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Eurocode 5 – Design of timber structures

Part 1-2: General rules – Structural fire design

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Clean version

Contents

Foreword

This European Standard EN 1995-1-2, Design of timber structures – General rules – Structural fire design, has been prepared on behalf of Technical Committee CEN/TC250 " Structural Eurocodes", the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1995-1-2 on YYYY-MM-DD.

No existing European Standard is superseded.

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

[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).

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 a 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.

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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 :

a) 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 ;
b) 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. ;
c) 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.

It may also contain
– 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 199x-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 and 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 build 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 are 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 various possibilities for 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 relevant.

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.

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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

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 the competent authority.

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 call for specific periods of fire resistance, takes into account (though not explicitly), the features and uncertainties described above.

Application of this Part 1-2 of EN 1995 is illustrated below. The prescriptive and performance-based approach 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.

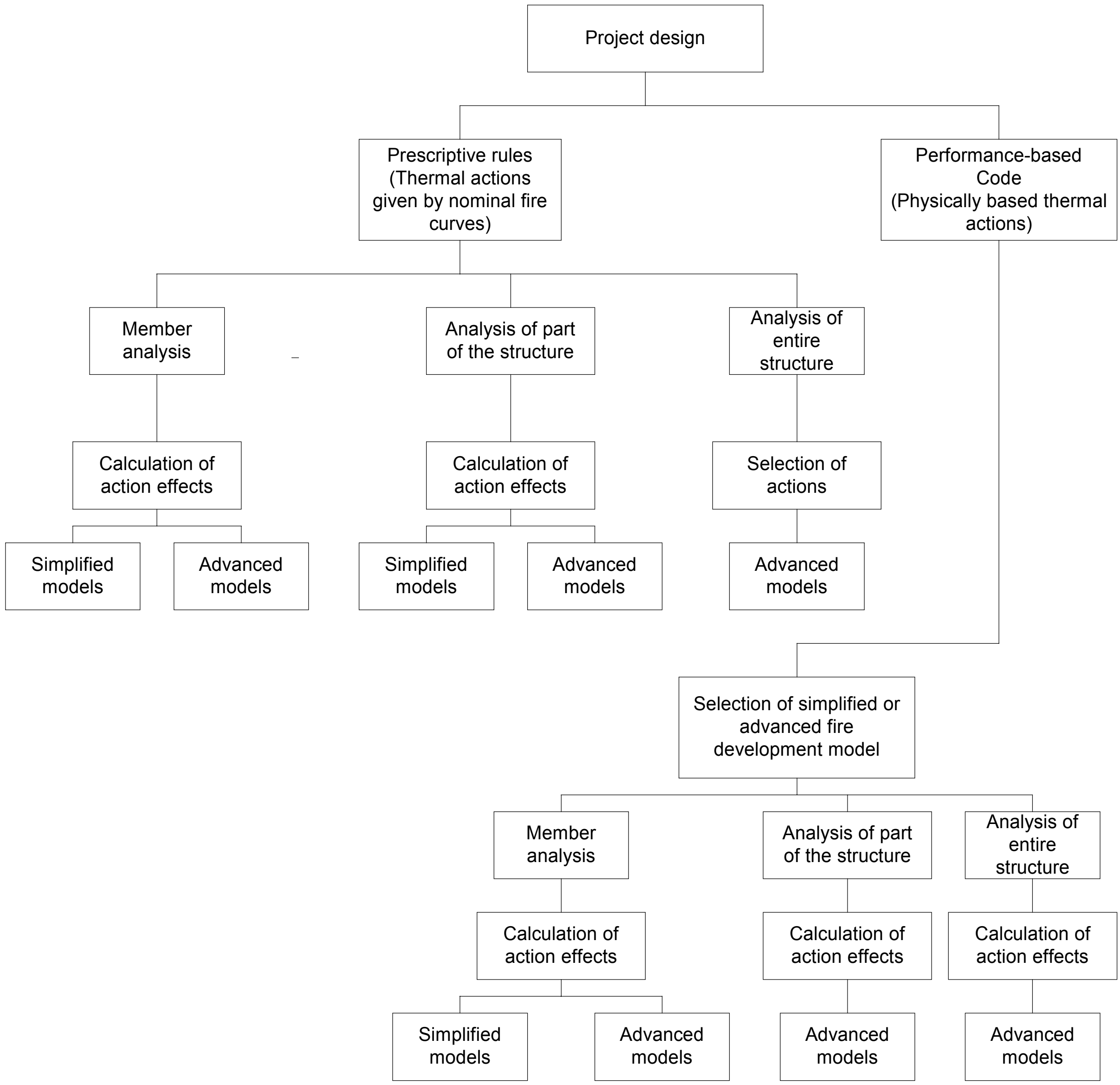

Figure – Design procedures

For design according to this part, EN 1991-1-2 is required for the determination of thermal and mechanical actions to the structure.

Design aids

It is expected, that design aids based on the calculation models given in ENV 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 for structural fire design of timber structures.

In an annex E (informative), guidance is given to help the user selecting relevant procedures for the design of timber structures.

National Annex for EN 1995-1-2

This standard gives alternative procedures, values and recommendations for classes 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:
2.3(1)P
2.3(2)
2.3(4)
2.4.2(3)
4.2.1(1)

Section 1 General

1.1 Scope

(1)P This Part 1-2 of EN 1995 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. This Part 1-2 of EN 1995 only identifies differences from, or supplements to, normal temperature design.

(2)P This Part 1-2 of EN 1995 deals only with passive methods of fire protection. Active methods are not covered.

(3)P This Part 1-2 of EN 1995 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 This Part 1-2 of EN 1995 gives principles and application rules for designing structures for specified requirements in respect of the aforementioned functions and levels of performance.

(5)P This Part 1-2 of EN 1995 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 this Part 1-2 of EN 1995 are applicable to all products covered by product standards made reference to in this Part.

