure

### A1 General

(1) This Annex deals with natural fire exposure according to the opening factor method using parametric time-temperature curves.

NOTE: A method for the determination of parametric time-temperature curves is given in EN 1991-1-2:2002, annex A.

### A2 Charring rates and charring depths

(1) For unprotected softwood the relation between the charring rate  $\beta$  and time  $t$  shown in figure A1 should be used. The charring rate  $\beta_{\text{par}}$  during the heating phase of a parametric fire curve is given by

$$\beta_{\text{par}} = 1,5 \beta_n \frac{0,2\sqrt{\Gamma} - 0,04}{0,16\sqrt{\Gamma} + 0,08} \quad (\text{A.1})$$

with

$$\Gamma = \frac{\left(\frac{O}{b}\right)^2}{\left(\frac{0,04}{1160}\right)^2} \quad (\text{A.2})$$

$$O = \frac{A_v}{A_t} \sqrt{h_{\text{eq}}} \quad (\text{A.3})$$

$$b = \sqrt{\rho c \lambda} \quad (\text{A.4})$$

$$h_{\text{eq}} = \sum \frac{A_i h_i}{A} \quad (\text{A.5})$$

where:

- $O$  is the opening factor, in  $\text{m}^{0.5}$ ;
- $\beta_n$  is the notional design charring rate, in  $\text{mm/min}$ ;
- $A_v$  is the total area of openings in vertical boundaries of the compartment (windows etc.), in  $\text{m}^2$ ;
- $A_t$  is the total area of floors, walls and ceilings that enclose the fire compartment, in  $\text{m}^2$ ;
- $A_i$  is the area of vertical opening "i", in  $\text{m}^2$ ;
- $h_{\text{eq}}$  is the weighted average of heights of all vertical openings (windows etc.), in metres;
- $h_i$  is the height of vertical opening "i", in metres;
- $\Gamma$  is a factor accounting for the thermal properties of the boundaries of the compartment;
- $b$  is the absorptivity for the total enclosure, see EN 1991-1-2:2002, annex A;
- $\lambda$  is the thermal conductivity of the boundary of the compartment, in  $\text{Wm}^{-1}\text{K}^{-1}$ ;
- $\rho$  is the density of the boundary of the compartment, in  $\text{kg/m}^3$ ;
- $c$  is the specific heat of the boundary of the compartment, in  $\text{Jkg}^{-1}\text{K}^{-1}$ .

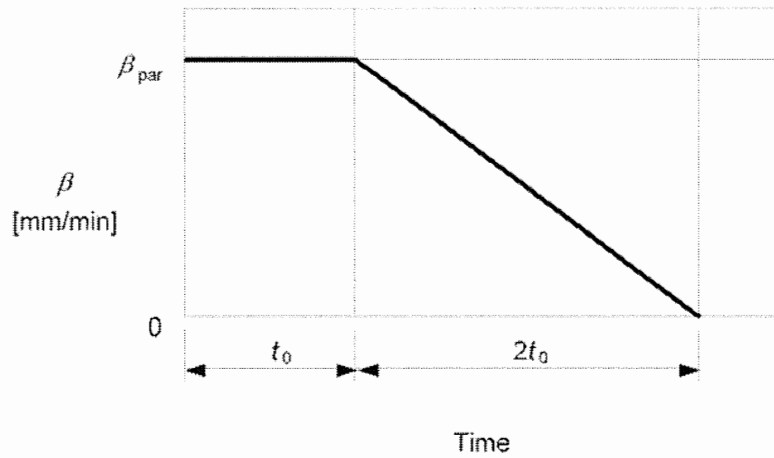


Figure A1 — Relationship between charring rate and time

(2) The charring depth should be taken as

$$\boxed{\text{AC2}} d_{\text{char}} = \begin{cases} \beta_{\text{par}} t & \text{for } t \leq t_0 & \text{(a)} \\ \beta_{\text{par}} \left( 1,5t - \frac{t^2}{4t_0} - \frac{t_0}{4} \right) & \text{for } t_0 < t \leq 3t_0 & \text{(b)} \\ 2\beta_{\text{par}} t_0 & \text{for } 3t_0 < t \leq 5t_0 & \text{(c)} \end{cases} \quad (\text{A.6}) \quad \boxed{\text{AC2}}$$

with

$$t_0 = 0,009 \frac{q_{t,d}}{O} \quad (\text{A.7})$$

where:

$t_0$  is the time period with a constant charring rate, in minutes;

$q_{t,d}$  is the design fire load density related to the total area of floors, walls and ceilings which enclose the fire compartment in  $\text{MJ/m}^2$ , see EN 1991-1-2:2002.

The rules given in (1) and (2) should only be used for:

$$- t_0 \leq 40 \text{ min}$$

$$- d_{\text{char}} \leq \frac{b}{4}$$

$$- d_{\text{char}} \leq \frac{h}{4}$$

where:

$b$  is the width of the cross-section;

$h$  is the depth of the cross-section.

### A3 Mechanical resistance of members in edgewise bending

(1) For members under edgewise bending with an initial width  $b \geq 130$  mm exposed to fire on three sides the mechanical resistance during the complete fire duration may be calculated using the residual cross-section. The residual cross-section of the member should be calculated by reducing the initial cross-section by the charring depth according to expression (A.6).

(2) For softwoods the modification factor for fire  $k_{\text{mod,fi}}$  should be calculated according to the following:

- for  $t \leq 3t_0$  the modification factor for fire should be calculated according to expression (4.2)
- for  $t = 5t_0$  as

$$k_{\text{mod,fi}} = 1,0 - 3,2 \frac{d_{\text{char,n}}}{b} \quad (\text{A.8})$$

where:

$d_{\text{char,n}}$  is the notional charring depth;

$b$  is the width of the member.

For  $3t_0 \leq t \leq 5t_0$  the modification factor for fire may be determined by linear interpolation.

NOTE: Where the reduced properties method given in 4.2.3 is invalidated by the National annex, for  $t \leq 3t_0$  the modification factor for fire can be derived from the reduced cross-section method as

$$k_{\text{mod,fi}} = \frac{W_{\text{ef}}}{W_{\text{r}}} \quad (\text{A.9})$$

where:

$W_{\text{ef}}$  is the section modulus of the effective cross-section determined according to 4.2.2;

$W_{\text{r}}$  is the section modulus of the residual cross-section.

## Annex B (informative) Advanced calculation methods

### B1 General

(1) Advanced calculation models may be used for individual members, parts of a structure or for entire structures.

(2) Advanced calculation methods may be applied for:

- the determination of the charring depth;
- the development and distribution of the temperature within structural members (thermal response model);
- the evaluation of structural behaviour of the structure or of any part of it (structural response model).

(3) The ambient temperature should be taken as 20°C.

(4) Advanced calculation methods for thermal response should be based on the theory of heat transfer.

(5) The thermal response model should take into account the variation of the thermal properties of the material with temperature.

NOTE: Where thermal models do not take into account phenomena such as increased heat transfer due to mass transport, e.g. due to the vaporisation of moisture, or increased heat transfer due to cracking which causes heat transfer by convection and/or radiation, the thermal properties are often modified in order to give results that can be verified by tests.

(6) The influence of any moisture content in the timber and of protection from gypsum plasterboard should be taken into account.

(7) Advanced calculation methods for the structural response should take into account the changes of mechanical properties with temperature and also, where relevant, with moisture.

(8) The effects of transient thermal creep should be taken into account. For timber and wood-based materials, special attention should be drawn to transient states of moisture.

NOTE: The mechanical properties of timber given in annex B include the effects of thermal creep and transient states of moisture.

(9) For materials other than timber or wood-based materials, the effects of thermally induced strains and stresses due to both temperature rise and temperature gradients, should be taken into account.

(10) The structural response model should take into account the effects of non-linear material properties.

### B2 Thermal properties

**AC<sub>2</sub>** For standard fire exposure, values of thermal conductivity, specific heat and the ratio of density to dry density of softwood may be taken as given in figures B1 to B3 and tables B1 and B2. **AC<sub>2</sub>**

NOTE 1: The thermal conductivity values of the char layer are apparent values rather than measured values of charcoal, in order to take into account increased heat transfer due to shrinkage cracks above about 500°C and the consumption of the char layer at about 1000°C. Cracks in the charcoal increase heat transfer due to radiation and convection. Commonly available computer models do not take into account these effects.

NOTE 2: Depending on the model used for calculation, modification of thermal properties given may be

necessary.

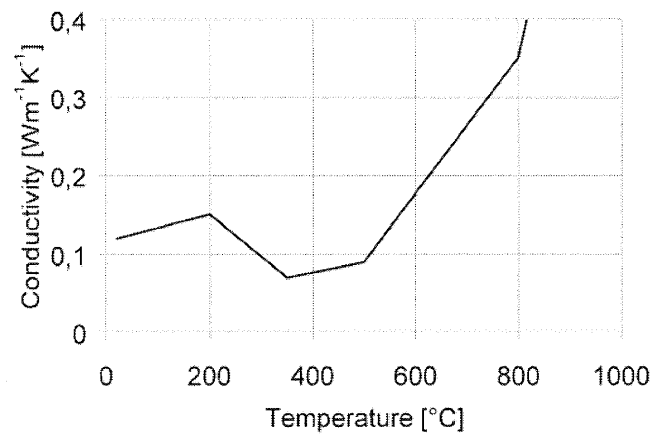


Figure B1 – Temperature-thermal conductivity relationship for wood and the char layer

Table B1 – Temperature-thermal conductivity relationship for wood and the char layer

Temperature °C	Thermal conductivity Wm <sup>-1</sup> K <sup>-1</sup>
20	0,12
200	0,15
350	0,07
500	0,09
800	0,35
1200	1,50

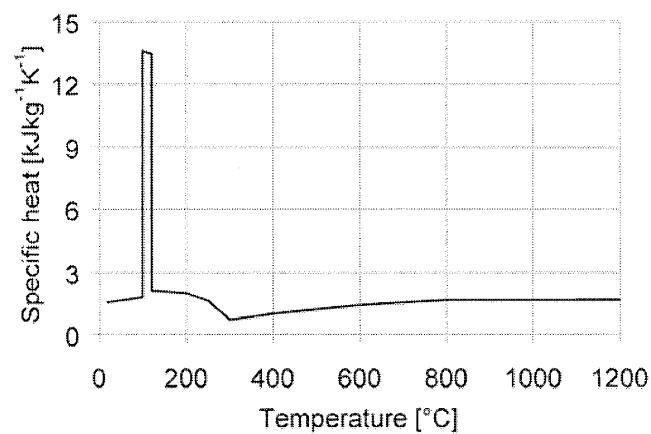


Figure B2 – Temperature-specific heat relationship for wood and charcoal

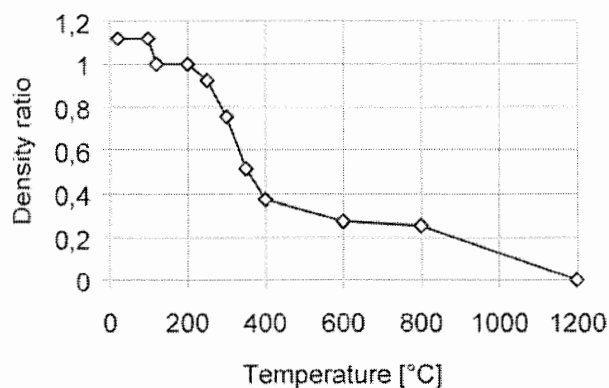


Figure B3 – Temperature-density ratio relationship for softwood with an initial moisture content of 12 %

AC2 Table B2 – Specific heat capacity and ratio of density to dry density of softwood for service class 1

Temperature °C	Specific heat capacity kJ kg <sup>-1</sup> K <sup>-1</sup>	Ratio of density to dry density <sup>a</sup>
20	1,53	1 + $\omega$
99	1,77	1 + $\omega$
99	13,60	1 + $\omega$
120	13,50	1,00
120	2,12	1,00
200	2,00	1,00
250	1,62	0,93
300	0,71	0,76
350	0,85	0,52
400	1,00	0,38
600	1,40	0,28
800	1,65	0,26
1200	1,65	0

<sup>a</sup>  $\omega$  is the moisture content

AC2

### B3 Mechanical properties

(1) The local values of strength and modulus of elasticity for softwood should be multiplied by a temperature dependent reduction factor according to figures B4 and B5.

