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

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.

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Contents

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of an identical text or by endorsement, at the latest by May 2005, and conflicting national standards shall be withdrawn at the latest by March 2010.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[1] 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

[1] 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[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards[3]. 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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[2] 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.

[3] 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[4]. 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[5]" 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.

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[4] 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.
[5] 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.

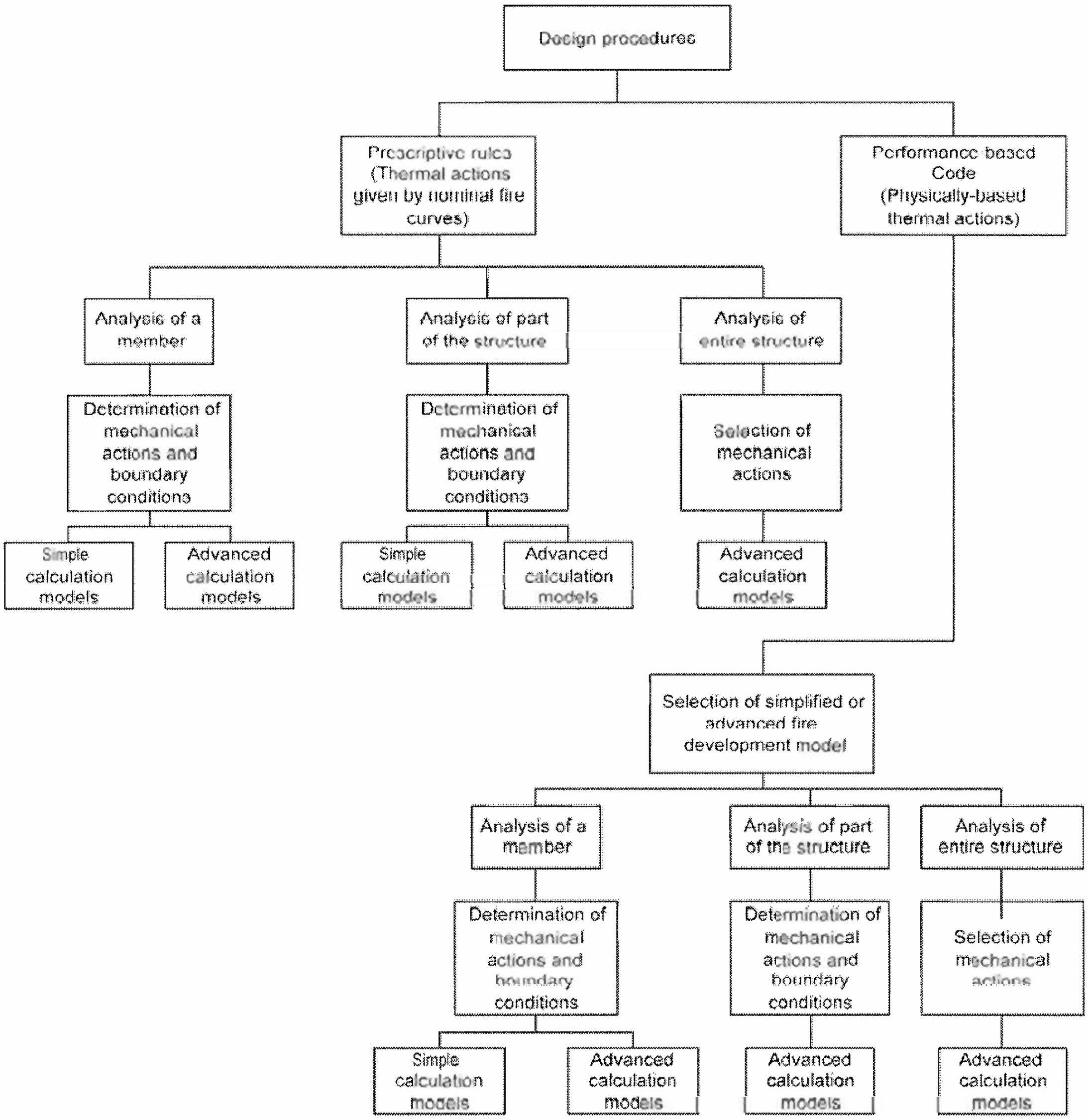


Figure 1 – Alternative design procedures

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_ρ	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_Θ	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

Γ	Factor accounting for the thermal properties of the boundaries of the compartment
Θ	Temperature

Greek lower case letters

β_0	Design charring rate for one-dimensional charring under standard fire exposure
β_n	Design notional charring rate under standard fire exposure
β_{par}	Design charring rate during heating phase of parametric fire curve
η	Conversion factor for the reduction of the load-bearing capacity in fire
η_f	Conversion factor for slip modulus
γ_{GA}	Partial factor for permanent actions in accidental design situations
γ_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
λ	Thermal conductivity
ρ	Density
ρ_k	Characteristic density
ω	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
ψ_{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

$$f_{\mathrm{d,fi}} = k_{\mathrm{mod,fi}} \frac{f_{20}}{\gamma_{\mathrm{M,fi}}} \tag{2.1}$$

$$S_{\mathrm{d,fi}} = k_{\mathrm{mod,fi}} \frac{S_{20}}{\gamma_{\mathrm{M,fi}}} \tag{2.2}$$

where:

$f_{\mathrm{d,fi}}$ is the design strength in fire;

$S_{\mathrm{d,fi}}$ is the design stiffness property (modulus of elasticity $E_{\mathrm{d,fi}}$ or shear modulus $G_{\mathrm{d,fi}}$) 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;

$k_{\mathrm{mod,fi}}$ is the modification factor for fire;

$\gamma_{\mathrm{M,fi}}$ 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 $k_{\mathrm{mod,fi}}$ are given in the relevant clauses.

NOTE 2: The recommended partial safety factor for material properties in fire is $\gamma_{\mathrm{M,fi}}$ = 1,0. Information on National choice may be found in the National annex..

(2)P The design value $R_{\mathrm{d,t,fi}}$ of a mechanical resistance (load-bearing capacity) shall be calculated as

$$R_{\mathrm{d,t,fi}} = \eta \frac{R_{20}}{\gamma_{\mathrm{M,fi}}} \tag{2.3}$$

where:

$R_{\mathrm{d,t,fi}}$ 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);

η 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 η is given in 6.2.2.1.

(3) The 20 % fractile of a strength or a stiffness property should be calculated as:

$$f_{20} = k_{fi}\, f_k \tag{2.4}$$

$$S_{20} = k_{fi}\, S_{05} \tag{2.5}$$

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

$$R_{20} = k_{fi}\, R_k \tag{2.6}$$

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:

$$E_{d,fi} \leq R_{d,t,fi} \tag{2.7}$$

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:

$$E_{d,fi} = \eta_{fi} E_d \tag{2.8}$$

where:

E_d is the design effect of actions for normal temperature design for the fundamental combination of actions, see EN 1990:2002;

η_{fi} is the reduction factor for the design load in the fire situation.

(3) The reduction factor η_{fi} for load combination (6.10) in EN 1990:2002 should be taken as

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{2.9}$$

or, for load combinations (6.10a) and (6.10b) in EN 1990:2002, as the smallest value given by the following two expressions

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{2.9a}$$

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\xi \gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{2.9b}$$

where:

$Q_{k,1}$ is the characteristic value of the leading variable action;

G_k is the characteristic value of the permanent action;

γ_G is the partial factor for permanent actions;

$\gamma_{Q,1}$ is the partial factor for variable action 1;

[AC2> ψ_{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; <AC2]

ξ is a reduction factor for unfavourable permanent actions G.

NOTE 1: An example of the variation of the reduction factor η_{fi} versus the load ratio $Q_{k,1}/G_k$ for different values of the combination factor ψ_{fi} according to expression (2.9) is shown in figure 2.1 with the following assumptions: γ_{GA} = 1,0, γ_G = 1,35 and γ_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.