1.2 Normative references

(1)P The following normative documents contain provisions which, through reference in this text, constitute provisions of this European Standard. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. However, parties to agreements based on this European Standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references, the latest edition of the normative document referred to applies.

EN 300	Oriented strand boards (OSB) – Definitions, classification and specifications
EN 301	Adhesives, phenolic and aminoplastic for load bearing timber structures; classification and performance requirements
EN 309	Particleboards – Definition and classification
EN 313-1	Plywood – Classification and terminology Part 1: Classification
EN 316	Wood fibreboards – Definition, classification and symbols
prEN 336	Structural timber – Coniferous and poplar – Sizes, permissible deviations
EN 338	Structural Timber – Strength classes
prEN 520	Gypsum plasterboards - Specifications - Test methods
EN 912	Timber fasteners – Specifications for connectors for timber
EN 1194	Glued laminated timber - Strength classes and determination of characteristic values
EN 1363-1	Fire resistance tests – 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	Eurocode: Basis of structural design
EN 1991-1-1	Eurocode 1 Actions on structures Part 1.1: General actions – Densities, self-weight and imposed loads
ENV 1991-1-2	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 rules – General rules and rules for buildings
EN 12 369–1	Wood-based panels – Characteristic values for structural design – Part 1: OSB, particleboards and fibreboards prENV 13381-7 Fire tests on elements of building construction – Test method for c
EN 13162	Thermal insulation products for buildings – factory made mineral wool (MW) products – Specifications M/103
prENV 13381-7	Test methods for determining the contribution to the fire resistance of structural members – Part 7: Applied protection to timber members
prEN 13986	Wood-based panels for use in construction - Characteristics, evaluation of conformity and marking
prEN 124-aaa	Timber structures – Structural laminated veneer lumber – Requirements

1.3 ASSUMPTIONS

(1) In addition to the general assumptions of EN 1990 it is assumed that any active fire protection measure taken into account in the design of the structure will be adequately maintained.

1.4 Distinction between principles and application rules

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

1.5 Definitions

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

(2)P The following terms are used in Part 1-2 of EN 1995 with the following meanings:

1.3.1
Char-line: Border line between the char-layer and the residual cross section

1.3.2
Effective cross section: Cross section of the member in structural fire design used in the effective cross-section method. It is obtained from the residual cross section by removing parts of the cross section with assumed zero strength and stiffness

1.3.3
Failure time of protection: Duration of protection against direct fire exposure; that is the time when the fire protective cladding or other protection falls off the timber member, a

structural member initially protecting the member fails due to collapse, or the protection from other structural member is terminated due to excessive deformation

1.3.4
Fire protection material: Any material or combination of materials applied to a structural member or element for the purpose of increasing its fire resistance.