NOTE: The relationships include the effects of transient creep of timber.

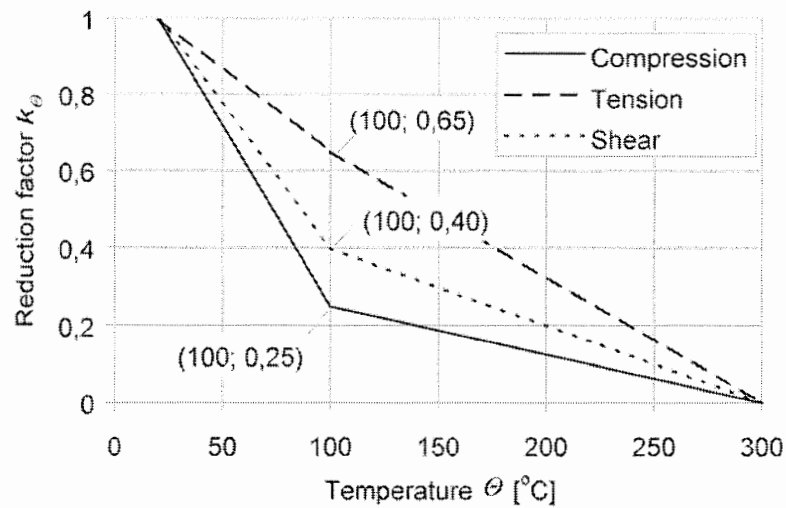


Figure B4 – Reduction factor for strength parallel to grain of softwood

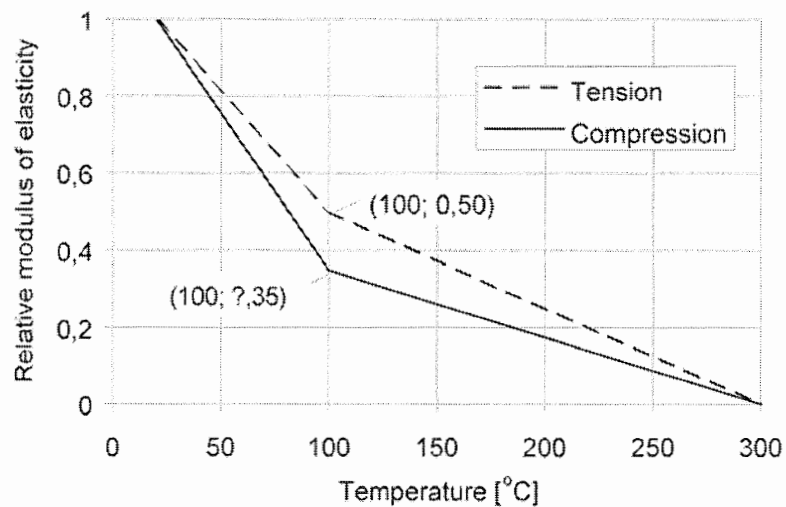


Figure B5 – Effect of temperature on modulus of elasticity parallel to grain of softwood

(2) For compression perpendicular to grain, the same reduction of strength may be applied as for compression parallel to grain.

(3) For shear with both stress components perpendicular to grain (rolling shear), the same reduction of strength may be applied as for compression parallel to grain.

## Annex C (Informative) Load-bearing floor joists and wall studs in assemblies whose cavities are completely filled with insulation

### C1 General

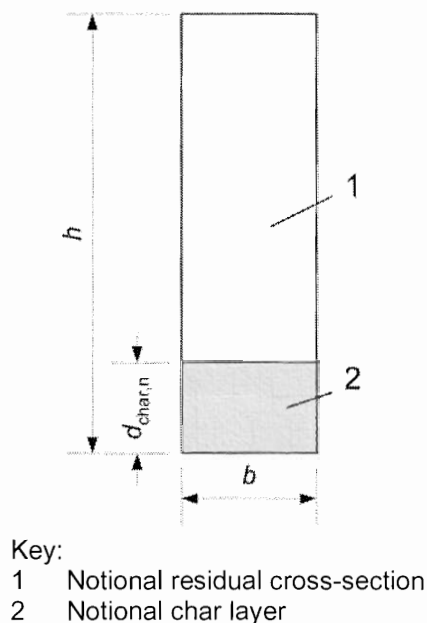
(1) This annex deals with the load-bearing function of timber frame wall and floor assemblies consisting of timber members (studs or joists) clad with panels on the fire-exposed side for a standard fire exposure of not more than 60 minutes. The following conditions apply:

- the cavities are completely filled with insulation made of rock or glass fibre;
- studs are braced against buckling in the plane of the wall and joists against torsional buckling by means of panels on the unexposed side or by noggins;
- for floors, the panels may also be fixed to steel channels with a maximum depth of 25 mm which are perpendicular to the direction of the timber joists;
- the separating function is verified according to 5.3.

### C2 Residual cross-section

#### C2.1 Charring rates

(1) The notional residual cross-section should be determined according to figure C1 where the notional charring depth is given by expression (3.2) and the notional charring rate is determined according to expressions (C.1) or (C.2).



**Figure C1 — Notional residual cross-section of timber frame member protected by cavity insulation**

(2) For timber members protected by claddings on the fire-exposed side, the notional charring rate should be calculated as:

$$\beta_n = k_s k_2 k_n \beta_0 \quad \text{for } t_{ch} \leq t \leq t_f \quad (\text{C.1})$$

$$\beta_n = k_s k_3 k_n \beta_0 \quad \text{for } t \geq t_f \quad (\text{C.2})$$

where:

$$k_n = 1,5$$

- $\beta_n$  is the notional design charring rate;  
 $k_s$  is the cross-section factor, see (3);  
 $k_2$  is the insulation factor, see (4);  
 $k_3$  is the post-protection factor, see (5);  
 $k_n$  is a factor to convert the irregular residual cross-section into a notional rectangular cross-section;  
 $\beta_0$  is the one-dimensional design charring rate, see 3.4.2 table 3.1;  
 $t$  is the time of fire exposure;  
 $t_{ch}$  is the time of start of charring of the timber frame member, see C2.2;  
 $t_f$  is the failure time of the cladding, see C2.3.

(3) The cross-section factor should be taken from table C1.

**Table C1 — Cross-section factor for different widths of timber frame member**

b mm	$k_s$
38	1,4
45	1,3
60	1,1

(4) For claddings made of gypsum plasterboard of type F, or a combination of type F and type A with type F as the outer layer, the insulation factor may be determined as:

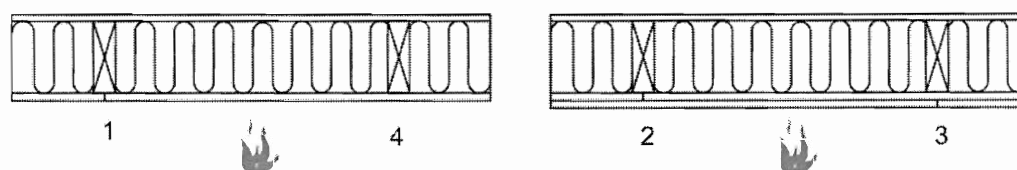
- at locations where the cladding is unjointed, or for joint configuration 2, see figure C2:

$$k_2 = 1,05 - 0,0073 h_p \quad (C.3)$$

- for joint configurations 1 and 3, see figure C2:

$$k_2 = 0,86 - 0,0037 h_p \quad (C.4)$$

where  $h_p$  is the total thickness of all panel layers in millimetres.



Key

- 1: Joint in single layer
- 2: Joint in inner board layer
- 3: Joint in outer board layer
- 4: Unjointed single layer

**Figure C2 — Joint configurations in gypsum plasterboard panels with one and two layers**

(5) Provided that the cavity insulation is made of rock fibre batts and remains in place after failure of the lining, the post-protection factor  $k_3$  should be calculated as

$$k_3 = 0,036 t_f + 1 \quad (\text{C.5})$$

where  $t_f$  is the failure time of the lining, in minutes.

(6) Where the cavity insulation is made of glass fibre, failure of the member should be assumed to take place at the time  $t_f$ .

## **C2.2 Start of charring**

(1) For fire protective claddings made of wood-based panels, the time of start of charring  $t_{ch}$  of the timber member should be taken as:

$$t_{ch} = t_f \quad (\text{C.6})$$

where the failure time  $t_f$  is calculated according to C2.3(1).

(2) Where the fire protective claddings are made of gypsum plasterboard of type A, H or F, the time of start of charring on the narrow fire-exposed side of the timber member should be determined according to 3.4.3.3(2), expressions (3.11) or (3.12).

## **C2.3 Failure times of panels**

(1) The failure time of claddings made of wood-based panels should be taken as:

$$t_f = \frac{h_p}{\beta_0} - 4 \quad (\text{C.7})$$

where:

$t_f$  is the failure time, in minutes;

$h_p$  is the panel thickness, in millimetres;

$\beta_0$  is the design charring rate for one-dimensional charring under standard fire exposure, in mm/min.

(2) The failure time of claddings made of gypsum plasterboard type A or H should be taken as:

$$t_f = 2,8 h_p - 14 \quad (\text{C.8})$$

(3) For claddings made of gypsum plasterboard type F, failure times should be determined with respect to:

- thermal degradation of the cladding;
- pull-out failure of fasteners due to insufficient penetration length into unburnt wood.

(4) The failure time due to the thermal degradation of the cladding should be assessed on the basis of tests.

NOTE: More information on test methods is given in EN 1363-1, EN 1365-1 and EN 1365-2.

(5) The failure time  $t_f$  of panels with respect to pull-out failure of fasteners may be calculated as

$$t_f = t_{ch} + \frac{l_f - l_{a,min} - h_p}{k_s k_2 k_n k_j \beta_0} \quad (\text{C.9})$$

with

$$k_j = 1,0 \quad \text{for panels not jointed over the timber member} \quad (\text{C.10})$$

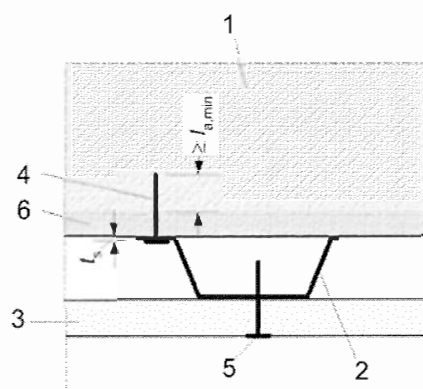
$$k_j = 1,15 \quad \text{for joint configurations 1 and 3} \quad (\text{C.11})$$

where:

- $t_{ch}$  is the time of start of charring;
- $l_f$  is the length of the fastener;
- $l_{a,min}$  is the minimum penetration length of the fastener into unburnt wood;
- $h_p$  is the total thickness of the panels;
- $k_s$  is the cross-section factor, see C2.1(3);
- $k_2$  is the insulation factor, see C2.1(4);
- $k_n$  is a factor to convert the irregular residual cross-section into a notional rectangular cross-section, see C2.1(2);
- $\beta_0$  is the design charring rate for one-dimensional charring under standard fire exposure, see 3.4.2 table 3.1.

The minimum penetration length  $l_{a,min}$  into unburnt wood should be taken as 10 mm.

(6) Where panels are fixed to steel channels, see figure C3, the failure time of the steel channels may be calculated according to expression (C.9) where  $h_p$  is replaced by the thickness  $t_s$  of the steel channel and  $k_j = 1,0$ .



Key:

- 1 Timber member
- 2 Steel channel
- 3 Panel
- 4 Fastener for fixing of steel channel to timber member
- 5 Fastener for fixing of panel to steel channel
- 6 Char layer

**Figure C3 — Illustration of use of steel channels for fixing panels in the ceiling**

(7) Where steel channels, after failure of the panels, are utilised to secure the insulation in the cavity, the failure time of the channels due to pull-out failure of the fastener may be calculated as:

$$t_{sf} = t_f + \frac{l_f - l_{a,min} - k_s k_2 k_n \beta_0 (t_f - t_{ch}) - t_s}{k_s k_3 k_n \beta_0} \quad (C.12)$$

where:

- $t_{sf}$  is the failure time of the steel channels;
- $t_s$  is the thickness of the steel channels;
- $k_3$  is the post-protection factor;

the other symbols are given in (5).