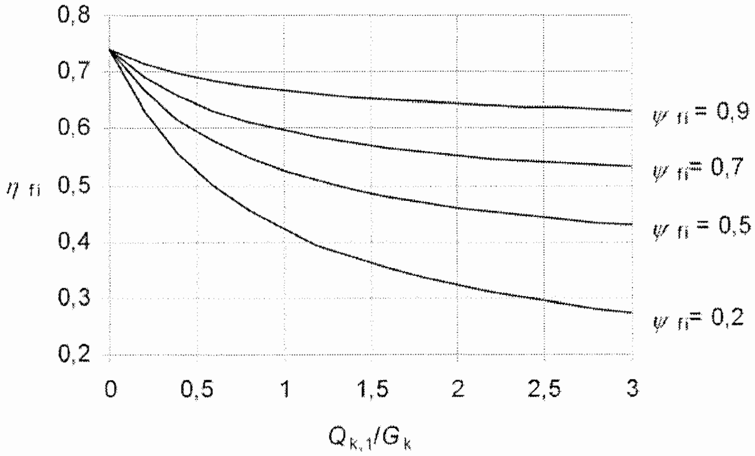


Figure 2.1 – Examples of reduction factor η_{fi} versus load ratio $Q_{k,1}/G_k$ according to expression (2.9)

NOTE 2: As a simplification, the recommended value is η_{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 η_{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:

$$d_{\mathrm{char},0} = \beta_0 \, t \tag{3.1}$$

where:

$d_{\mathrm{char},0}$ is the design charring depth for one-dimensional charring;

β_0 is the one-dimensional design charring rate under standard fire exposure;

t is the time of fire exposure.

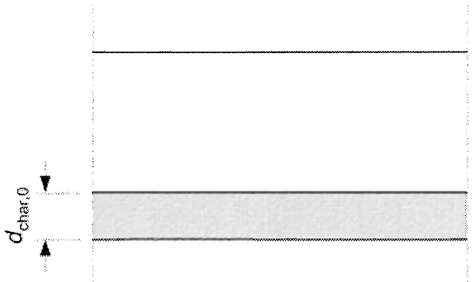


Figure 3.1 — One-dimensional charring of wide cross section (fire exposure on one side)

(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

$$d_{\mathrm{char},n} = \beta_n \, t \tag{3.2}$$

where:

$d_{\mathrm{char},n}$ is the notional design charring depth, which incorporates the effect of corner roundings;

β_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, $b_{\min}$, where

$$b_{\min} = \begin{cases} 2\, d_{\mathrm{char},0} + 80 & \text{for } d_{\mathrm{char},0} \geq 13 \text{ mm} \\ 8{,}15\, d_{\mathrm{char},0} & \text{for } d_{\mathrm{char},0} < 13 \text{ mm} \end{cases} \tag{3.3}$$

When the smallest width of the cross section is smaller than $b_{\min}$, 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 $d_{\mathrm{char},0}$.

AC2 (5) For surfaces of timber and wood-based materials, unprotected throughout the time of fire exposure, design charring rates β_0 and β_n are given in table 3.1. AC2

NOTE: For timber members in wall and floor assemblies where the cavities are completely filled with insulation, values for notional design charring rates β_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^3, 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.

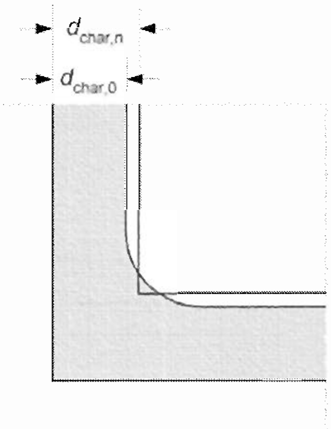


Figure 3.2 — Charring depth $d_{char,0}$ for one-dimensional charring and notional charring depth $d_{char,n}$

(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^3 and a panel thickness of 20 mm.

(9) For other characteristic densities ρ_k and panel thicknesses h_p smaller than 20 mm, the charring rate should be calculated as

$$\beta_{0,\rho,t} = \beta_0 \, k_\rho \, k_h \tag{3.4}$$

with

$$k_\rho = \sqrt{\frac{450}{\rho_k}} \tag{3.5}$$

$$k_h = \sqrt{\frac{20}{h_p}} \tag{3.6}$$

where:

ρ_k is the characteristic density, in kg/m^3;

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 β_0 and β_n of timber, LVL, wood panelling and wood-based panels

	β_0 mm/min	β_n mm/min
a) Softwood and beech		
Glued laminated timber with a characteristic density of ≥ 290 kg/m^3	0,65	0,7
Solid timber with a characteristic density of ≥ 290 kg/m^3	0,65	0,8
b) Hardwood		
Solid or glued laminated hardwood with a characteristic density of 290 kg/m^3	0,65	0,7
Solid or glued laminated hardwood with a characteristic density of ≥ 450 kg/m^3	0,50	0,55
c) LVL		
with a characteristic density of ≥ 480 kg/m^3	0,65	0,7
d) Panels		
Wood panelling	0,9[a]	–
Plywood	1,0[a]	–
Wood-based panels other than plywood	0,9[a]	–
[a] The values apply to a characteristic density of 450 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.

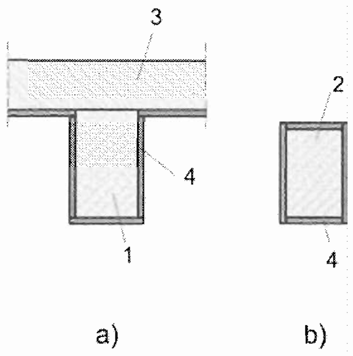
NOTE 3: The different stages of protection, the times of transition between stages and corresponding charring rates are illustrated in figures 3.4 to 3.6.

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 $t_f > t_{ch}$.

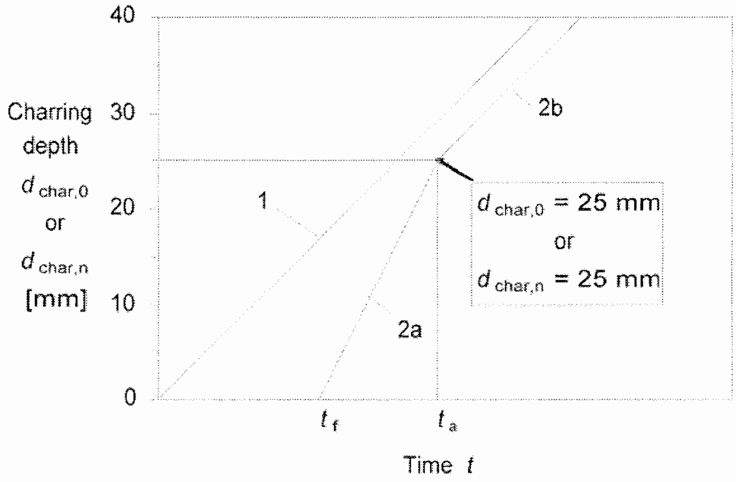
NOTE: Test methods are given in ENV 13381-7.

(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:

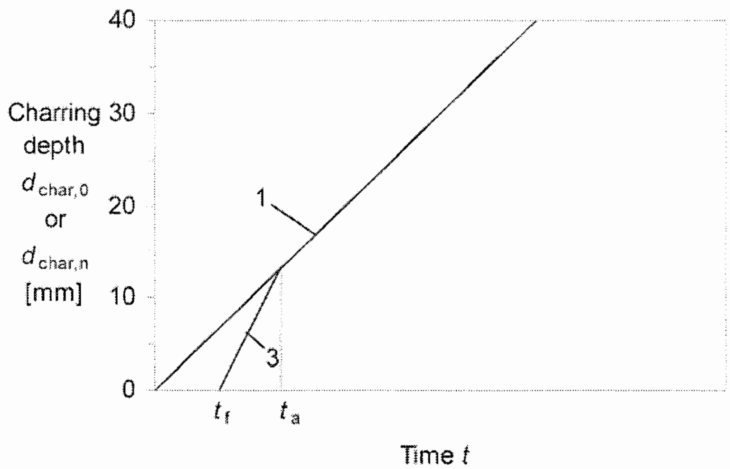
1 Relationship for members unprotected throughout the time of fire exposure for charring rate β_n (or β_0)

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

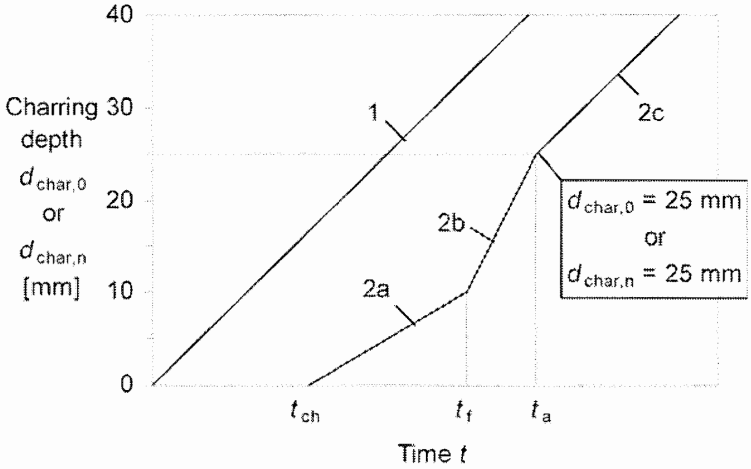


Key:

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 β_n (or β_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

$$k_2 = 1 - 0{,}018\, h_p \tag{3.7}$$

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 kg/m^3 which remain coherent up to 1000°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
≥ 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

$$t_a = \min \begin{cases} 2\,t_f & \text{(a)} \\ \dfrac{25}{k_3\,\beta_n} + t_f & \text{(b)} \end{cases} \qquad (3.8)$$

or for $t_{ch} < t_f$ (see figure 3.6)

$$t_a = \frac{25 - (t_f - t_{ch})\,k_2\beta_n}{k_3\,\beta_n} + t_f \qquad (3.9)$$

where β_n is the notional design charring rate, in mm/min. Expressions (3.8) and (3.9) also apply to one-dimensional charring when β_n is replaced by β_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

$$t_{ch} = \frac{h_p}{\beta_0} \qquad (3.10)$$

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

$$t_{ch} = 2{,}8\,h_p - 14 \qquad (3.11)$$

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

$$t_{ch} = 2{,}8\,h_p - 23 \qquad (3.12)$$

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 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

$$t_{ch} = 0{,}07\left(h_{ins} - 20\right)\sqrt{\rho_{ins}} \tag{3.13}$$

where:

t_{ch} is the time of start of charring in minutes;

h_{ins} is the thickness of the insulation material in millimetres;

ρ_{ins} is the density of the insulating material in kg/m^3.

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:

$$t_f = t_{ch} \tag{3.14}$$

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:

$$t_f = t_{ch} \tag{3.15}$$

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

$$l_{f,req} = h_p + d_{char,0} + l_a \tag{3.16}$$

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)

$$d_{ef} = d_{char,n} + k_0 d_0 \tag{4.1}$$

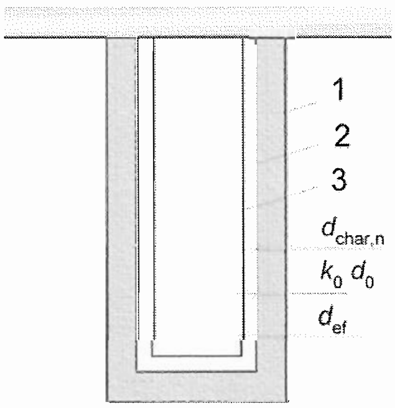
with:

d_0 = 7 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.

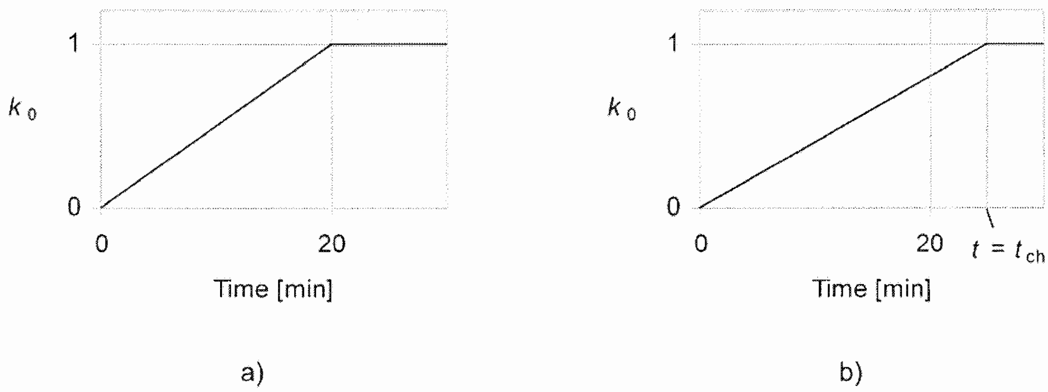


Figure 4.2 — Variation of k_0: a) for unprotected members and protected members where $t_{ch} \leq$ 20 minutes, b) for protected members where t_{ch} > 20 minutes

(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 ≥ 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:

$$k_{mod,fi} = 1{,}0 - \frac{1}{200}\frac{p}{A_r} \tag{4.2}$$

- for compressive strength:

$$k_{\text{mod,fi}} = 1{,}0 - \frac{1}{125}\frac{p}{A_r} \tag{4.3}$$

– for tensile strength and modulus of elasticity:

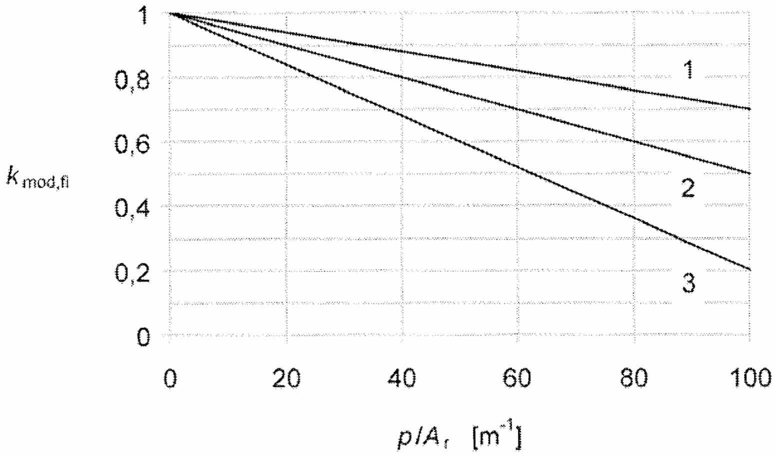
$$k_{\text{mod,fi}} = 1{,}0 - \frac{1}{330}\frac{p}{A_r} \tag{4.4}$$

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 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.

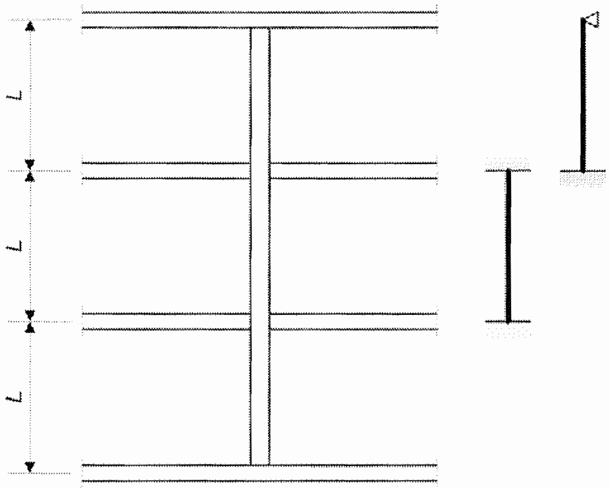


Figure 4.4 — Continuous column

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

$$K_{fi} = K_u \, \eta_f \qquad (4.5)$$

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;

η_f is a conversion factor according to table 4.2.

Table 4.2 — Conversion factor η_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**[a]
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
[a] 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:

$$a_{fi} = \beta_n \, k_{flux} \, (t_{req} - t_{d,fi}) \tag{6.1}$$

β_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.