1.3.5
Normal temperature design: Ultimate limit state design for ambient temperatures according to ENV 1995-1-1

1.3.6
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;

1.3.7
Residual cross section: Cross section of the original member reduced with the charring depth;

1.3.8
Resistance ratio in the fire situation: The ratio of the characteristic resistance of a member or a connection in the fire situation and the corresponding characteristic resistance at normal temperature.

1.6 Symbols

For the purpose of this Part 1-2 of EN 1995, the following symbols apply:

Latin upper case letters

A	Total area of vertical openings of fire compartment
A_r	Area of the residual cross
A_t	Total area of floors, walls and ceilings that enclose the fire compartment
E_{20}	20 % fractile of modulus of elasticity at normal temperature
$E_{0,05}$	Characteristic value of modulus of elasticity (5 % fractile)
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 the connection for the fire situation
F_{Rk}	Characteristic mechanic resistance of the connection at normal temperature without the effect of load duration and moisture (k_{mod} = 1)
$F_{R,20}$	20 % fractile of a resistance
K_{fi}	Slip modulus in the fire situation
K_u	Slip modulus for the ultimata limit state at normal temperature
O	Opening factor
$Q_{k,1}$	Characteristic value of leading variable action 1
G_k	Characteristic value of permanent action
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_{fi}	Extra thickness of member for improved mechanic resistance of connections

b	Width
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
d	Depth
h_{eq}	Weighted average of heights of all vertical openings
h_{ins}	Insulation thickness
h_p	Fire protective panel thickness
k	Parameter
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_n	Notional cross section coefficient
k_{mod}	Modification factor
$k_{mod,fi}$	Modification factor for fire
$k_{mod,fi,E}$	Modification factor for modulus of elasticity in the fire situation
$k_{mod,fi,fm}$	Modification factor for bending strength in the fire situation
k_{pos}	Position coefficient
k_ρ	Density coefficient
k_Θ	Temperature dependent reduction factor for local strength or stiffness property
l_p	Span of the panel
l_a	Anchorage length of fastener
$l_{a,min}$	Minimum anchorage length of fastener
l_f	Length of fastener
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_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_f	Failure time of protection
$t_{fi,d}$	Time of the fire resistance of the unprotected connection
t_{ins}	Time of temperature increase on the unexposed
$t_{ins,0,i}$	Basic insulation value of layer “i”
t_j	
$t_{p,min}$	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

Θ	Temperature

Greek lower case letters

β_0	Basic charring rate for one-dimensional charring
β_n	Notional charring rate
β_{par}	Charring rate during heating phase of parametric fire curve
η_{conn}	Conversion factor for the reduction of the load-bearing capacity in fire
η_f	Conversion coefficient

γ_{GA}	Partial factor for permanent actions in accidental design situations
γ_M	Partial factor for a material property, also accounting for for model uncertainties and dimensional variations
$\gamma_{M,fi}$	Partial factor for timber in fire
$\gamma_{Q,1}$	Partial factor for variable action 1
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 in the fire situation

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[6] Drafting note: 1.5 Units deleted as decided by Coordination Group

Section 2 Basic principles and rules

2.1 Performance requirements

2.1.1 General

(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 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, i.e.

– integrity failure does not occur;
– insulation failure does not occur;.
– thermal radiation from the unexposed side is limited.

NOTE: There is no risk of fire spread due to radiation with a unexposed surface temperature 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) Deformation criteria need not be applied where the efficiency of the means of protection has been verified by tests.

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 function: criteria R, E and, when requested, I

(2) For criterion R the load bearing function should be maintained during the required time of standard fire exposure.

(3) For criterion I the average temperature rise over the whole of the non-exposed surface should be limited to 140 K, and the maximum temperature rise at any point of that surface should not exceed 180 K.