(8) For a fire resistance of  $\leq 60$  min, a verification of the load-bearing capacity and stiffness of the steel channels need not be performed.

### C3 Reduction of strength and stiffness parameters

(1) The modification factor for fire for strength of timber frame members should be calculated as

$$k_{\text{mod,fi}} = a_0 - a_1 \frac{d_{\text{char,n}}}{h} \quad (\text{C.13})$$



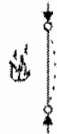
where:

$a_0, a_1$  are values given in table C2 and C3;

$d_{\text{char,n}}$  is the notional charring depth according to expression (3.2) with  $\beta_n$  according to expression (C.1) and (C.2);

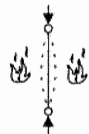
$h$  is the depth of the joist or the stud.

**Table C2 — Values<sup>a</sup> of  $a_0$  and  $a_1$  for reduction of strength of joists or studs in assemblies exposed to fire on one side**

Case			h mm	$a_0$	$a_1$
1	Bending strength with exposed side in tension		95	0,60	0,46
			145	0,68	0,49
			195	0,73	0,51
			220	0,76	0,51
2	Bending strength with exposed side in compression		95	0,46	0,37
			145	0,55	0,40
			195	0,65	0,48
			220	0,67	0,47
3	Compressive strength		95	0,46	0,37
			145	0,55	0,40
			195	0,65	0,48
			220	0,67	0,47

<sup>a</sup> For intermediate values of  $h$ , linear interpolation may be applied

**Table C3 — Values of  $a_0$  and  $a_1$  for reduction of compressive strength of studs in walls exposed to fire on both sides**

Case			h mm	$a_0$	$a_1$
1	Compressive strength		145	0,39	1,62

(2) The modification factor for modulus of elasticity should be calculated as

$$k_{\text{mod,E,fi}} = b_0 - b_1 \frac{d_{\text{char,n}}}{h} \quad (\text{C.14})$$

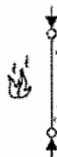
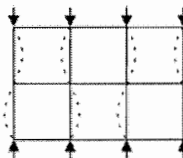
where:

$b_0, b_1$  are values given in tables C4 and C5;

$d_{\text{char,n}}$  is the notional charring depth according to expression (3.2) with  $\beta_n$  according to expression (C.1) and (C.2);


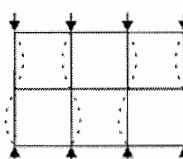
$h$  is the depth of the joist.

**Table C4 — Values<sup>a</sup> of  $b_0$  and  $b_1$  for reduction of modulus of elasticity of studs in walls exposed to fire on one side**

Case			$h$ mm	$b_0$	$b_1$
1	Buckling perpendicular to wall plane		95	0,50	0,79
			145	0,60	0,84
			195	0,68	0,77
2	Buckling in plane of wall		95	0,54	0,49
			145	0,66	0,55
			195	0,73	0,63

<sup>a</sup> For intermediate values of  $h$ , linear interpolation may be applied.  
NOTE: In the illustration to case 2 the studs are braced by noggins.

**Table C5 — Values<sup>a</sup> of  $b_0$  and  $b_1$  for reduction of modulus of elasticity of studs in walls exposed to fire on both sides**

Case			$h$ mm	$b_0$	$b_1$
1	Buckling perpendicular to wall plane		145	0,37	1,87
2	Buckling in plane of wall		145	0,44	2,18

<sup>a</sup> For intermediate values of  $h$ , linear interpolation may be applied.  
NOTE: In the illustration to case 2 the studs are braced by noggins.

## Annex D (informative) Charring of members in wall and floor assemblies with void cavities

### D1 General

- (1) The rules of this annex apply to standard fire exposure.
- (2) Clause 3.4.3.1 applies.

### D2 Charring rates

AC2 (1) 3.4.3.2(1), (2), (4) and (5) apply. AC2

### D3 Start of charring

- (1) For fire protective claddings made of wood-based panels or wood panelling the time of start until charring of timber members should be taken as:

$$t_{ch} = t_f \quad (D.1)$$

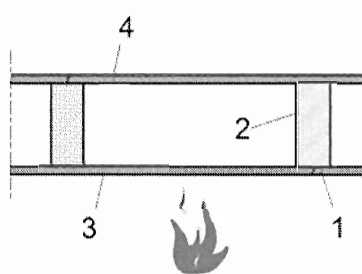
where  $t_f$  is determined according to D4(1).

- (2) For fire protective claddings made of gypsum plasterboard the time until start of charring  $t_{ch}$  of timber members should be determined according to the following:

- on the narrow side of the timber exposed to the fire, see figure D1, according to expression (3.11) or (3.12);
- on the wide sides of the timber member facing the void cavity, see figure D1, as:

$$t_{ch} = t_f \quad (D.2)$$

where the failure time  $t_f$  is determined according to D4(2). For definition of narrow and wide sides of timber member, see figure D1.



Key:

- 1 Narrow side of member exposed to fire
- 2 Wide side of member facing the cavity
- 3 Fire protective cladding on exposed side of assembly
- 4 Fire protective cladding on side of assembly not exposed to fire

Figure D1 — Definition of narrow and wide sides of timber member

### D4 Failure times of panels

- (1) For fire protective claddings of wood panelling and wood-based panels attached to the timber members, the failure time  $t_f$  should be taken as

$$t_f = \frac{h_p}{\beta_0} - 4 \quad (D.3)$$

where:

$t_f$  is the failure time, in minutes;

$h_p$  is the panel thickness, in millimetres;

$\beta_0$  is the one-dimensional charring rate, in mm/min.

(2) Failure times of gypsum plasterboard due to mechanical degradation of the material should be determined by testing. For type A and H gypsum plasterboard the failure time  $t_f$  may be taken as:

- for floors with the cladding fixed to timber members or resilient steel channels with a spacing of not more than 400 mm, and walls:

$$t_f = 2,8 h_p - 11 \quad (\text{D.4})$$

- for floors with the cladding fixed to timber members spaced more than 400 mm but not more than 600 mm:

$$t_f = 2,8 h_p - 12 \quad (\text{D.5})$$

where  $h_p$  is the thickness of the cladding, in mm. For claddings consisting of two layers, the thickness  $h_p$  should be taken as the thickness of the outer layer and 50 % of the thickness of the inner layer, provided that the spacing of fasteners in the inner layer is not greater than the spacing of fasteners in the outer layer.

## Annex E (informative) Analysis of the separating function of wall and floor assemblies

### E1 General

**AC2** (1) The fixing of the panel on the side of the assembly not exposed to fire should be secured into unburnt timber. **AC2**

(2) Requirements with respect to integrity (criterion E) are assumed to be satisfied where the requirements with respect to insulation (criterion I) have been satisfied and panels remain fixed to the timber frame on the unexposed side.

(3) The rules apply to timber frame members, claddings made of wood-based panels according to EN 13986 and gypsum plasterboard of type A, F and H according to EN 520. For other materials, integrity should be determined by testing.

NOTE: A test method is given in ENV 13381-7.

(4) For separating members it should be verified that

$$t_{\text{ins}} \geq t_{\text{req}} \quad (\text{E.1})$$

where:

$t_{\text{ins}}$  is the time taken for the temperature increases on the unexposed side given in 2.1.2(3) to occur;

$t_{\text{req}}$  is the required fire resistance period for the separating function of the assembly.

### E2 Simplified method for the analysis of insulation

#### E2.1 General

(1) The value of  $t_{\text{ins}}$  should be calculated as the sum of the contributions of the individual layers used in the construction, according to

$$t_{\text{ins}} = \sum_i t_{\text{ins},0,i} k_{\text{pos}} k_j \quad (\text{E.2})$$

where:

$t_{\text{ins},0,i}$  is the basic insulation value of layer "i" in minutes, see E2.2;

$k_{\text{pos}}$  is a position coefficient, see E2.3;

$k_j$  is a joint coefficient, see E2.4.

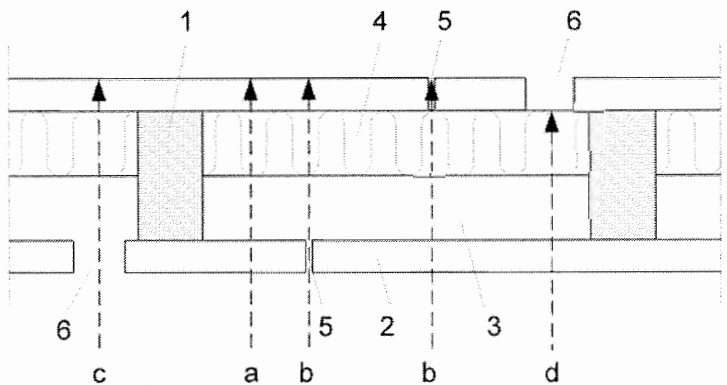
**AC2** The relevant number of layers should be determined from table E1 and figure E1. **AC2**

NOTE: A joint does not have an effect on the separating performance if it is backed with a batten or a structural element, which will prevent the travel of hot gases into the structure.

(2) Where a separating construction consists of only one layer, e.g. an uninsulated wall with a sheathing only on one side,  $t_{\text{ins}}$  should be taken as the basic insulation value of the sheathing and, if relevant, multiplied by  $k_j$ .

Table E1 — Heat transfer path through layer

	Temperature rise on unexposed side	Heat transfer path according to figure E1
	K	
General construction	140	a
Joints	180	b
Services	180	c, d



- Key:
- 1 timber frame member
  - 2 panel
  - 3 void cavity
  - 4 cavity insulation
  - 5 panel joint not being backed with a batten, stud or joist
  - 6 position of services
  - a – d heat transfer paths

Figure E1 — Illustration of heat transfer paths through a separating construction

E2.2 Basic insulation values

(1) The values given in this subclause may be applied for verification of fire resistance periods up to 60 minutes.

(2) Basic insulation values of panels should be determined from the following expressions:

– for plywood with a characteristic density of greater than or equal to 450 kg/m<sup>3</sup>

$$t_{ins,0} = 0,95 h_p \tag{E.3}$$

– for particleboard and fibreboard with a characteristic density greater than or equal to 600 kg/m<sup>3</sup>

$$t_{ins,0} = 1,1 h_p \tag{E.4}$$

– for wood panelling with a characteristic density greater than or equal to 400 kg/m<sup>3</sup>

$$t_{ins,0} = 0,5 h_p \tag{E.5}$$

– for gypsum plasterboard of type A, F, R and H

$$t_{\text{ins},0} = 1,4 h_p \quad (\text{E.6})$$

where:

$t_{\text{ins},0}$  is the basic insulation value, in minutes;

$h_p$  is the panel thickness, in millimetres.

(3) Where cavities are partially or completely filled with insulation made of glass or rock fibre, basic values of the insulation should be determined as:

– for rock fibre

$$t_{\text{ins},0,i} = 0,2 h_{\text{ins}} k_{\text{dens}} \quad (\text{E.7})$$

– for glass fibre

$$t_{\text{ins},0,i} = 0,1 h_{\text{ins}} k_{\text{dens}} \quad (\text{E.8})$$

where:

$h_{\text{ins}}$  is the insulation thickness in millimetres;

$k_{\text{dens}}$  is given in table E2.

(4) For a void cavity of depth from 45 to 200 mm the basic insulation value should be taken as  $t_{\text{ins},0} = 5,0$  min.

### E2.3 Position coefficients

**AC2** (1) For walls with single layered claddings, the position coefficient for panels on the exposed side of walls should be taken from table E3, and for panels on the unexposed side of walls from table E4, utilising the following expressions:

$$k_{\text{pos}} = \min \begin{cases} 0,02 h_p + 0,54 \\ 1 \end{cases} \quad (\text{E.9})$$

$$k_{\text{pos}} = 0,07 h_p - 0,17 \quad (\text{E.10})$$

where  $h_p$  is the thickness of the panel on the exposed side.