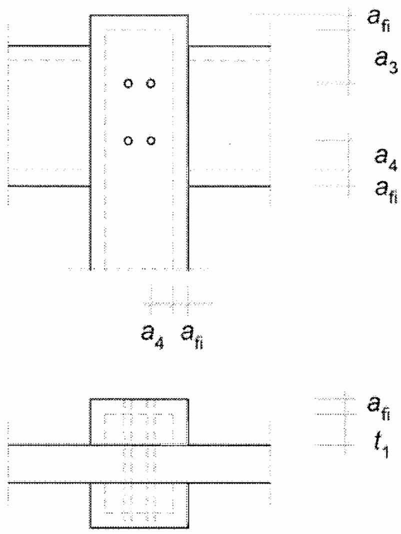


Figure 6.1 — Extra thickness and extra end and edge distances of connections

(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

$$t_{ch} \geq t_{req} - 0{,}5\, t_{d,fi} \tag{6.2}$$

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

$$t_{ch} \geq t_{req} - 1{,}2\, t_{d,fi} \tag{6.3}$$

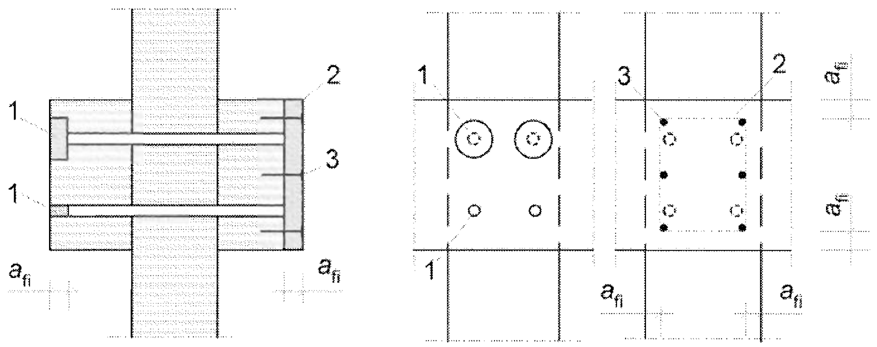
(3) For connections where the fasteners are protected by glued-in timber plugs, the length of the plugs should be determined according to expression (6.1), see figure 6.2.

(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.

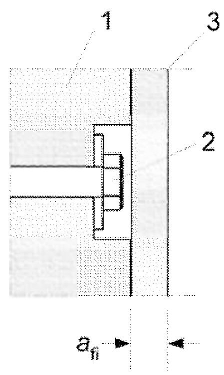
(6) The following rules apply for the fixing of additional protection by nails or screws:
- the distance between fasteners should be not more than 100 mm along the board edges and not more than 300 mm for internal fastenings;
- the edge distance of fasteners should be equal or greater than a_{fi} calculated 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 6*d* 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

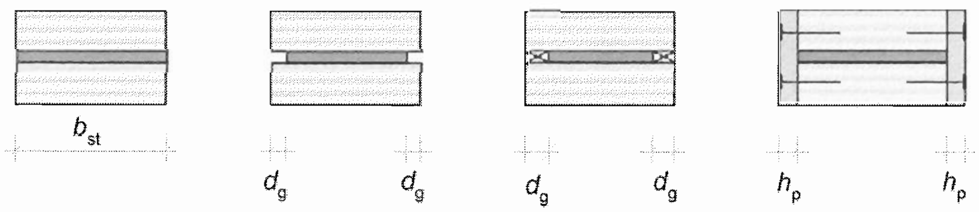
(1) For joints with internal steel plates of a thickness equal or greater than 2 mm, and which do not project beyond the timber surface, the width b_{st} of the steel plates should observe the conditions given in table 6.2.

Table 6.2 — Widths of steel plates with unprotected edges

		b_{st}
Unprotected edges in general	R 30	≥ 200 mm
	R 60	≥ 280 mm
Unprotected edges on one or two sides	R 30	≥120 mm
	R 60	≥ 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.



a) b) c) d)

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]

$$t_1 = \max\begin{cases}50\\50+1{,}25(d-12)\end{cases} \tag{6.4}$$

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

$$F_{v,Rk,fi} = \eta\, F_{v,Rk} \tag{6.5}$$

with

$$\eta = e^{-k\,t_{d,fi}} \tag{6.6}$$

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;

η 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:

$$t_{d,fi} = -\frac{1}{k} \ln \frac{\eta_{fi}\, \eta_0\, k_{mod}\, \gamma_{M,fi}}{\gamma_M\, k_{fi}} \tag{6.7}$$

where:

k is a parameter given in table 6.3;

η_{fi} is the reduction factor for the design load in the fire situation, see 2.4.2 (2);

η_0 is the degree of utilisation at normal temperature;

k_{mod} is the modification factor from EN 1995-1-1, subclause 3.1.3;

γ_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[a] with $d \geq 12$ mm	0,04	40
Dowels steel-to-wood[a] with $d \geq 12$ mm	0,085	30
Connectors in accordance with EN 912	0,065	30
[a] 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:

$$a_{fi} = \beta_n \left(t_{req} - t_{d,fi}\right) \tag{6.8}$$

where:

β_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} accordding 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 η for the reduction in the axial resistance of the screw in the fire situation should be calculated using expression (6.11):

$$a_2 \geq a_1 + 40 \tag{6.9}$$

$$a_3 \geq a_1 + 20 \tag{6.10}$$

where a_1, a_2 and a_3 are the distances, in millimetres.

$$\eta = \begin{cases} 0 & \text{for } a_1 \leq 0{,}6\ t_{fi,d} & \text{(a)} \\ \dfrac{0{,}44\ a_1 - 0{,}264\ t_{d,fi}}{0{,}2\ t_{d,fi} + 5} & \text{for } 0{,}6\ t_{d,fi} \leq a_1 \leq 0{,}8\ t_{d,fi} + 5 & \text{(b)} \\ \dfrac{0{,}56\ a_1 - 0{,}36 \cdot t_{d,fi} + 7{,}32}{0{,}2\ t_{d,fi} + 23} & \text{for } 0{,}8\ t_{d,fi} + 5 \leq a_1 \leq t_{d,fi} + 28 & \text{(c)} \\ 1{,}0 & \text{for } a_1 \geq t_{d,fi} + 28 & \text{(d)} \end{cases} \tag{6.11}$$

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 η 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 $t_{d,fi}$.

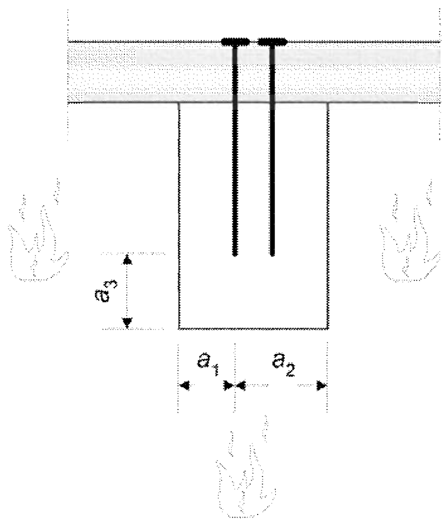


Figure 6.5 — Cross-section and definition of distances

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

$$t_{\mathrm{p,min}} = \max \begin{cases} \dfrac{l_{\mathrm{p}}}{70} \\ 8 \end{cases} \tag{7.1}$$

where:

$t_{\mathrm{p,min}}$ 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^3.

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.

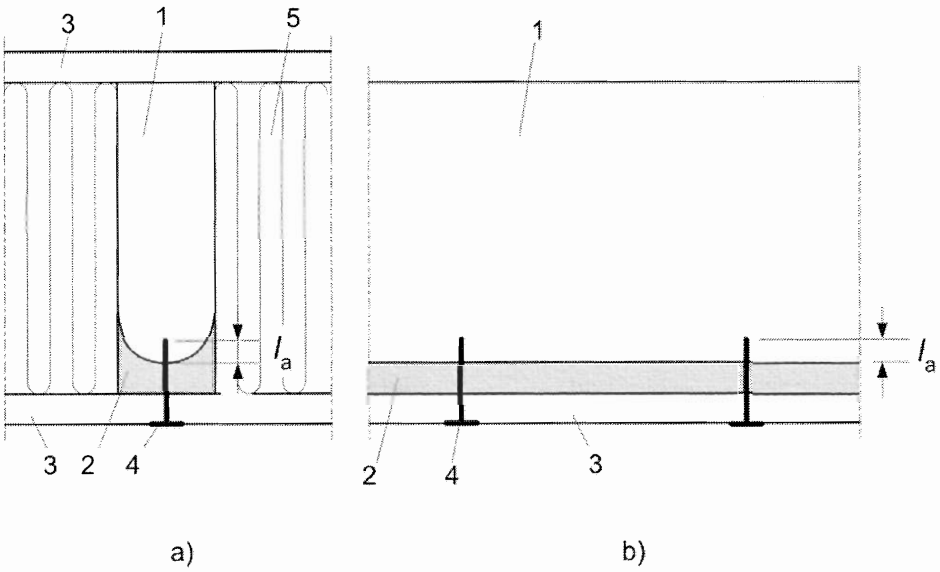
7.1.3 Insulation

(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

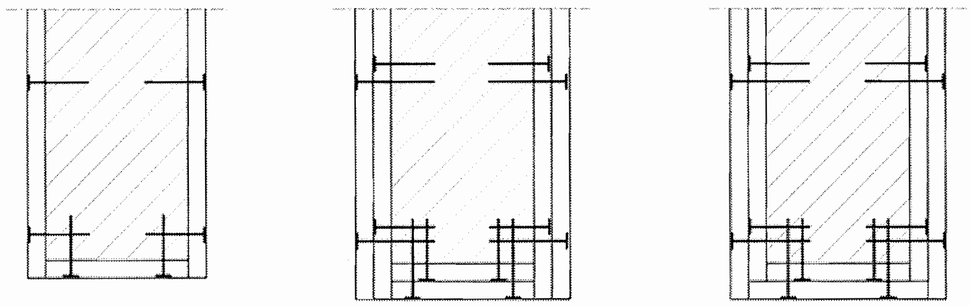


Figure 7.2 — Examples of fixing of fire protective panels to beams or columns

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 β and time t shown in figure A1 should be used. The charring rate β_{par} during the heating phase of a parametric fire curve is given by