2.1.3 Parametric fire exposure

(1) The load-bearing function should be maintained during the complete endurance of the fire including the decay phase, or a specified period of time.

(2) For the verification of the separating function the following applies:
– 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 gas temperature in the fire compartment is reached;

– the average temperature rise of the unexposed side of the construction should be limited to 180 K and the maximum temperature rise of the unexposed side should not exceed 240 K during the decay phase or for a required period of time;

assuming that the normal temperature is 20°C.

2.2 Actions

(1)P Thermal and mechanical actions shall be taken from EN 1991-1-2.

(2) For surfaces of wood, wood-based materials and gypsum plasterboard the emissivity coefficient should be taken equal to 0,8.

2.3 Design values of material properties and resistances

(1)P For verification of mechanical resistance, the design strengthand stiffness parameters shall be determined from

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

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

where

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

$E_{d,fi}$ is the design stiffness parameter (modulus of elasticity or shear modulus) in fire;

f_{20} is the 20 % fractile of strength at normal temperature;

E_{20} is the 20 % fractile of modulus of elasticity at normal temperature;

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

$\gamma_{M,fi}$ is the partial safety factor for timber in fire.

NOTE 1: The modification factor for fire takes into account the reduction of strength and stiffness parameters at elevated temperatures. The modification factor for fire replaces the modification factor for normal temperature design k_{mod} given in EN 1995-1-1. Values of $k_{mod,fi}$ are given in the relevant clauses.

NOTE 2: The recommended partial safety factor for mechanical material properties is $\gamma_{M,fi} = 1,0$. The choice of the value is to be made by at the national level. Information about the values to be used in the country of application may be given in a National Informative Annex to this European Standard.

(2) The design mechanical resistance of connections with fasteners in shear should be calculated as

$$F_{Rd,fi} = \eta_{conn} \frac{F_{R20}}{\gamma_{M,fi}} \tag{2.3}$$

where

$F_{Rd,fi}$ is the design mechanical resistance of connections in the fire situation at time t;

F_{R20} is the 20 % fractile value of the mechanical resistance of connections at normal temperature without the effect of load duration and moisture ($k_{mod} = 1$);

η_{conn} is a conversion factor, for standard fire exposure given in 6.2.2.1;

$\gamma_{M,fi}$ is the partial safety factor for timber in fire.

Note: See (2) Note 2

(3) The 20 % fractiles of strength and modulus of elasticity may be calculated as

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

$$E_{20} = k_{fi}\, E_{0,05} \tag{2.5}$$

where k_{fi} should be taken from 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 side members of wood and wood-based panels	1,15
Connections with side members of steel	1,05

(4) The 20 % fractiles of the mechanical resistance of connections should be calculated as

$$F_{R,20} = k_{fi}\, F_{R,k} \tag{2.6}$$

where

k_{fi} is given in table 2.1.

$F_{r,k}$ is the characteristic mechanic resistance of connections 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 Assessment 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:

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

where

$E_{fi,d}$ is the design effect of actions for the fire situation, determined in accordance with EN 1991-1-2, including effects of thermal expansions and deformations

$R_{fi,t,d}$ is the corresponding design resistance in the fire situation.

– (3) The analysis for the fire situation should be carried out according to EN 1990 5.1.4(2).

NOTE: A member analysis is performed as an equivalent to standard fire testing of elements or members.

(4)P The effect of thermal expansions of materials other than timber shall be taken into account.

(5) Where application rules given in this Part 1-2 of EN 1995 are valid only for the standard temperature-time curve, this is identified in the relevant clauses.

(5) 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 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 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

$$E_{d,fi} = \eta_{fi}\, E_d \qquad (2.8)$$

where

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

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 should be taken as

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

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

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

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

where

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

G_k is the characteristic value of a permanent action;

γ_G is the partial factor for permanent actions;

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

ψ_{fi} is the combination factor for frequent values of variable actions, see EN 1991-1-2

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. Expressions (2.9a) and (2.9b) give slightly higher values.