Where the exposed panel is made of materials other than gypsum plasterboard type F, the position coefficient,  $k_{\text{pos}}$ , for a void cavity and an insulation layer should be taken as 1,0. Where the exposed panel is made of gypsum plasterboard type F, the position coefficient should be taken as:

- $k_{\text{pos}} = 1,5$  for a void cavity, or a cavity filled with rock fibre insulation;
- $k_{\text{pos}} = 2,0$  for a cavity filled with glass fibre insulation. **AC2**

(2) For walls with double layered claddings, see figure E2, the position coefficients should be taken from table E5.

(3) For floors exposed to fire from below, the position coefficients for the exposed panels given in table E.3 should be multiplied by 0,8.

### E2.4 Effect of joints

(1) The joint coefficient  $k_j$  should be taken as  $k_j = 1$  for the following:

- panel joints fixed to a batten of at least the same thickness or to a structural element;
- wood panelling.

NOTE: For wood panelling the effect of joints is included in the basic insulation values  $t_{\text{ins},0}$  given by expression (E.5).

(2) For panel joints not fixed to a batten, the joint coefficient  $k_j$  should be taken from tables E6 and E7.

(3) For joints in insulation batts, the joint coefficient should be taken as  $k_j = 1$ .

Table E2 — Values of  $k_{\text{dens}}$  for cavity insulation materials

Cavity material	Density kg/m <sup>3</sup>	$k_{\text{dens}}$ <sup>a</sup>
Glass fibre	15	0,9
	20	1,0
	26	1,2
Rock fibre	26	1,0
	50	1,1
<sup>a</sup> For intermediate densities, linear interpolation may be applied		

AC<sub>2</sub> Table E3 — Position coefficients  $k_{\text{pos}}$  for single layered panels on the exposed side

Panel on the exposed side	Thickness mm	Position coefficient for panels backed by	
		rock or glass fibre insulation	void
Plywood with characteristic density ≥ 450 kg/m <sup>3</sup>	9 to 25	Expression (E.9)	0,8
Particleboard, fibreboard with characteristic density ≥ 600 kg/m <sup>3</sup>	9 to 25		
Wood panelling with characteristic density ≥ 400 kg/m <sup>3</sup>	15 to 19		
Gypsum plasterboard type A, H, F	9 to 15		

AC<sub>2</sub>

**Table E4 — Position coefficients  $k_{pos}$  for single layered panels on the unexposed side**

Panel on the exposed side	Thickness of panel on exposed side mm	Position coefficient for panels preceded by				
		Glass fibre	Rock fibre of thickness <sup>a</sup>			Void
			45 to 95	145	195	
Plywood with density $\geq 450$ kg/m <sup>3</sup>	9 to 25	Expression (E.10)	<b>1,5</b>	<b>3,9</b>	<b>4,9</b>	<b>0,6</b>
Particleboard and fibreboard with density $\geq 600$ kg/m <sup>3</sup>	9 to 25	Expression (E.10)				0,6
Wood panelling with density $\geq 400$ kg/m <sup>3</sup>	15 19	0,45 0,67				0,6
Gypsum plasterboard type A, H, F	9 to 15	Expression (E.10)				0,7

<sup>a</sup> For intermediate values, linear interpolation may be applied.

AC<sub>2</sub>

**Table E5 — Position coefficients  $k_{pos}$  for walls with double layered panels**

Construction:		Layer number				
Layer number and material		1	2	3	4	5
1, 2, 4, 5 3	Wood-based panel Void	0,7	0,9	1,0	0,5	0,7
1, 2, 4, 5 3	Gypsum plasterboard type A or H Void	1,0	0,8	1,0	0,8	0,7
1, 5 2, 4 3	Gypsum plasterboard type A or H Wood-based panel Void	1,0	0,8	1,0	0,8	0,7
1, 5 2, 4 3	Wood-based panel Gypsum plasterboard type A or H Void	1,0	0,6	1,0	0,8	0,7
1, 2, 4, 5 3	Wood-based panel Rock fibre batts	0,7	0,6	1,0	1,0	1,5
1, 2, 4, 5 3	Gypsum plasterboard type A or H Rock fibre batts	1,0	0,6	1,0	0,9	1,5
1, 5 2, 4 3	Gypsum plasterboard type A or H Wood-based panel Rock fibre batts	1,0	0,8	1,0	1,0	1,2
1, 5 2, 4 3	Wood-based panel Gypsum plasterboard type A or H Rock fibre batts	1,0	0,6	1,0	1,0	1,5

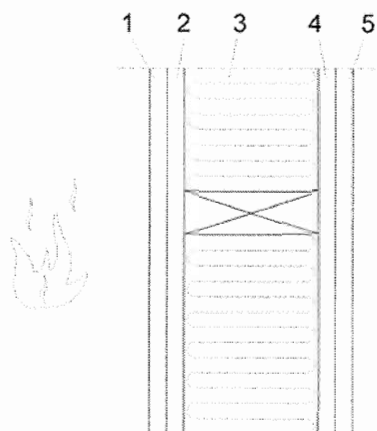


Figure E2 — Definition of layer numbers

Table E6 — Joint coefficient  $k_j$  to account for the effect of joints in wood-based panels which are not backed by battens

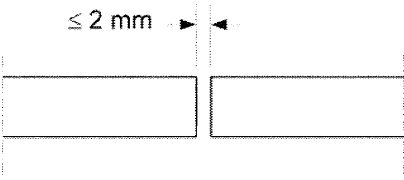
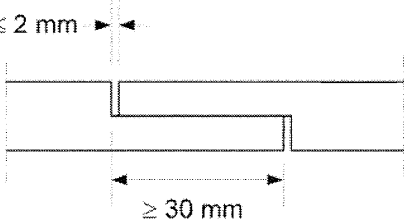
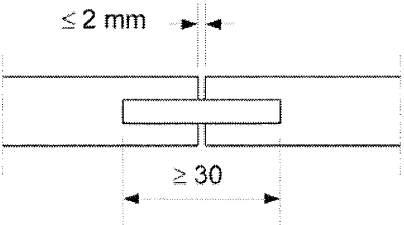
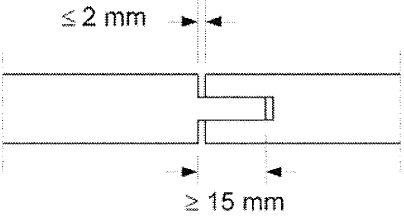
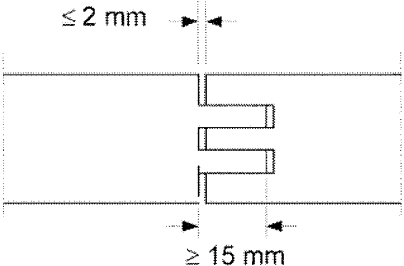
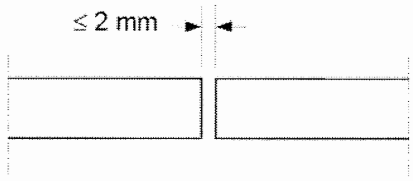
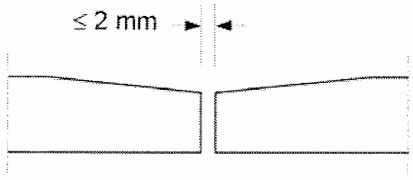
	Joint type	$k_j$
a		0,2
b		0,3
c		0,4
d		0,4
e		0,6

Table E7 — Joint coefficient  $k_j$  to account for the effect of joints in panels of gypsum plasterboard which are not backed by battens

	Joint type	Type	$k_j$	
			Filled joints	Unfilled joints
a		A, H, F	1,0	0,2
b		A, H, F	1,0	0,15

## Annex F (informative) Guidance for users of this Eurocode Part

(1) In this annex flow charts are given as guidance for users of EN 1995-1-2, see figure F1 and F2.

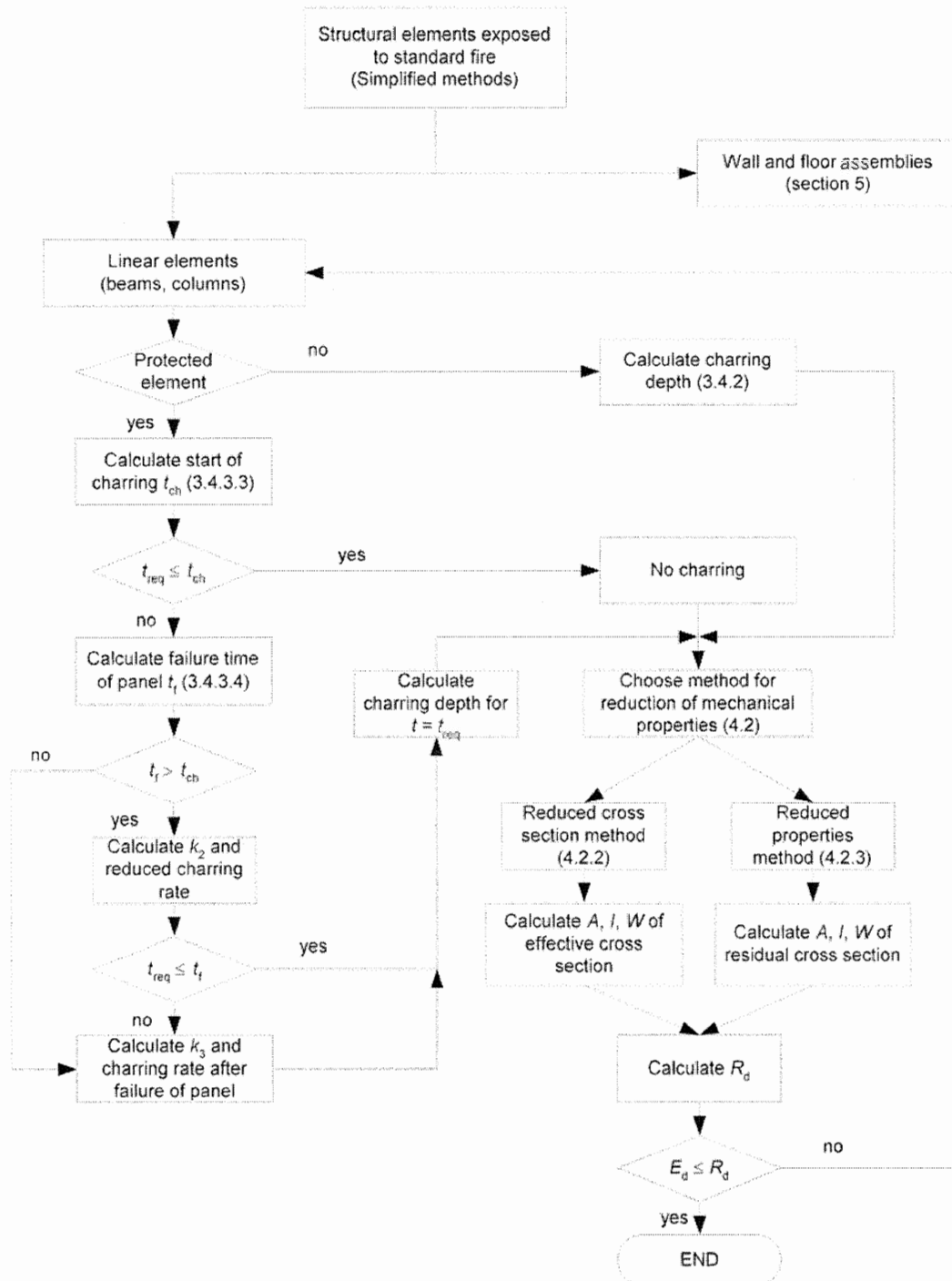


Figure F1 — Flow chart outlining the design procedure to check the load-bearing function of structural members