$$\beta_{\text{par}} = 1{,}5\, \beta_{\text{n}} \frac{0{,}2\sqrt{\Gamma} - 0{,}04}{0{,}16\sqrt{\Gamma} + 0{,}08} \tag{A.1}$$

with

$$\Gamma = \frac{\left(\dfrac{O}{b}\right)^2}{\left(\dfrac{0{,}04}{1160}\right)^2} \tag{A.2}$$

$$O = \frac{A_{\text{v}}}{A_{\text{t}}}\sqrt{h_{\text{eq}}} \tag{A.3}$$

$$b = \sqrt{\rho c \lambda} \tag{A.4}$$

$$h_{\text{eq}} = \sum \frac{A_i\, h_i}{A} \tag{A.5}$$

where:

O is the opening factor, in $m^{0,5}$;

β_{n} is the notional design charring rate, in mm/min;

A_{v} is the total area of openings in vertical boundaries of the compartment (windows etc.), in m^2;

A_{t} is the total area of floors, walls and ceilings that enclose the fire compartment, in m^2;

A_{i} is the area of vertical opening "i", in m^2;

h_{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;

Γ 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;

λ is the thermal conductivity of the boundary of the compartment, in $Wm^{-1}K^{-1}$;

ρ is the density of the boundary of the compartment, in kg/m^3;

c is the specific heat of the boundary of the compartment, in $Jkg^{-1}K^{-1}$.

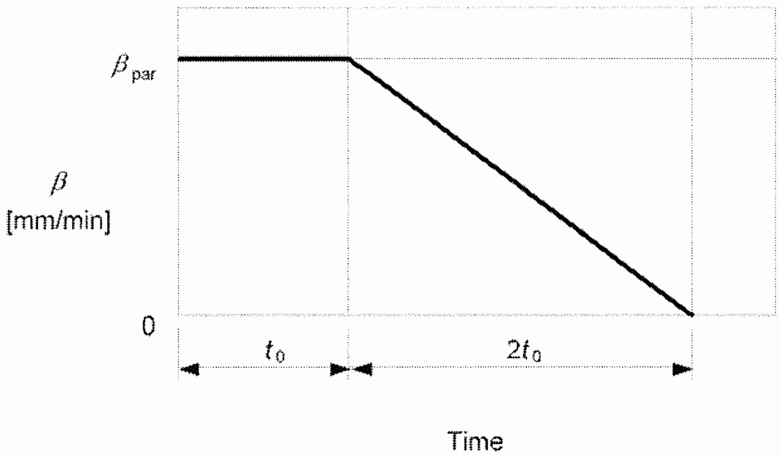


Figure A1 — Relationship between charring rate and time

(2) The charring depth should be taken as

AC2 $$d_{char} = \begin{cases} \beta_{par} t & \text{for } t \le t_0 & \text{(a)} \\ \beta_{par}\left(1{,}5t - \dfrac{t^2}{4t_0} - \dfrac{t_0}{4}\right) & \text{for } t_0 < t \le 3t_0 & \text{(b)} \\ 2\beta_{par} t_0 & \text{for } 3t_0 < t \le 5t_0 & \text{(c)} \end{cases} \quad \text{(A.6)}$$ 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 MJ/m^2, see EN 1991-1-2:2002.

The rules given in (1) and (2) should only be used for:

– $t_0 \le 40$ min

– $d_{char} \le \dfrac{b}{4}$

– $d_{char} \le \dfrac{h}{4}$

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

$$k_{\text{mod,fi}} = 1{,}0 - 3{,}2\,\frac{d_{\text{char,n}}}{b} \tag{A.8}$$

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

$$k_{\text{mod,fi}} = \frac{W_{\text{ef}}}{W_{\text{r}}} \tag{A.9}$$

where:

W_{ef} is the section modulus of the effective cross-section determined according to 4.2.2;

W_{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

AC2) 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. (AC2

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.

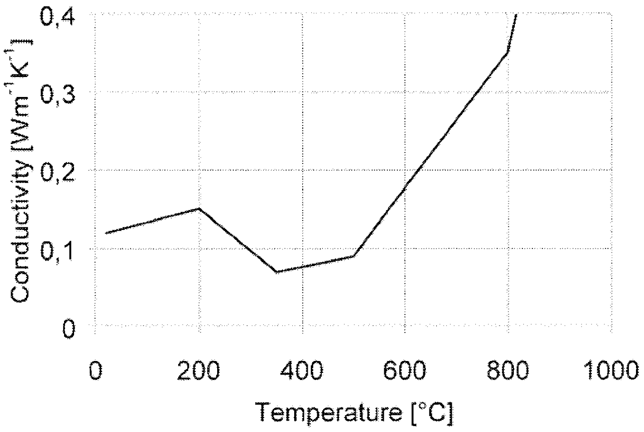


Figure B1 – Temperature-thermal conductivity relationship for wood and the char layer

Table B1 – Temperature-thermal conductivity relationship for wood and the char layer

Temperature °C	**Thermal conductivity** $Wm^{-1}K^{-1}$
20	0,12
200	0,15
350	0,07
500	0,09
800	0,35
1200	1,50

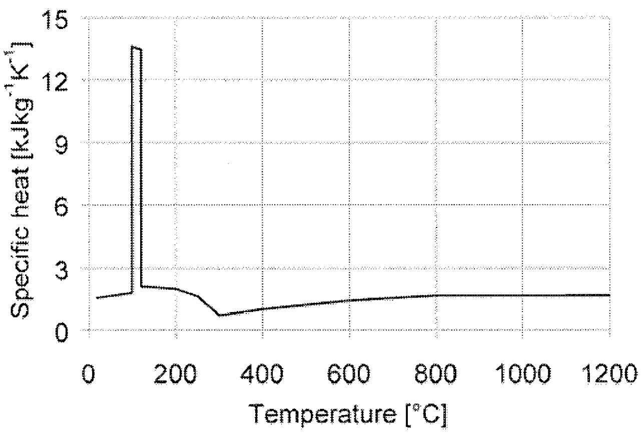


Figure B2 – Temperature-specific heat relationship for wood and charcoal

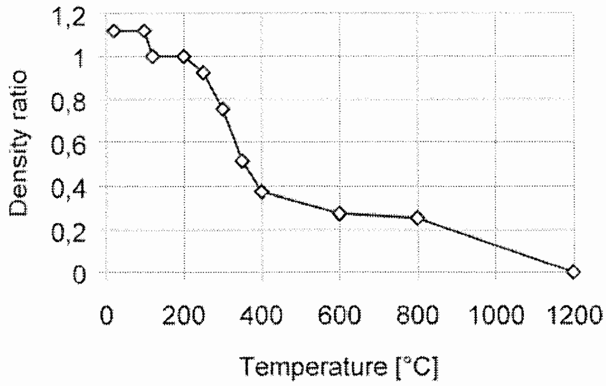


Figure B3 – Temperature-density ratio relationship for softwood with an initial moisture content of 12 %

[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^{-1} K^{-1}	**Ratio of density to dry density**[a]
20	1,53	1 + ω
99	1,77	1 + ω
99	13,60	1 + ω
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

[a] ω 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.