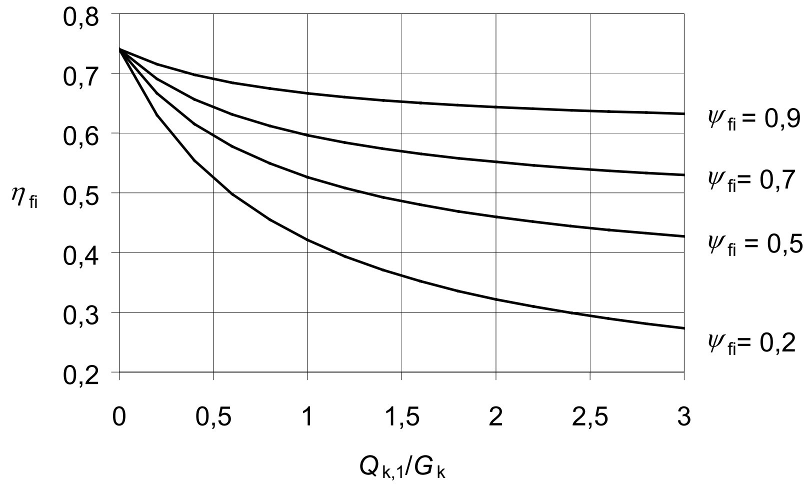


Figure 2.1 – Examples of reduction factor η_{fi} versus load ratio $Q_{k,1}/G_k$ according to expression (2.9)

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 (areas susceptible to accumulation to goods, including access areas) where the recommended value is η_{fi} = 0,7. The recommended values may be altered in the National annex.

(4) The boundary conditions at supports may be assumed as 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 a global structural analysis for normal temperature as given in 2.4.2(2)-(3).

(3) The part of the structures 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) 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) should be taken into account.

(5) The boundary conditions at supports and forces and moments at boundaries of part of the structure may be assumed as 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 Mechanical properties

(1) Simplified methods for the reduction of the strength and stiffness parameters of the cross section is 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 insulated wall and floor assemblies 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).

3.2 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).

3.3 Charring

3.3.1 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 protected surfaces, where charring of the wood occurs during the relevant time of fire exposure.

(2) The charring depth should be calculated as the position of the char-line taking into account the time of fire exposure and the relevant charring rate.

(3)The calculation of cross section properties should be based on the actual char 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
- initially unprotected surfaces;
- protected surfaces prior to failure of the protection;
- surfaces directly exposed to fire after failure of the protection.

(5) The rules of subclauses 3.3.2 and 3.3.3 apply to standard fire exposure.

NOTE: For parametric fire exposure, see annex A (informative).

3.3.2 Unprotected surfaces

(1) The charring rate for one-dimensional charring should be taken as constant with time andthe design charring depth should be calculated as (see figure 3.1)

$$d_{\text{char,0}} = \beta_0 \, t \quad (3.1)$$

where

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

β_0 is the basic design charring rate for one-dimensional charring;

t is the relevant time of fire exposure.

(2) The notional charring rate including the effect of corner roundings should be taken as constant with time and the notional design charring depth should be calculated as

$$d_{\text{char,n}} = \beta_{\text{n}} \, t \quad (3.2)$$

where

$d_{\text{char,n}}$ is the notional design charring depth, including the effect of corner roundings;

β_{n} is the notional design charring rate, including the effect of corner roundings and fissures;

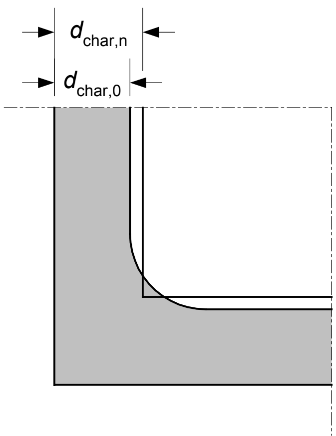


Figure 3.1 — Charring depth $d_{\text{char,0}}$ for one-dimensional charring and notional charring depth $d_{\text{char,n}}$

(3) For unprotected surfaces of timber design charring rates β_0 and β_{n} are given in table 3.1. The charring rates of table 3.1 apply for timber cross sections with

- a minimum residual thickness of 40 mm when charring takes place on both sides in direction of the thickness
- a minimum residual thickness of 20 mm when charring takes place on one side in direction of the thickness

For smaller residual thicknesses the charring rates should be increased by 50 percent.