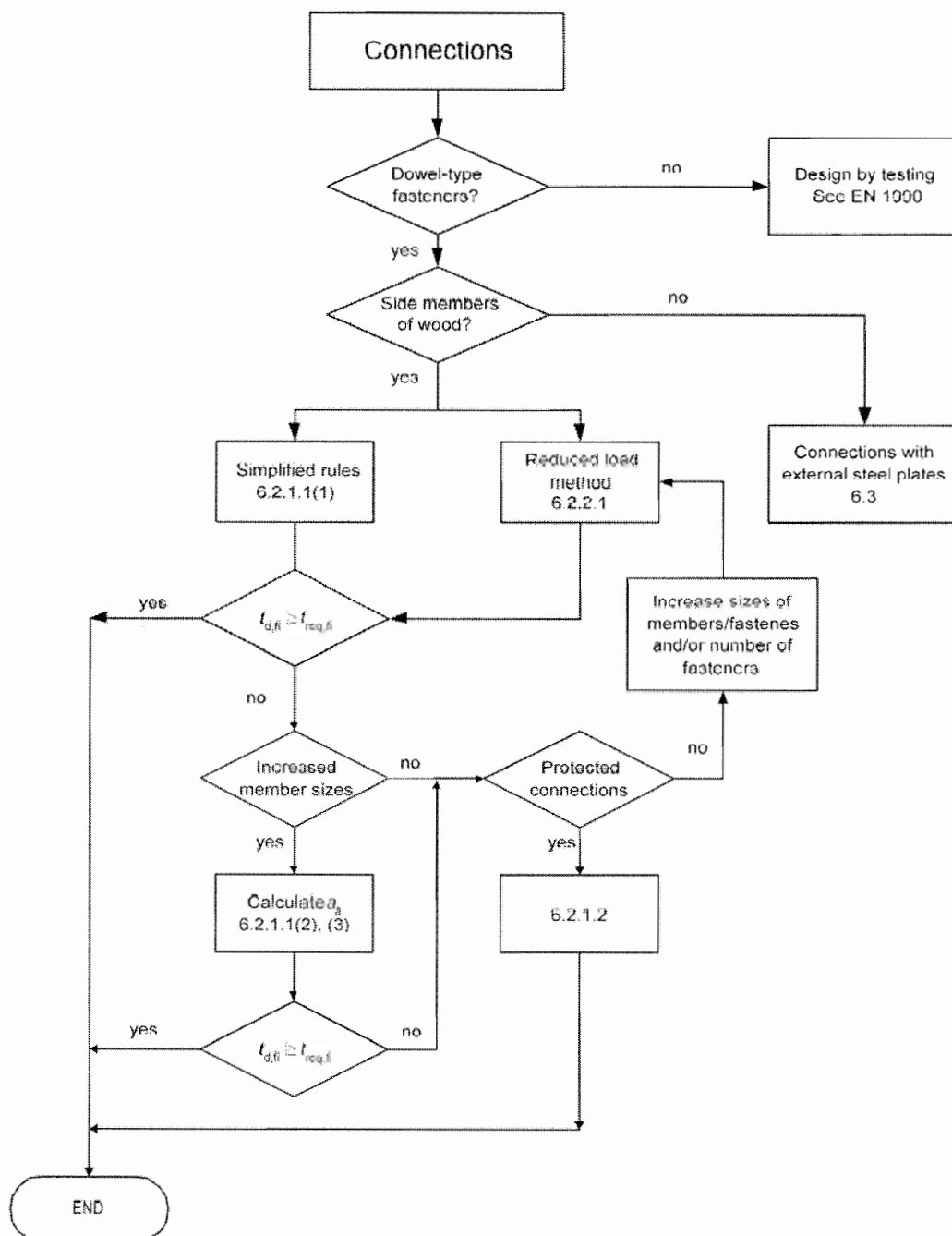


Figure F2 — Flow chart for the design procedure of connections

# *The European Union*

## **EDICT OF GOVERNMENT**

In order to promote public education and public safety, equal justice for all, a better informed citizenry, the rule of law, world trade and world peace, this legal document is hereby made available on a noncommercial basis, as it is the right of all humans to know and speak the laws that govern them.

EN 1995-2 (2004) (English): Eurocode 5: Design of timber structures - Part 2: Bridges [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]



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English version

## Eurocode 5: Design of timber structures - Part 2: Bridges

Eurocode 5: Conception et calcul des structures bois -  
Partie 2: Ponts

Eurocode 5: Bemessung und Konstruktion von Holzbauten  
- Teil 2: Brücken

This European Standard was approved by CEN on 26 August 2004.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Central Secretariat or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Central Secretariat has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.



EUROPEAN COMMITTEE FOR STANDARDIZATION  
COMITÉ EUROPÉEN DE NORMALISATION  
EUROPÄISCHES KOMITEE FÜR NORMUNG

Management Centre: rue de Stassart, 36 B-1050 Brussels

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## Foreword

This European Standard EN 1995-2 has been prepared by Technical Committee CEN/TC250 "Structural Eurocodes", the Secretariat of which is held by BSI.

This European Standard shall be given the status of a National Standard, either by publication of an identical text or by endorsement, at the latest by May 2005, and conflicting national standards shall be withdrawn at the latest by March 2010.

This European Standard supersedes ENV 1995-2:1997.

CEN/TC250 is responsible for all Structural Eurocodes.

According to the CEN/CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxemburg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

## Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement<sup>1</sup> between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links de facto the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products – CPD – and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards, generally consisting of a number of Parts:

EN 1990:2002	Eurocode: Basis of Structural Design
EN 1991	Eurocode 1: Actions on structures
EN 1992	Eurocode 2: Design of concrete structures
EN 1993	Eurocode 3: Design of steel structures
EN 1994	Eurocode 4: Design of composite steel and concrete structures
EN 1995	Eurocode 5: Design of timber structures
EN 1996	Eurocode 6: Design of masonry structures
EN 1997	Eurocode 7: Geotechnical design

<sup>1</sup> Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

EN 1998 Eurocode 8: Design of structures for earthquake resistance  
EN 1999 Eurocode 9: Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

### Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services ;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents<sup>2</sup> referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards<sup>3</sup>. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

### National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex.

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e.:

- values and/or classes where alternatives are given in the Eurocode;
- values to be used where a symbol only is given in the Eurocode;
- country specific data (geographical, climatic, etc.), e.g. snow map;
- the procedure to be used where alternative procedures are given in the Eurocode;

---

<sup>2</sup> According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

<sup>3</sup> According to Art. 12 of the CPD the interpretative documents shall :  
give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;  
indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;  
serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

- decisions on the application of informative annexes;
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

### **Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products**

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works<sup>4</sup>. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

### **Additional information specific to EN 1995-2**

EN 1995 describes the Principles and requirements for safety, serviceability and durability of timber bridges. It is based on the limit state concept used in conjunction with a partial factor method.

For the design of new structures, EN 1995-2 is intended to be used, for direct application, together with EN 1995-1-1 and EN1990:2002 and relevant Parts of EN 1991.

Numerical values for partial factors and other reliability parameters are recommended as basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and of quality management applies. When EN 1995-2 is used as a base document by other CEN/TCs the same values need to be taken.

### **National annex for EN 1995-2**

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1995-2 should have a National annex containing all Nationally Determined Parameters to be used for the design of bridges to be constructed in the relevant country.

National choice is allowed in EN 1995-2 through clauses:

- 2.3.1.2(1) Load-duration assignment
- 2.4.1 Partial factors for material properties
- 7.2 Limiting values for deflection
- 7.3.1(2) Damping ratios

<sup>4</sup> see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

## Section 1 General

### 1.1 Scope

#### 1.1.1 Scope of EN 1990

(1) EN 1990 applies to the design of buildings and civil engineering works in timber (solid timber, sawn, planed or in pole form, glued laminated timber or wood-based structural products e.g. LVL) or wood-based panels jointed together with adhesives or mechanical fasteners. It complies with the principles and requirements for the safety and serviceability of structures, and the basis of design and verification that are given in EN 1990:2002.

(2) EN 1990 is only concerned with requirements for mechanical resistance, serviceability, durability and fire resistance of timber structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(3) EN 1990 is intended to be used in conjunction with:

EN 1990:2002 Eurocode – Basis of structural design

EN 1991 “Actions on structures”

EN’s for construction products relevant to timber structures

EN 1998 “Design of structures for earthquake resistance”, when timber structures are built in seismic regions

(4) EN 1990 is subdivided into various parts:

EN 1995-1 General

EN 1995-2 Bridges

(5) EN 1995-1 “General” comprises:

EN 1995-1-1 General – Common rules and rules for buildings

EN 1995-1-2 General – Structural Fire Design

#### 1.1.2 Scope of EN 1995-2

(1) EN 1995-2 gives general design rules for the structural parts of bridges, i.e. structural members of importance for the reliability of the whole bridge or major parts of it, made of timber or other wood-based materials, either singly or compositely with concrete, steel or other materials.

(2) The following subjects are dealt with in EN 1995-2:

Section 1: General

Section 2: Basis of design

Section 3: Material properties

Section 4: Durability

Section 5: Basis of structural analysis

Section 6: Ultimate limit states

Section 7: Serviceability limit states

Section 8: Connections

Section 9: Structural detailing and control

(3) Section 1 and Section 2 also provide additional clauses to those given in EN 1990:2002 “Eurocode: Basis of structural design”.

(4) Unless specifically stated, EN 1995-1-1 applies.

### 1.2 Normative references

(1) The following normative documents contain provisions which, through references in this text, constitute provisions of this European standard. For dated references, subsequent amendments

to or revisions of any of these publications do not apply. However, parties to agreements based on this European standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references the latest edition of the normative document referred to applies.

#### European Standards:

EN 1990:2002	Eurocode – Basis of structural design
EN1990:2002/A1	Eurocode – Basis of structural design/Amendment A1 – Annex A2: Application to Bridges
EN 1991-1-4	Eurocode 1: Actions on structures – Part 1-4: Wind loads
EN 1991-2	Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges
EN 1992-1-1	Eurocode 2: Design of concrete structures – Part 1-1: Common rules and rules for buildings
EN 1992-2	Eurocode 2: Design of concrete structures – Part 2: Bridges
EN 1993-2	Eurocode 3: Design of steel structures – Part 2: Bridges
EN 1995-1-1	Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings
EN 10138-1	Prestressing steels – Part 1: General requirements
EN 10138-4	Prestressing steels – Part 4: Bars

### 1.3 Assumptions

(1) Additional requirements for execution, maintenance and control are given in section 9.

### 1.4 Distinction between principles and application rules

(1) See 1.4(1) of EN 1995-1-1.

### 1.5 Definitions

#### 1.5.1 General

(1)P The definitions of EN 1990:2002 clause 1.5 and EN 1995-1-1 clause 1.5 apply.

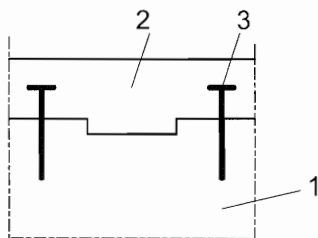
#### 1.5.2 Additional terms and definitions used in this present standard

##### 1.5.2.1

##### **Grooved connection**

Shear connection consisting of the integral part of one member embedded in the contact face of the other member. The contacted parts are normally held together by mechanical fasteners.

NOTE: An example of a grooved connection is shown in figure 1.1.