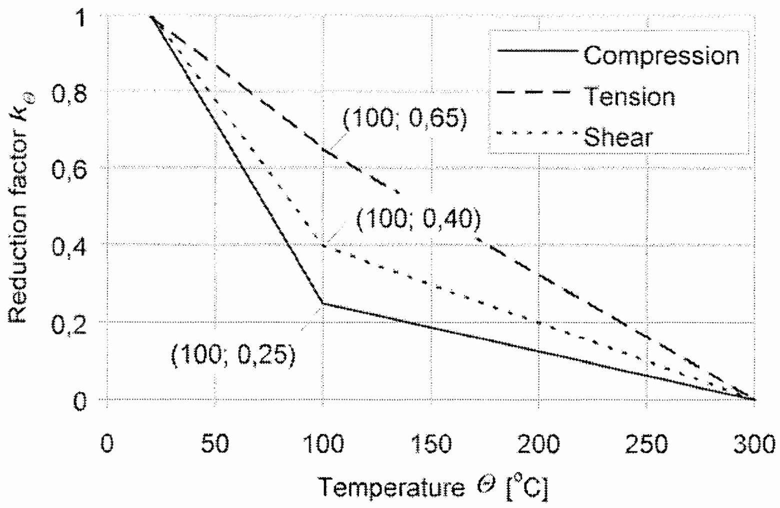


Figure B4 – Reduction factor for strength parallel to grain of softwood

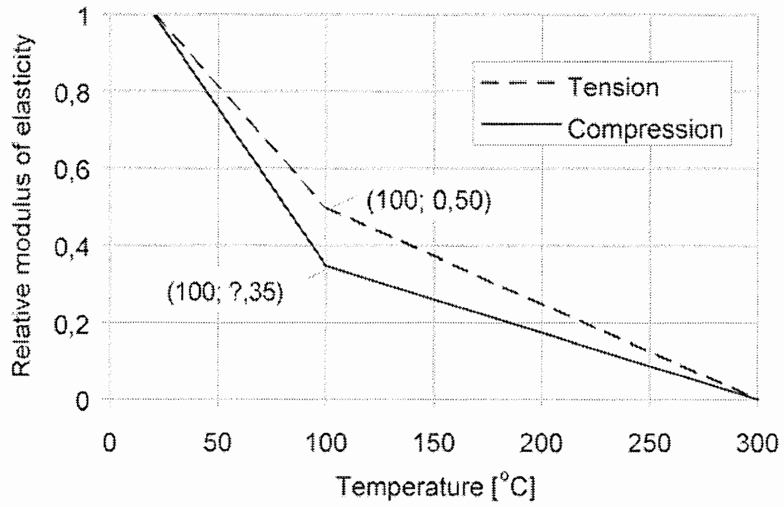


Figure B5 – Effect of temperature on modulus of elasticity parallel to grain of softwood

(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

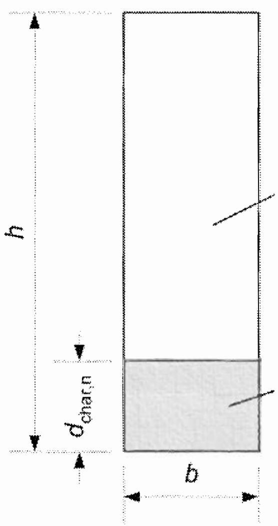
(1) This annex deals with the load-bearing function of timber frame wall and floor assemblies consisting of timber members (studs or joists) clad with panels on the fire-exposed side for a standard fire exposure of not more than 60 minutes. The following conditions apply:

- 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).



Key:
1 Notional residual cross-section
2 Notional char layer

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:

$$\beta_n = k_s \, k_2 \, k_n \, \beta_0 \qquad \text{for } t_{ch} \leq t \leq t_f \tag{C.1}$$

$$\beta_n = k_s \, k_3 \, k_n \, \beta_0 \qquad \text{for } t \geq t_f \tag{C.2}$$

where:

$k_n = 1{,}5$

β_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;
β_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

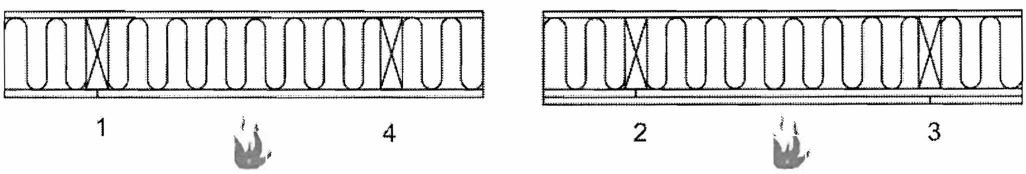
(4) For claddings made of gypsum plasterboard of type F, or a combination of type F and type A with type F as the outer layer, the insulation factor may be determined as:
– at locations where the cladding is unjointed, or for joint configuration 2, see figure C2:

$$k_2 = 1{,}05 - 0{,}0073\, h_p \quad \text{(C.3)}$$

– for joint configurations 1 and 3, see figure C2:

$$k_2 = 0{,}86 - 0{,}0037\, h_p \quad \text{(C.4)}$$

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

$$k_3 = 0{,}036\ t_f + 1 \qquad \text{(C.5)}$$

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:

$$t_{ch} = t_f \qquad \text{(C.6)}$$

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:

$$t_f = \frac{h_p}{\beta_0} - 4 \qquad \text{(C.7)}$$

where:

t_f is the failure time, in minutes;

h_p is the panel thickness, in millimetres;

β_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:

$$t_f = 2{,}8\ h_p - 14 \qquad \text{(C.8)}$$

(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

$$t_f = t_{ch} + \frac{l_f - l_{a,min} - h_p}{k_s\ k_2\ k_n\ k_j\ \beta_0} \qquad \text{(C.9)}$$

with

$$k_j = 1{,}0 \quad \text{for panels not jointed over the timber member} \qquad \text{(C.10)}$$

$$k_j = 1{,}15 \quad \text{for joint configurations 1 and 3} \qquad \text{(C.11)}$$

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);

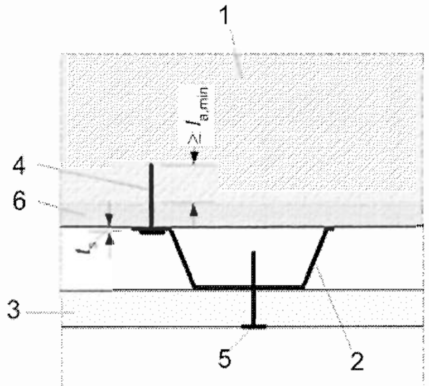
k_2 is the insulation factor, see C2.1(4);

k_n is a factor to convert the irregular residual cross-section into a notional rectangular cross-section, see C2.1(2);

β_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:

$$t_{sf} = t_f + \frac{l_f - l_{a,min} - k_s\, k_2\, k_n\, \beta_0 (t_f - t_{ch}) - t_s}{k_s\, k_3\, k_n\, \beta_0} \qquad \text{(C.12)}$$

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 ≤ 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

$$k_{\mathrm{mod,fm,fi}} = a_0 - a_1 \frac{d_{\mathrm{char,n}}}{h} \tag{C.13}$$

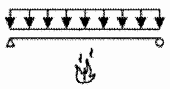
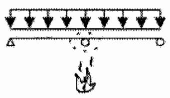
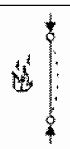
where:

a_0, a_1 are values given in table C2 and C3;

$d_{\mathrm{char,n}}$ is the notional charring depth according to expression (3.2) with β_{n} according to expression (C.1) and (C.2);

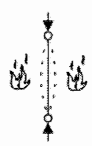
h is the depth of the joist or the stud.

Table C2 — Values[a] 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

[a] 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

$$k_{\text{mod,E,fi}} = b_0 - b_1 \frac{d_{\text{char,n}}}{h} \tag{C.14}$$

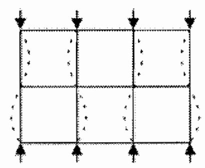
where:

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 β_n according to expression (C.1) and (C.2);

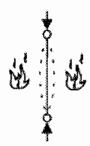
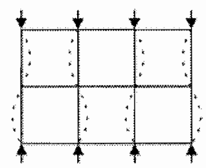
h is the depth of the joist.

Table C4 — Values[a] 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

[a] 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[a] 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

[a] 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:

$$t_{ch} = t_f \tag{D.1}$$

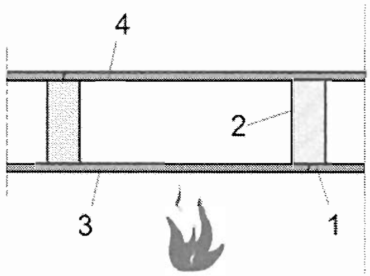
where t_f is determined according to D4(1).