(4) For solid hardwood with characteristic densities between 290 and 450 kg/m^3, in table 3.1 intermediate values may be obtained by linear interpolation. Charring rates of beech should be taken as given for solid softwood.

(5) For unprotected surfaces of LVL according to prEN 13986 and prEN124-aaa, design charring rates β_0 and β_n are given in table 3.1. Clause 3.3.2(3) applies with respect to minimum thicknesses of the residual cross section.

(6) When applying the basic charring rate, the shape of the char-line at corners should be assumed as circular with a radius equal to the charring depth. This is valid for radii not greater than $b_r/2$ or $h_r/2$, whichever is the smallest, where b_r and h_r are the width and depth of the residual cross section respectively.

(7) For wood panelling, wood-based panels according to EN 309, EN 313-1, EN 300 and EN 316, charring rates 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.

(8) For other characteristic densities ρ_k and thicknesses h_p of panels the charring rate should be calculated as

$$\beta_{0,\rho,t} = \beta_0 \, k_\rho \, k_h \tag{3.3}$$

with

$$k_\rho = \sqrt{\frac{450}{\rho_k}} \tag{3.4}$$

$$k_h = \max\begin{cases} \sqrt{\dfrac{20}{h_p}} \\ 1{,}0 \end{cases} \tag{3.5}$$

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 prEN 12 369.

Table 3.1 – Design charring rates β_0 and β_n of timber, LVL, wood panelling and wood-based panels

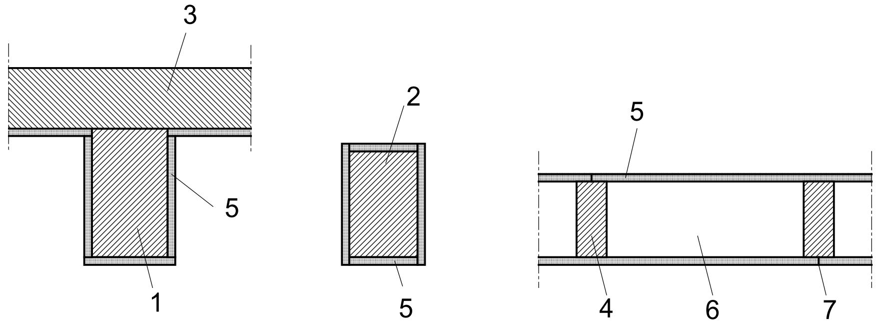
	β_0	β_n
	mm/min	mm/min
a) Softwood and beech		
Glued laminated timber with a characteristic density of ≥290 kg/m^3	0,65	0,7
Solid timber with a characteristic density of ≥290 kg/m^3	0,65	0,8
b) Hardwood		
Solid or glued laminated hardwood with a characteristic density of ≥290 kg/m^3	0,65	0,7
Solid[7] or glued laminated hardwood with a characteristic density of ≥450 kg/m^3	0,50	0,55
c) LVL with a characteristic density of ≥ 500 kg/m^3	0,65	0,7
d) Panels[a]		
Wood panelling	0,9	–
Plywood	1,0	–
Wood-based panels other than plywood	0,9	–
[a] The values apply to a characteristic density of 450 kg/m^3 and a panel thickness of 20 mm.		

3.3.3 Protected surfaces

(1) For surfaces protected by fire protective claddings, see figure 3.1, other protection materials or by other structural members, it should be taken into account that
– the start of charring is delayed until time t_{ch};
– the charring rate is reduced until failure time t_f of the fire protection;
– the charring rate may be increased after failure time t_f of the fire protection.

NOTE 1: Other fire protection are available such as 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 members;
– excessive deformations of the protecting member.