Key:

- 1 Timber
- 2 Concrete
- 3 Fastener

**Figure 1.1 – Example of grooved connection**

### 1.5.2.2

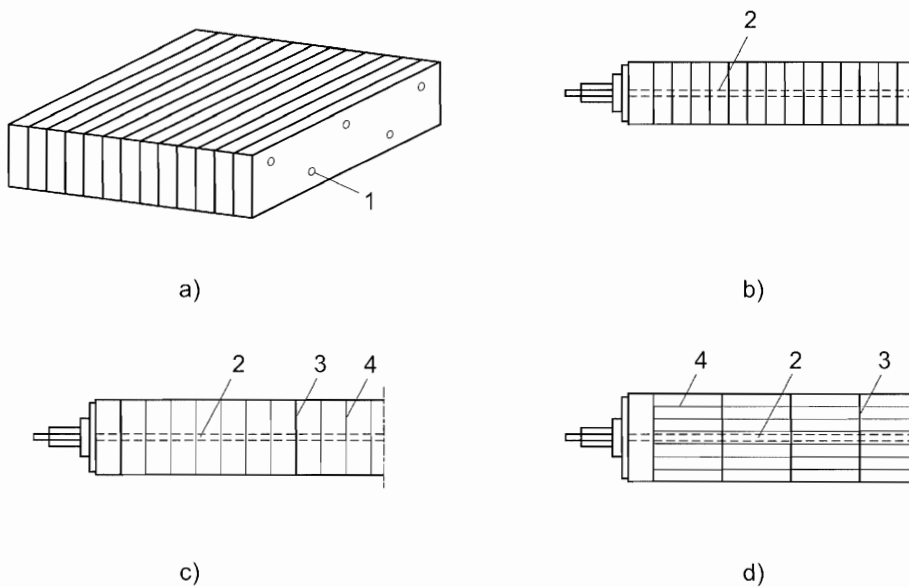
#### Laminated deck plates

Deck plates made of laminations, arranged edgewise or flatwise, held together by mechanical fasteners or gluing, see figures 1.2 and 1.3.

### 1.5.2.3

#### Stress-laminated deck plates

Laminated deck plates made of edgewise arranged laminations with surfaces either sawn or planed, held together by pre-stressing, see figure 1.2.b, c and d.



Key:

- 1 Nail or screw
- 2 Pre-stressing bar or tendon
- 3 Glue-line between glued laminated members
- 4 Glue-line between laminations in glued laminated members

**Figure 1.2 – Examples of deck plates made of edgewise arranged laminations**

a) nail-laminated or screw-laminated

b) pre-stressed, but not glued

c) glued and pre-stressed glued laminated beams positioned flatwise

d) glued and pre-stressed glued laminated beams positioned edgewise

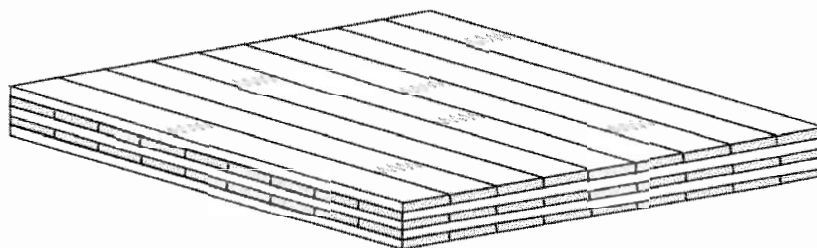
**1.5.2.4****Cross-laminated deck plates**

Laminated deck plates made of laminations in layers of different grain direction (crosswise or at different angles). The layers are glued together or connected using mechanical fasteners, see figure 1.3.

**1.5.2.5****Pre-stressing**

A permanent effect due to controlled forces and/or deformations imposed on a structure.

NOTE: An example is the lateral pre-stressing of timber deck plates by means of bars or tendons, see figure 1.2 b to d.



**Figure 1.3 – Example of cross-laminated deck plate**

**1.6 Symbols used in EN 1995-2**

For the purpose of EN 1995-2, the following symbols apply.

Latin upper case letters

$A$	Area of bridge deck
$E_{0,\text{mean}}$	Mean modulus of elasticity parallel to grain
$E_{90,\text{mean}}$	Mean modulus of elasticity perpendicular to the grain
$F$	Force
$F_{t,\text{Ed}}$	Design tensile force between timber and concrete
$F_{v,\text{Ed}}$	Design shear force between timber and concrete
$G_{0,\text{mean}}$	Mean shear modulus parallel to grain
$G_{90,\text{mean}}$	Mean shear modulus perpendicular to grain (rolling shear)
$M$	Total mass of bridge
$M_{\text{beam}}$	Bending moment in a beam representing a plate
$M_{\text{max,beam}}$	Maximum bending moment in a beam representing a plate
$N_{\text{obs}}$	Number of constant amplitude stress cycles per year
$R$	Ratio of stresses

Latin lower case letters

$a$	Distance; fatigue coefficient
$a_{\text{hor},1}$	Horizontal acceleration from one person crossing the bridge
$a_{\text{hor},n}$	Horizontal acceleration from several people crossing the bridge
$a_{\text{vert},1}$	Vertical acceleration from one person crossing the bridge
$a_{\text{vert},n}$	Vertical acceleration from several people crossing the bridge
$b$	Fatigue coefficient
$b_{\text{ef}}$	Effective width
$b_{\text{ef},c}$	Total effective width of concrete slab
$b_{\text{ef},1}; b_{\text{ef},2}$	Effective width of concrete slab

$b_{\text{lam}}$	Width of the lamination
$b_w$	Width of the loaded area on the contact surface of deck plate
$b_{w,\text{middle}}$	Width of the loaded area in the middle of the deck plate
$d$	Diameter; outer diameter of rod; distance
$h$	Depth of beam; thickness of plate
$f_{c,90,d}$	Design compressive strength perpendicular to grain
$f_{\text{fat},d}$	Design value of fatigue strength
$f_k$	Characteristic strength
$f_{m,d,\text{deck}}$	Design bending strength of deck plate
$f_{v,d,\text{deck}}$	Design shear strength of deck plate
$f_{m,d,\text{lam}}$	Design bending strength of laminations
$f_{v,d,\text{lam}}$	Design shear strength of laminations
$f_{\text{vert}}, f_{\text{hor}}$	Fundamental natural frequency of vertical and horizontal vibrations
$k_{c,90}$	Factor for compressive strength perpendicular to the grain
$k_{\text{fat}}$	Factor representing the reduction of strength with number of load cycles
$k_{\text{hor}}$	Coefficient
$k_{\text{mod}}$	Modification factor
$k_{\text{sys}}$	System strength factor
$k_{\text{vert}}$	Coefficient
$\ell$	Span
$\ell_1$	Distance
$m$	Mass; mass per unit length
$m_{\text{plate}}$	Bending moment in a plate per unit length
$m_{\text{max,plate}}$	Maximum bending moment in a plate
$n$	Number of loaded laminations; number of pedestrians
$n_{\text{ADT}}$	Expected annual average daily traffic over the lifetime of the structure
$t$	Time; thickness of lamination
$t_L$	Design service life of the structure expressed in years

#### Greek lower case letters

$\alpha$	Expected percentage of observed heavy lorries passing over the bridge
$\beta$	Factor based on the damage consequence; angle of stress dispersion
$\gamma_M$	Partial factor for timber material properties, also accounting for model uncertainties and dimensional variations
$\gamma_{M,c}$	Partial factor for concrete material properties, also accounting for model uncertainties and dimensional variations
$\gamma_{M,s}$	Partial factor for steel material properties, also accounting for model uncertainties and dimensional variations
$\gamma_{M,v}$	Partial factor for shear connectors, also accounting for model uncertainties and dimensional variations
$\gamma_{M,\text{fat}}$	Partial safety factor for fatigue verification of materials, also accounting for model uncertainties and dimensional variations
$\kappa$	Ratio for fatigue verification
$\rho_{\text{mean}}$	Mean density
$\mu_d$	Design coefficient of friction
$\sigma_{d,\text{max}}$	Numerically largest value of design stress for fatigue loading
$\sigma_{d,\text{min}}$	Numerically smallest value of design stress for fatigue loading
$\sigma_{p,\text{min}}$	Minimum long-term residual compressive stress due to pre-stressing;
$\zeta$	Damping ratio

## Section 2 Basis of design

### 2.1 Basic requirements

(1)P The design of timber bridges shall be in accordance with EN 1990:2002.

### 2.2 Principles of limit state design

(1) See 2.2 of EN 1995-1-1.

### 2.3 Basic variables

#### 2.3.1 Actions and environmental influences

##### 2.3.1.1 General

(1) Actions to be used in design of bridges may be obtained from the relevant parts of EN 1991.

Note 1: The relevant parts of EN 1991 for use in design include:

EN 1991-1-1	Densities, self-weight and imposed loads
EN 1991-1-3	Snow loads
EN 1991-1-4	Wind loads
EN 1991-1-5	Thermal actions
EN 1991-1-6	Actions during execution
EN 1991-1-7	Accidental actions due to impact and explosions
EN 1991-2	Traffic loads on bridges.

##### 2.3.1.2 Load-duration classes

(1) Variable actions due to the passage of vehicular and pedestrian traffic should be regarded as short-term actions.

NOTE: Examples of load-duration assignments are given in note to 2.3.1 of EN 1995-1-1. The recommended load-duration assignment for actions during erection is short-term. The National choice may be given in the National annex.

(2) Initial pre-stressing forces perpendicular to the grain should be regarded as short-term actions.

### 2.4 Verification by the partial factor method

#### 2.4.1 Design value of material property

NOTE: For fundamental combinations, the recommended partial factors for material properties,  $\gamma_M$ , are given in table 2.1. For accidental combinations, the recommended value of partial factor is  $\gamma_M = 1,0$ . Information on the National choice may be found in the National annex.

**Table 2.1 – Recommended partial factors for material properties**

1. Timber and wood-based materials	
– normal verification	$\gamma_M = 1,3$
– solid timber	$\gamma_M = 1,25$
– glued laminated timber	$\gamma_M = 1,2$
– LVL, plywood, OSB	$\gamma_{M,fat} = 1,0$
– fatigue verification	
2. Connections	
– normal verification	$\gamma_M = 1,3$
– fatigue verification	$\gamma_{M,fat} = 1,0$
3. Steel used in composite members	$\gamma_{M,s} = 1,15$
4. Concrete used in composite members	$\gamma_{M,c} = 1,5$
5. Shear connectors between timber and concrete in composite members	
– normal verification	$\gamma_{M,v} = 1,25$
– fatigue verification	$\gamma_{M,v,fat} = 1,0$
6. Pre-stressing steel elements	$\gamma_{M,s} = 1,15$

### **Section 3 Material properties**

(1)P Pre-stressing steels shall comply with EN 10138-1 and EN 10138-4.

## Section 4 Durability

### 4.1 Timber

(1) The effect of precipitation, wind and solar radiation should be taken into account.

NOTE 1: The effect of direct weathering by precipitation or solar radiation of structural timber members can be reduced by constructional preservation measures, or by using timber with sufficient natural durability, or timber preservatively treated against biological attacks.

NOTE 2: Where a partial or complete covering of the main structural elements is not practical, durability can be improved by 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 (e.g. 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
- choosing a geometry for the structure that ensures natural ventilation of all timber parts.

NOTE 3: The risk of increased moisture content near the ground, e.g. due to insufficient ventilation due to vegetation between the timber and the ground, or splashing water, can be reduced by one or more of the following measures:

- covering of the ground by coarse gravel or similar to limit vegetation;
- use of an increased distance between the timber parts and the ground level.

(2)P Where structural timber members are exposed to abrasion by traffic, the depth used in the design shall be the minimum permitted before replacement.

### 4.2 Resistance to corrosion

(1) EN 1995-1-1 clause 4.2 applies to fasteners. EN 1993-2 applies to steel parts other than fasteners.

NOTE: An example of especially corrosive conditions is a timber bridge where corrosive de-icing cannot be excluded.

(2)P The possibility of stress corrosion shall be taken into account.

(3) Steel parts encased in concrete, such as reinforcing bars and pre-stressing cables, should be protected according EN 1992-1-1 clause 4.4.1 and EN 1992-2.

(4) The effect of chemical treatment of timber, or timber with high acidic content, on the corrosion protection of fasteners should be taken into account.

### 4.3 Protection of timber decks from water by sealing

(1)P The elasticity of the seal layers shall be sufficient to follow the movement of the timber deck.

## Section 5 Basis of structural analysis

### 5.1 Laminated deck plates

#### 5.1.1 General

(1) The analysis of laminated timber deck plates should be based upon one of the following:

- the orthotropic plate theory;
- modelling the deck plate by a grid;
- a simplified method according to 5.1.3.

NOTE: In an advanced analysis, for deck plates made of softwood laminations, the relationships for the system properties should be taken from table 5.1. The Poisson ratio  $\nu$  may be taken as zero.