(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:

$$t_{ch} = t_f \tag{D.2}$$

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

$$t_f = \frac{h_p}{\beta_0} - 4 \tag{D.3}$$

where:

t_f is the failure time, in minutes;

h_p is the panel thickness, in millimetres;

β_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:

$$t_f = 2{,}8\, h_p - 11 \qquad \text{(D.4)}$$

- for floors with the cladding fixed to timber members spaced more than 400 mm but not more than 600 mm:

$$t_f = 2{,}8\, h_p - 12 \qquad \text{(D.5)}$$

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

$$t_{\mathrm{ins}} \geq t_{\mathrm{req}} \tag{E.1}$$

where:

t_{ins} is the time taken for the temperature increases on the unexposed side given in 2.1.2(3) to occur;

t_{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_{ins} should be calculated as the sum of the contributions of the individual layers used in the construction, according to

$$t_{\mathrm{ins}} = \sum_{\mathrm{i}} t_{\mathrm{ins,0,i}}\, k_{\mathrm{pos}}\, k_{\mathrm{j}} \tag{E.2}$$

where:

$t_{\mathrm{ins,0,i}}$ is the basic insulation value of layer “i” in minutes, see E2.2;

k_{pos} is a position coefficient, see E2.3;

k_{j} is a joint coefficient, see E2.4.

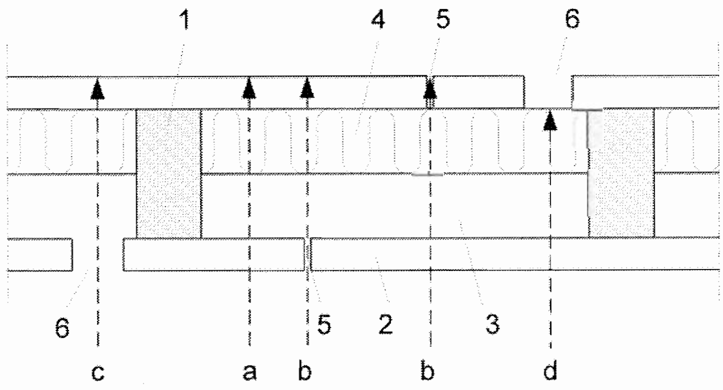
AC2 The relevant number of layers should be determined from table E1 and figure E1. AC2

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_{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 K	Heat transfer path according to figure E1
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^3

$$t_{\mathrm{ins},0} = 0{,}95\, h_{\mathrm{p}} \tag{E.3}$$

– for particleboard and fibreboard with a characteristic density greater than or equal to 600 kg/m^3

$$t_{\mathrm{ins},0} = 1{,}1\, h_{\mathrm{p}} \tag{E.4}$$

– for wood panelling with a characteristic density greater than or equal to 400 kg/m^3

$$t_{\mathrm{ins},0} = 0{,}5\, h_{\mathrm{p}} \tag{E.5}$$

– for gypsum plasterboard of type A, F, R and H

$$t_{ins,0} = 1,4\ h_p \tag{E.6}$$

where:

$t_{ins,0}$ 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

$$t_{ins,0,i} = 0,2\ h_{ins}\ k_{dens} \tag{E.7}$$

– for glass fibre

$$t_{ins,0,i} = 0,1\ h_{ins}\ k_{dens} \tag{E.8}$$

where:

h_{ins} is the insulation thickness in millimetres;

k_{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 $t_{ins,0}$ = 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:

$$k_{pos} = \min \begin{cases} 0,02\ h_p + 0,54 \\ 1 \end{cases} \tag{E.9}$$

$$k_{pos} = 0,07\ h_p - 0,17 \tag{E.10}$$

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_{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_{pos} = 1,5 for a void cavity, or a cavity filled with rock fibre insulation;
– k_{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 $t_{ins,0}$ 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_{dens} for cavity insulation materials

Cavity material	Density kg/m³	k_{dens} [a]
Glass fibre	15 20 26	0,9 1,0 1,2
Rock fibre	26 50	1,0 1,1
[a] For intermediate densities, linear interpolation may be applied		

AC2 **Table E3 — Position coefficients k_{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/m³	9 to 25	Expression (E.9)	0,8
Particleboard, fibreboard with characteristic density ≥ 600 kg/m³	9 to 25		
Wood panelling with characteristic density ≥ 400 kg/m³	15 to 19		
Gypsum plasterboard type A, H, F	9 to 15		

AC2

AC2 Table E4 — Position coefficients k_{pos} for single layered panels on the unexposed side

<table>
<tr><th rowspan="3">Panel on the exposed side</th><th rowspan="3">Thickness of panel on exposed side
mm</th><th colspan="5">Position coefficient for panels preceded by</th></tr>
<tr><th rowspan="2">Glass fibre</th><th colspan="3">Rock fibre of thickness[a]</th><th rowspan="2">Void</th></tr>
<tr><th>45 to 95</th><th>145</th><th>195</th></tr>
<tr><td>Plywood with density ≥ 450 kg/m³</td><td>9 to 25</td><td>Expression (E.10)</td><td rowspan="4">1,5</td><td rowspan="4">3,9</td><td rowspan="4">4,9</td><td>0,6</td></tr>
<tr><td>Particleboard and fibreboard with density ≥ 600 kg/m³</td><td>9 to 25</td><td>Expression (E.10)</td><td>0,6</td></tr>
<tr><td>Wood panelling with density ≥ 400 kg/m³</td><td>15
19</td><td>0,45
0,67</td><td>0,6</td></tr>
<tr><td>Gypsum plasterboard type A, H, F</td><td>9 to 15</td><td>Expression (E.10)</td><td>0,7</td></tr>
<tr><td colspan="7">[a] For intermediate values, linear interpolation may be applied.</td></tr>
</table>

AC2

Table E5 — Position coefficients k_{pos} for walls with double layered panels

Construction: Layer number and material		Layer number 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

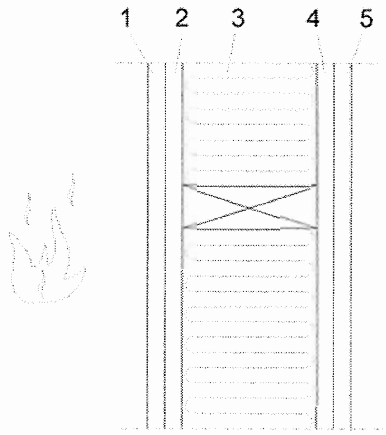


Figure E2 — Definition of layer numbers

Table E6 — Joint coefficient k_j to account for the effect of joints in wood-based panels which are not backed by battens

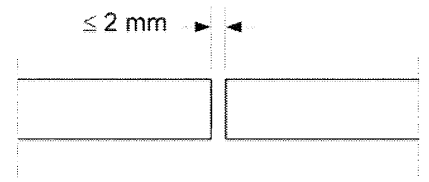
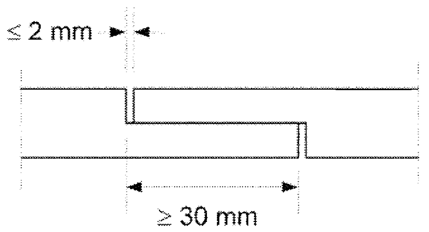
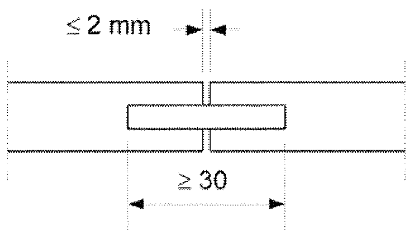
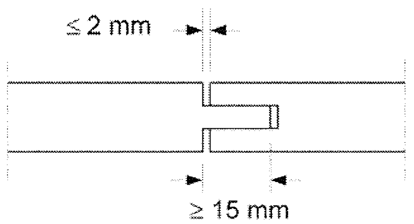
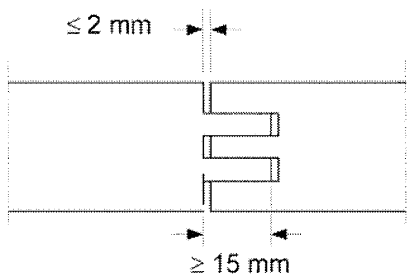
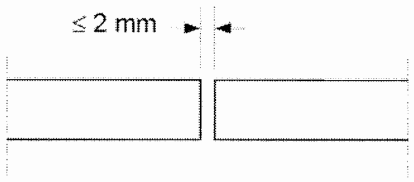
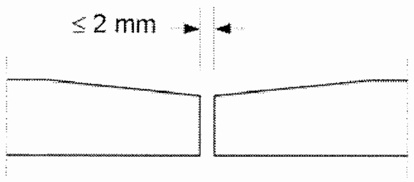
	Joint type	k_j
a		0,2
b		0,3
c		0,4
d		0,4
e		0,6

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.