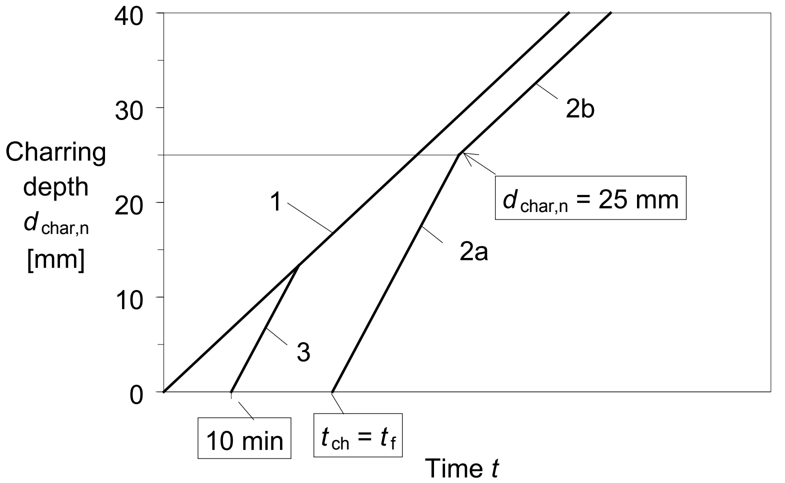
Key

1 beam
2 column
3 deck
4 stud or joist
5 cladding
6 void cavity
7 joint

Figure 3.1 — Examples of panels used as fire protective claddings

(2) For protected surfaces with failure times t_f of the protection smaller than 10 minutes, the effect of the protection should be disregarded, see figure 3.2.

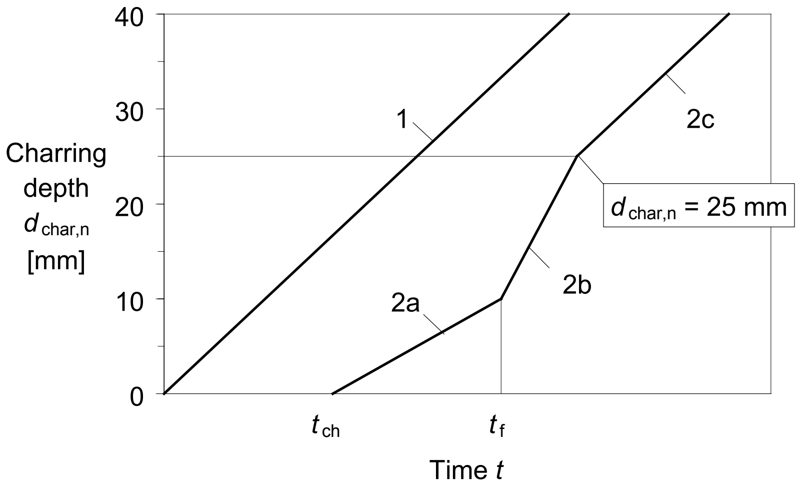
(3) For failure times t_f of the protection of 10 minutes or more, for the stage immediately after failure of the protection, the charring rates of table 3.1 should be multiplied by 2 until a charring depth $d_{char,n}$ of 25 mm is reached or is equal to the charring depth of an unprotected surface, whichever is the smallest. Thereafter the charring rates of table 3.1 should be used, see figure 3.2 and 3.3.



Key

1 Relationship for unprotected members for charring rate β_n
2 Relationship for protected members after failure of the fire protection
  2a After the fire protection has fallen off and charring starts at double rate
  2b After char depth exceeds 25 mm charring rate reduces to β_n
3 Relationship for protected members with failure of fire protection after 10 minutes

Figure 3.2 — Illustration of charring depth vs. time for $t_{ch} = t_f$



Key

1 Relationship for unprotected members for charring rate β_n
2 Relationship for 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 and charring starts at double rate
  2c After char depth exceeds 25 mm charring rate reduces to β_n

Figure 3.3 — Illustration of charring depth vs. time for $t_{ch} < t_f$ and $t_f \geq 10$ minutes

(4) The effect of joints of the cladding for unfilled gaps greater than 2 mm on the start of charring and, where relevant, on the charring rate before failure of the protection should be taken into account.

(5) 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: A test method is given in prENV 13381-7.

(6) For fire protective claddings of wood panelling and wood-based panels, the failure time should be determined as

$$t_f = \frac{h_p}{\beta_0} - 4 \qquad (3.6)$$

where

t_f is the failure time in minutes;

β_0 is the basic charring rate of the panel according to table 3.1 in mm/minute;

h_p is the total cladding thicknes of all layers in millimetres.