**Table 5.1 – System properties of laminated deck plates**

Type of deck plate	$E_{90, \text{mean}}/E_{0, \text{mean}}$	$G_{0, \text{mean}}/E_{0, \text{mean}}$	$G_{90, \text{mean}}/G_{0, \text{mean}}$
Nail-laminated	0	0,06	0,05
Stress-laminated			
– sawn	0,015	0,06	0,08
– planed	0,020	0,06	0,10
Glued-laminated	0,030	0,06	0,15

(2) For cross-laminated deck plates, see Figure 1.3, shear deformations should be taken into account.

#### 5.1.2 Concentrated vertical loads

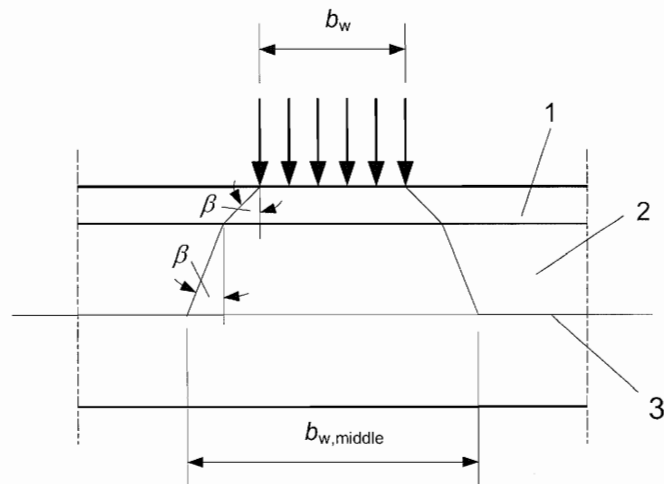
(1) Loads should be considered at a reference plane in the middle of the deck plate.

(2) For concentrated loads an effective load area with respect to the middle plane of the deck plate should be assumed, see figure 5.1, where:

$b_w$  is the width of the loaded area on the contact surface of the pavement;

$b_{w, \text{middle}}$  is the width of the loaded area at the reference plane in the middle of the deck plate;

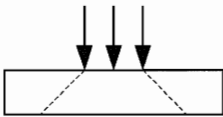
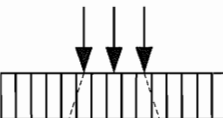
$\beta$  is the angle of dispersion according to table 5.2.



Key:

- 1 Pavement
- 2 Timber deck plate
- 3 Reference in middle of timber deck plate

Figure 5.1 – Dispersion of concentrated loads from contact area width  $b_w$ Table 5.2 – Dispersion angle  $\beta$  of concentrated loads for various materials

Pavement (in accordance with EN 1991-2 clause 4.3.6)		45°
Boards and planks		45°
Laminated timber deck plates:		
– in the direction of the grain		45°
– perpendicular to the grain		15°
Plywood and cross-laminated deck plates		45°

### 5.1.3 Simplified analysis

(1) 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

$$b_{ef} = b_{w,middle} + a \quad (5.1)$$

where:

$b_{w,middle}$  should be calculated according to 5.1.2(2);

$a$  should be taken from table 5.3.

Table 5.3 – Width  $a$  in m for determination of effective width of beam

Deck plate system	$a$ m
Nail-laminated deck plate	0,1
Stress-laminated or glued laminated	0,3
Cross-laminated timber	0,5
Composite concrete/timber deck structure	0,6

## 5.2 Composite members

(1)P For composite action of deck plate systems, the influence of joint slip shall be taken into account.

NOTE: See clause 8.2

## 5.3 Timber-concrete composite members

(1) The concrete part should be designed according to EN 1992-2.

(2) The steel fasteners and the grooved connections should be designed to transmit all forces due to composite action. Friction and adhesion between wood and concrete should not be taken into account, unless a special investigation is carried out.

(3) The effective width of the concrete plate of composite timber beam/concrete deck structures should be determined as:

$$b_{\text{ef,c}} = b + b_{\text{ef,1}} + b_{\text{ef,2}} \quad (5.2)$$

where:

$b$  is the width of the timber beam;

$b_{\text{ef,1}}, b_{\text{ef,2}}$  are the effective widths of the concrete flanges, as determined for a concrete T-section according to EN 1992-1-1, subclause 5.3.2.1.

(4)P For verification at ultimate limit state, cracks in the concrete plate shall be taken into account.

(5) The effect of concrete tension stiffening may be included. As a simple approach the stiffness of the cracked part of the concrete cross-section may be taken as 40 % of the stiffness in uncracked condition. In such areas the need for an adequate crack distributing reinforcement should be observed.

## Section 6 Ultimate limit states

### 6.1 Deck plates

#### 6.1.1 System strength

(1) The relevant rules given in EN 1995-1-1 clause 6.7 apply

(2) The design bending and shear strength of the deck plate should be calculated as:

$$f_{m,d,deck} = k_{sys} f_{m,d,lam} \quad (6.1)$$

$$f_{v,d,deck} = k_{sys} f_{v,d,lam} \quad (6.2)$$

where:

$f_{m,d,lam}$  is the design bending strength of the laminations;

$f_{v,d,lam}$  is the design shear strength of the laminations;

$k_{sys}$  is the system strength factor, see EN 1995-1-1. For decks in accordance to Fig. 1.2d EN 1995-1-1 figure 6.14 line 1 should be used.

For the calculation of  $k_{sys}$ , the number of loaded laminations should be taken as:

$$n = \frac{b_{ef}}{b_{lam}} \quad (6.3)$$

with:

$b_{ef}$  is the effective width;;

$b_{lam}$  is the width of the laminations.

(3) The effective width  $b_{ef}$  should be taken as (see figure 6.1):

$$b_{ef} = \frac{M_{max,beam}}{m_{max,plate}} \quad (6.4)$$

where:

$M_{max,beam}$  is the maximum bending moment in a beam representing the plate;

$m_{max,plate}$  is the maximum bending moment in the plate calculated by a plate analysis.

NOTE: In 5.1.3 a simplified method is given for the determination of the effective width.

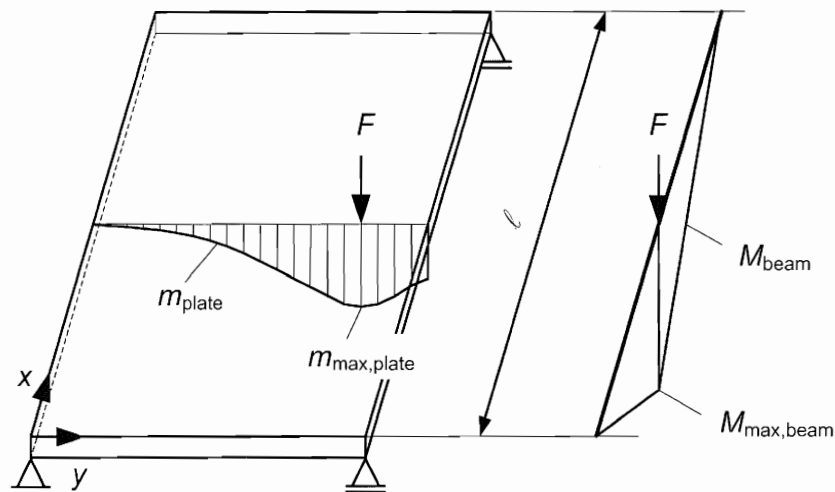


Figure 6.1 – Example of bending moment distribution in the plate for determination of effective width

### 6.1.2 Stress-laminated deck plates

(1)P The long-term pre-stressing forces shall be such that no inter-laminar slip occurs.

(2) The following requirement should be satisfied:

$$F_{v,Ed} \leq \mu_d \sigma_{p,min} h \quad (6.5)$$

where:

$F_{v,Ed}$  is the design shear force per unit length, caused by vertical and horizontal actions;

$\mu_d$  is the design value of coefficient of friction;

$\sigma_{p,min}$  is the minimum long-term residual compressive stress due to pre-stressing;

$h$  is the thickness of the plate.

(3) The coefficient of friction should take into account the following:

- wood species;
- roughness of contact surface;
- treatment of the timber;
- residual stress level between laminations.

(4) Unless other values have been verified, the design static friction coefficients,  $\mu_d$ , between softwood timber laminations, and between softwood timber laminations and concrete, should be taken from table 6.1. For moisture contents between 12 and 16 %, the values may be obtained by linear interpolation.

(5) In areas subjected to concentrated loads, the minimum long-term residual compressive stress,  $\sigma_{p,min}$ , due to pre-stressing between laminations should be not less than  $0,35 \text{ N/mm}^2$ .

(6) The long-term residual pre-stressing stress may 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%;

- the variation of in-service moisture content in the deck plate is limited by adequate protection, e.g. a sealing layer.

**Table 6.1 – Design values of coefficient of friction  $\mu_d$**

Lamination surface roughness	Perpendicular to grain		Parallel to grain	
	Moisture content $\leq 12\%$	Moisture content $\geq 16\%$	Moisture content $\leq 12\%$	Moisture content $\geq 16\%$
Sawn timber to sawn timber	0,30	0,45	0,23	0,35
Planed timber to planed timber	0,20	0,40	0,17	0,30
Sawn timber to planed timber	0,30	0,45	0,23	0,35
Timber to concrete	0,40	0,40	0,40	0,40

(7) The resulting pre-stressing forces should act centrally on the timber cross-section.

(8)P The compressive stress perpendicular to the grain during pre-stressing in the contact area of the anchorage plate shall be verified.

(9) The factor  $k_{c,90}$  according to EN 1995-1-1 may be taken as 1,3.

(10) Not more than one butt joint should occur in any four adjacent laminations within a distance  $\ell_1$  given as

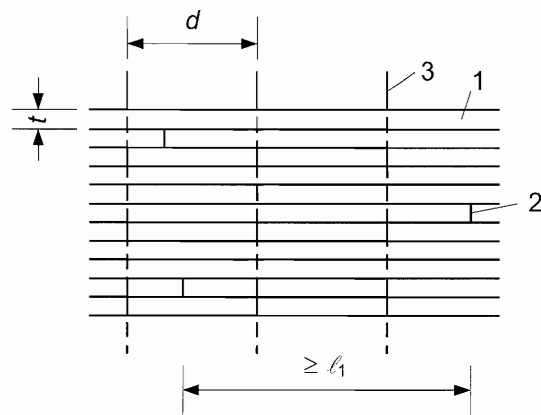
$$\ell_1 = \min \begin{cases} 2d \\ 30t \\ 1,2 \text{ m} \end{cases} \quad (6.6)$$

where:

$d$  is the distance between the pre-stressing elements;

$t$  is the thickness of the laminations in the direction of pre-stressing.

(11) In calculating the longitudinal strength of stress-laminated deck plates, the section should be reduced in proportion to the number of butt joints within a distance of 4 times the thickness of laminations in the direction of pre-stressing.



Key:

- 1 Lamination
- 2 Butt joint
- 3 Pre-stressing element

**Figure 6.2 — Butt joints in stress-laminated deck plates**

## 6.2 Fatigue

(1)P For structures or structural parts and connections that are subjected to frequent stress variations from traffic or wind loading, it shall be verified that no failure or major damage will occur due to fatigue.

NOTE 1: A fatigue verification is normally not required for footbridges.

NOTE 2: A simplified verification method is given in annex A (informative).

## Section 7 Serviceability limit states

### 7.1 General

(1) In the calculations, mean values of density should be used.

### 7.2 Limiting values for deflections

NOTE: The range of limiting values for deflections due to traffic load only, for beams, plates or trusses with span  $\ell$  is given in Table 7.1. The recommended values are underlined. Information on National choice may be found in the National annex.

**Table 7.1 – Limiting values for deflections for beams, plates and trusses**

Action	Range of limiting values
Characteristic traffic load	<u><math>\ell/400</math></u> to $\ell/500$
Pedestrian load and low traffic load	<u><math>\ell/200</math></u> to $\ell/400$

### 7.3 Vibrations

#### 7.3.1 Vibrations caused by pedestrians

(1) For comfort criteria EN1990:2002/A1 applies.

(2) Where no other values have been verified, the damping ratio should be taken as:

- $\zeta = 0,010$  for structures without mechanical joints;
- $\zeta = 0,015$  for structures with mechanical joints.