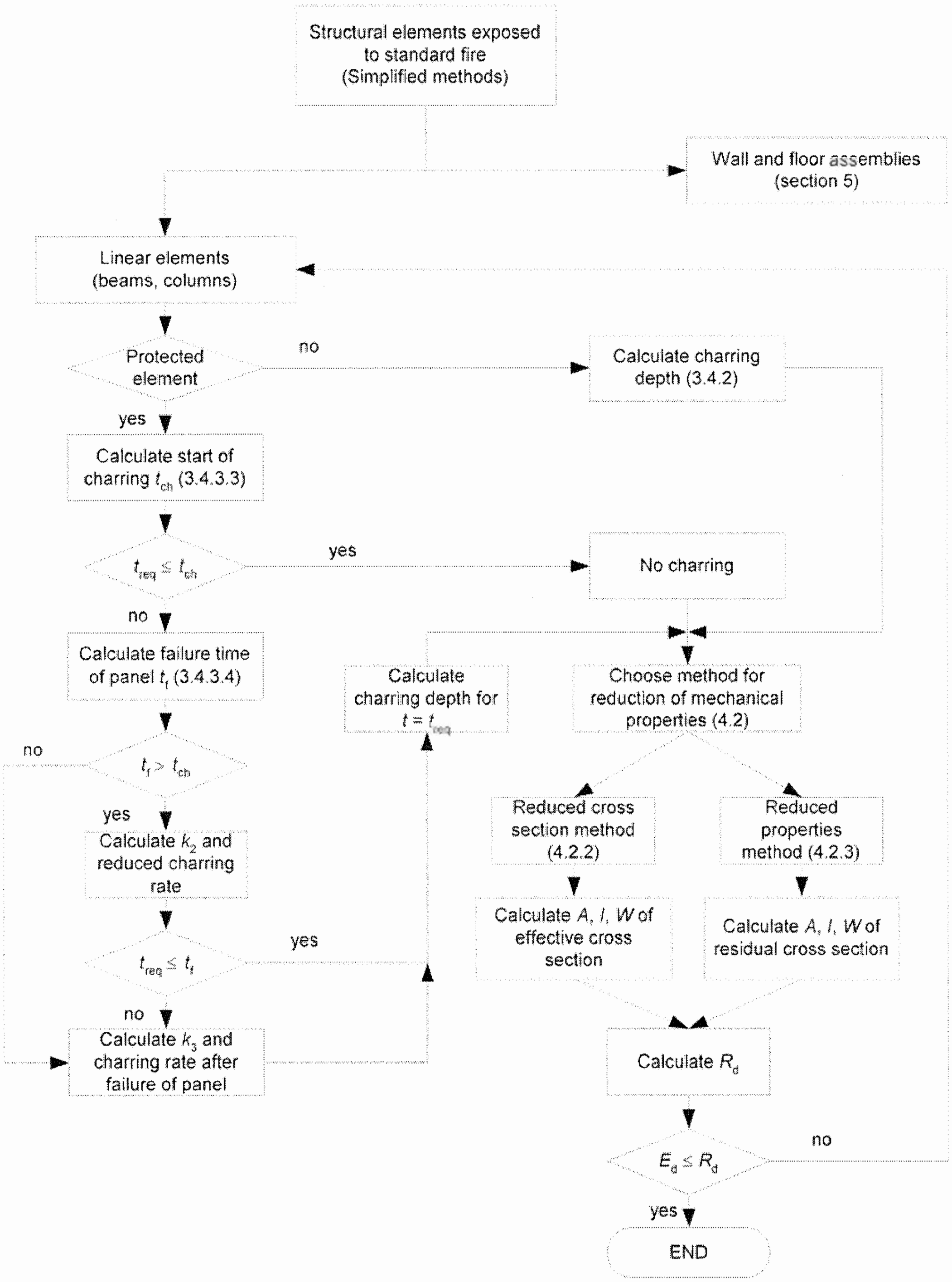


Figure F1 — Flow chart outlining the design procedure to check the load-bearing function of structural members

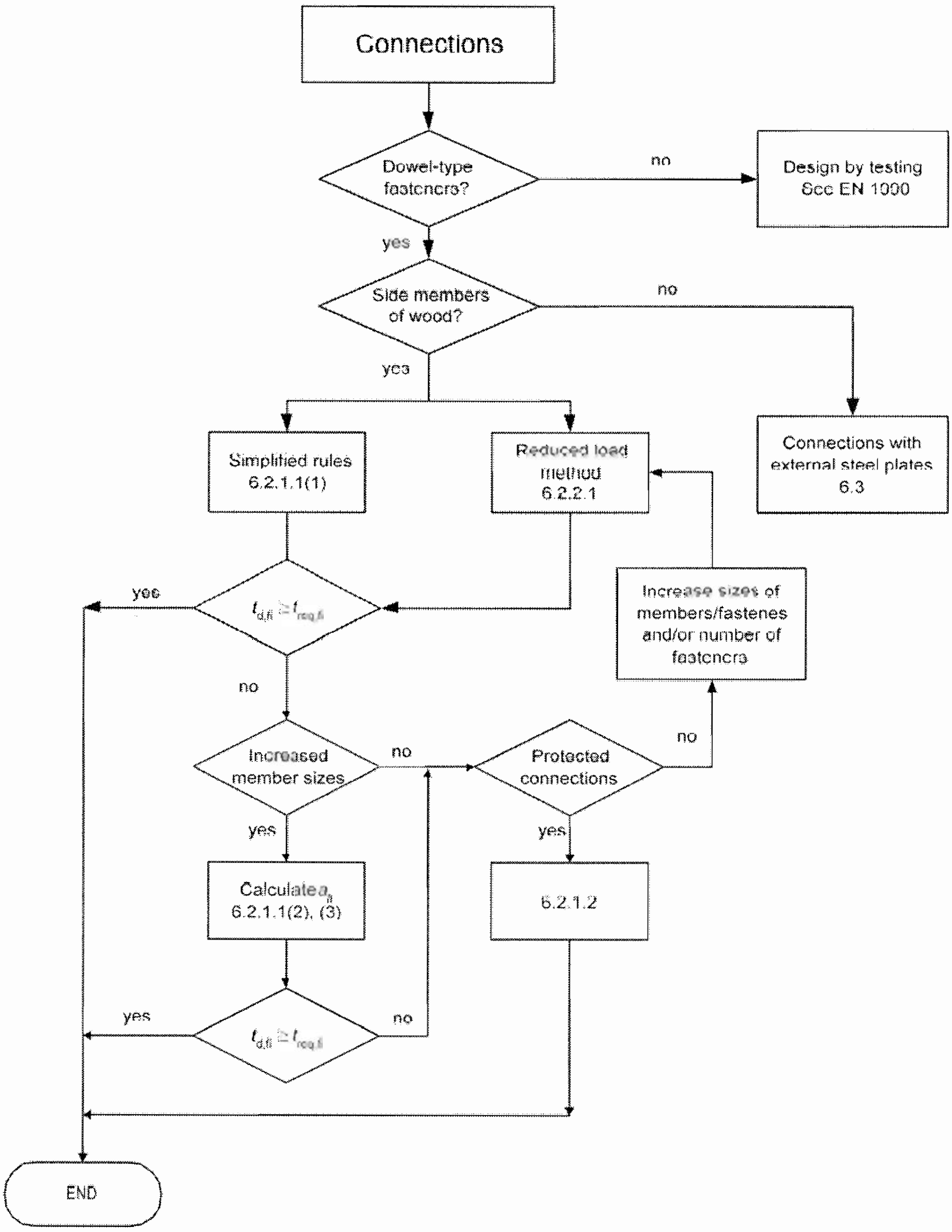


Figure F2 — Flow chart for the design procedure of connections