NOTE 1: For specific structures, alternative damping ratios may be given in the National annex.

NOTE 2: A simplified method for assessing vibrations of timber bridges constructed with simply supported beams or trusses is given in Annex B.

#### 7.3.2 Vibrations caused by wind

(1)P EN 1991-1-4 applies

## Section 8 Connections

### 8.1 General

(1)P The following shall not be used in bridges:

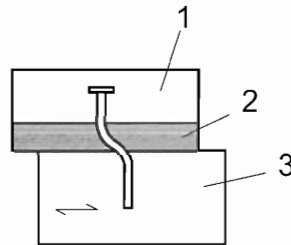
- axially loaded nails;
- stapled connections;
- connections made with punched metal plate fasteners.

### 8.2 Timber-concrete connections in composite beams

#### 8.2.1 Laterally loaded dowel-type fasteners

(1) The rope effect should not be used.

(2) Where there is an intermediate non-structural layer between the timber and the concrete (e.g. for formwork), see figure 8.1, the strength and stiffness parameters should be determined by a special analysis or by tests.



Key:

- 1 Concrete
- 2 Non-structural intermediate layer
- 3 Timber

**Figure 8.1 – Intermediate layer between concrete and timber**

#### 8.2.2 Grooved connections

(1) For grooved connections, see figure 1.1, the shear force should be taken by direct contact pressure between the wood and the concrete cast in the groove.

(2) It should be verified that the resistance of the concrete part and the timber part of the connection is sufficient.

(3)P The concrete and timber parts shall be held together so that they can not separate.

(4) The connection should be designed for a tensile force between the timber and the concrete with a magnitude of:

$$F_{t,Ed} = 0,1 F_{v,Ed} \quad (8.1)$$

where:

$F_{t,Ed}$  is the design tensile force between the timber and the concrete;

$F_{v,Ed}$  is the design shear force between the timber and the concrete.

## **Section 9 Structural detailing and control**

(1)P The relevant rules given in EN 1995-1-1 Section 10 also apply to the structural parts of bridges, with the exception of clauses 10.8 and 10.9.

(2) Before attaching a seal layer on a deck plate, the deck system should be dry and the surface should satisfy the requirements of the seal layer.

## Annex A (informative) Fatigue verification

### A.1 General

(1) This simplified method is based on an equivalent constant amplitude fatigue loading, representing the fatigue effects of the full spectrum of loading events.

NOTE: More advanced fatigue verification for varying stress amplitude can be based on a cumulative linear damage theory (Palmgren-Miner hypothesis).

(2) The stress should be determined by an elastic analysis under the specified action. The stresses should allow for stiff or semi-rigid connections and second order effects from deformations and distortions.

(3) A fatigue verification is required if the ratio  $\kappa$  given by expression (A.1) is greater than:

- For members in compression perpendicular or parallel to grain: 0,6
- For members in bending or tension: 0,2
- For members in shear: 0,15
- For joints with dowels: 0,4
- For joints with nails: 0,1
- Other joints: 0,15

where:

$$\kappa = \frac{|\sigma_{d,max} - \sigma_{d,min}|}{\frac{f_k}{\gamma_{M,fat}}} \quad (A.1)$$

$\sigma_{d,max}$  is the numerically largest design stress from the fatigue loading;

$\sigma_{d,min}$  is the numerically smallest design stress from the fatigue loading;

$f_k$  is the relevant characteristic strength;

$\gamma_{M,fat}$  is the material partial factor for fatigue loading.

### A.2 Fatigue loading

(1) A simplified fatigue load model is built up of reduced loads (effects of actions) compared to the static loading models. The load model should give the maximum and minimum stresses in the actual structural members.

(2) The fatigue loading from traffic should be obtained from the project specification in conjunction with EN 1991-2.

(3) The number of constant amplitude stress cycles per year,  $N_{obs}$ , should either be taken from table 4.5 of EN 1991-2 or, if more detailed information about the actual traffic is available, be taken as:

$$N_{obs} = 365 n_{ADT} \alpha \quad (A.2)$$

where:

$N_{obs}$  is the number of constant amplitude stress cycles per year;

$n_{ADT}$  is the expected annual average daily traffic over the lifetime of the structure; the value of  $n_{ADT}$  should not be taken less than 1000;

$\alpha$  is the expected fraction of observed heavy lorries passing over the bridge, see EN 1991-2 clause 4.6 (e.g.  $\alpha = 0,1$ );

### A.3 Fatigue verification

(1) Unless the verification model is defined below or by special investigations, the ratio  $\kappa$  should be limited to the value defined in the previous clause A1(3).

(2) For a constant amplitude loading the fatigue verification criterion is:

$$\sigma_{d,max} \leq f_{fat,d} \quad (A.3)$$

where:

$\sigma_{d,max}$  is the numerically largest design stress from the fatigue loading;

$f_{fat,d}$  is the design value of fatigue strength.

(3) The design fatigue strength should be taken as:

$$f_{fat,d} = k_{fat} \frac{f_k}{\gamma_{M,fat}} \quad (A.4)$$

where:

$f_k$  is the characteristic strength for static loading;

$k_{fat}$  is a factor representing the reduction of strength with number of load cycles.

(4) The value of  $k_{fat}$  should be taken as:

$$k_{fat} = 1 - \frac{1-R}{a(b-R)} \log(\beta N_{obs} t_L) \geq 0 \quad (A.5)$$

where:

$$R = \sigma_{d,min} / \sigma_{d,max} \quad \text{with } -1 \leq R \leq 1; \quad (A.6)$$

$\sigma_{d,min}$  is the numerically smallest design stress from the fatigue loading;

$\sigma_{d,max}$  is the numerically largest design stress from the fatigue loading;

$N_{obs}$  is the number of constant amplitude stress cycles as defined above;

$t_L$  is the design service life of the structure expressed in years according to EN 1990:2002 (e.g. 100 years);  $\beta$  is a factor based on the damage consequence for the actual structural component;

$a, b$  are coefficients representing the type of fatigue action according to table A.1.

The factor  $\beta$  should be taken as:

- Substantial consequences:  $\beta = 3$
- Without substantial consequences:  $\beta = 1$

**Table A.1 – Values of coefficients  $a$  and  $b$** 

	$a$	$b$
Timber members in		
– compression, perpendicular or parallel to grain	2,0	9,0
– bending and tension	9,5	1,1
– shear	6,7	1,3
Connections with		
– dowels with $d \leq 12$ mm <sup>a</sup>	6,0	2,0
– nails	6,9	1,2
<sup>a</sup> The values for dowels are mainly based on tests on 12 mm tight-fitting dowels. Significantly larger diameter dowels or non-fitting bolts may have less favourable fatigue properties.		

## Annex B (informative) Vibrations caused by pedestrians

### B.1 General

(1) The rules given in this annex apply to timber bridges with simply supported beams or truss systems excited by pedestrians.

NOTE: Corresponding rules will be found in future versions of EN 1991-2.

### B.2 Vertical vibrations

(1) For one person crossing the bridge, the vertical acceleration  $a_{\text{vert},1}$  in  $\text{m/s}^2$  of the bridge should be taken as:

$$a_{\text{vert},1} = \begin{cases} \frac{200}{M \zeta} & \text{for } f_{\text{vert}} \leq 2,5 \text{ Hz} \\ \frac{100}{M \zeta} & \text{for } 2,5 \text{ Hz} < f_{\text{vert}} \leq 5,0 \text{ Hz} \end{cases} \quad (\text{B.1})$$

where:

$M$  is the total mass of the bridge in kg, given by  $M = m \ell$ ;

$\ell$  is the span of the bridge;

$m$  is the mass per unit length (self-weight) of the bridge in kg/m;

$\zeta$  is the damping ratio;

$f_{\text{vert}}$  is the fundamental natural frequency for vertical deformation of the bridge.

(2) For several persons crossing the bridge, the vertical acceleration  $a_{\text{vert},n}$  in  $\text{m/s}^2$  of the bridge should be calculated as:

$$a_{\text{vert},n} = 0,23 a_{\text{vert},1} n k_{\text{vert}} \quad (\text{B.2})$$

where:

$n$  is the number of pedestrians;

$k_{\text{vert}}$  is a coefficient according to figure B.1;

$a_{\text{vert},1}$  is the vertical acceleration for one person crossing the bridge determined according to expression (B.1).

The number of pedestrians,  $n$ , should be taken as:

- $n = 13$  for a distinct group of pedestrians;
- $n = 0,6 A$  for a continuous stream of pedestrians.

where  $A$  is the area of the bridge deck in  $\text{m}^2$ .

(3) If running persons are taken into account, the vertical acceleration  $a_{\text{vert},1}$  in  $\text{m/s}^2$  of the bridge caused by one single person running over the bridge, should be taken as:

$$a_{\text{vert},1} = \frac{600}{M \zeta} \quad \text{for } 2,5 \text{ Hz} < f_{\text{vert}} \leq 3,5 \text{ Hz} \quad (\text{B.3})$$

### B.3 Horizontal vibrations

(1) For one person crossing the bridge the horizontal acceleration  $a_{\text{hor},1}$  in  $\text{m/s}^2$  of the bridge should be calculated as:

$$a_{\text{hor},1} = \frac{50}{M \zeta} \quad \text{for } 0,5 \text{ Hz} \leq f_{\text{hor}} \leq 2,5 \text{ Hz} \quad (\text{B.4})$$

where  $f_{\text{hor}}$  is the fundamental natural frequency for horizontal deformation of the bridge.

(2) For several persons crossing the bridge, the horizontal acceleration  $a_{\text{hor},n}$  in  $\text{m/s}^2$  of the bridge should be calculated as:

$$a_{\text{hor},n} = 0,18 a_{\text{hor},1} n k_{\text{hor}} \quad (\text{B.5})$$

where:

$k_{\text{hor}}$  is a coefficient according to figure B.2.

The number of pedestrians,  $n$ , should be taken as:

- $n = 13$  for a distinct group of pedestrians;
- $n = 0,6 A$  for a continuous stream of pedestrians,

where  $A$  is the area of the bridge deck in  $\text{m}^2$ .

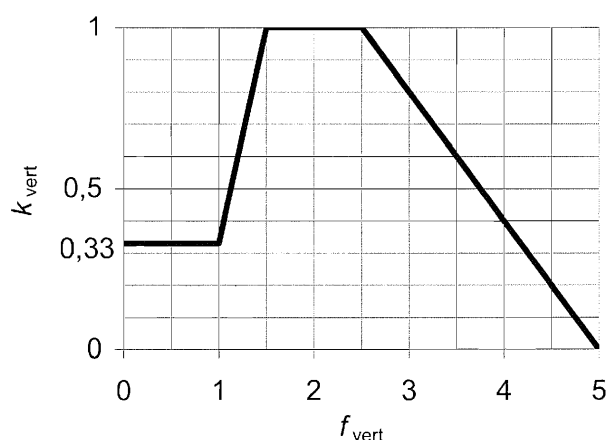


Figure B.1 – Relationship between the vertical fundamental natural frequency  $f_{\text{vert}}$  and the coefficient  $k_{\text{vert}}$

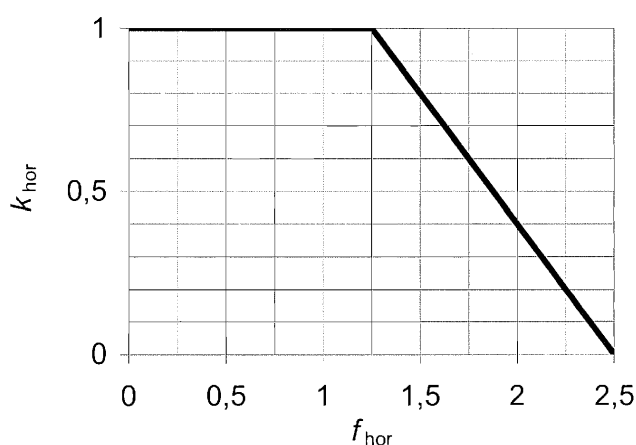


Figure B.2 – Relationship between the horizontal fundamental natural frequency  $f_{\text{hor}}$  and the coefficient  $k_{\text{hor}}$