For wood-based panels and wood panelling, it may be assumed that charring of the protected timber member starts at the failure time of the panel, i.e. $t_{ch} = t_f$.

(7) For claddings consisting of one layer of gypsum plasterboard of type A, F or H according to prEN 520, at locations remote from panel joints, or adjacent to filled or unfilled gaps with a width of 2 mm or less, the time of start of charring may be taken as

$$t_{ch} = 2{,}8\ h_p - 14 \qquad (3.7)$$

where h_p is the total thickness of panels in mm.

At locations adjacent to joints with unfilled gaps with a width of more than 2 mm, the time of start of charring should be calculated as

$$t_{ch} = 2{,}8\ h_p - 23 \qquad (3.8)$$

NOTE: Gypsum plasterboard type E, D, R and I according to prEN 520 have equal or better thermal and mechanical properties than type A and H.

(8) For claddings consisting two layers of gypsum plasterboard where both layers remain in place and will both fail simultaneously, at locations remote from panel joints in the outer layer the time of start of charring may be taken according to expression (3.7), where h_p is the total thickness of panels in mm.

At locations adjacent to joints in the outer layer, the time of start of charring should be calculated according to expression (3.8).

NOTE: For example, when the outer layer is of type F and the inner layer of type A or H, both layers will normally fall off simultaneously.

(9) For claddings consisting two layers where the layers fall off separately, expressions (3.7) and (3.8) are not valid.

NOTE: Where two layers of gypsum plasterboard type A or H are used, both layers will normally fall off at different times.

(10) Failure times of gypsum plasterboard due to mechanical degradation of the material should be determined by testing. For type A and H the failure time t_f should be taken as $t_f = t_{ch}$.

NOTE 1: Test methods are given in EN 1363-1, EN 1365-1, EN 1365-2 and prENV 13381-7.

NOTE 2: 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.

NOTE 3: The failure time depends also on the length of fasteners, providing anchorage in unburned timber. Design rules are given in annex C (informative).

(11) For timber protected by a single layer of gypsum plasterboard type F, for $t_{ch} \leq t \leq t_f$ the charring rates according to table 3.1 should be multiplied by

$$k_2 = 1 - 0{,}018\, h_p \tag{3.9}$$

where

h_p is the layer thickness in millimetres.

Expression (3.9) applies also for two layers of gypsum plasterboard, where the outer layer is type F and the inner layer is type A or H.

NOTE: For members in wall and floor assemblies, expressions are given in annex C (informative).

(12) For beams or columns protected by rock fibre batts with a thickness of more than 20 mm and a density of more than 26 kg/m^3 which remain coherent up to 1000°C the protection time may be taken as

$$t_{ch} = 0{,}07\left(h_{ins} - 20\right)\sqrt{\rho_{ins}} \tag{3.10}$$

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 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 according to type 1 adhesive according to EN 301 and adhesive for plywood and LVL according to EN 314 should be used.

(3) For glued-in rods, the softening temperature of the adhesive should be determined by tests.

Section 4 Design procedures for mechanical resistance

4.1 General

(1) The rules of EN 1995-1-1 apply with cross sectional properties determined according to 4.2 and 4.3.

4.2 Simplified rules for cross sectional resistance

4.2.1 General

(1) The cross-sectional resistance may either be determined by the rules given in 4.2.2, or, alternatively, given in 4.2.3.

NOTE: The National choice may be given 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 (see figure 4.1 line 3)

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

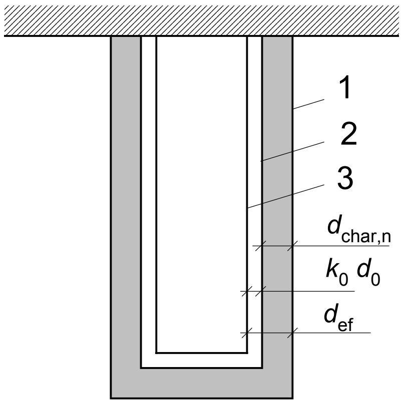
with

d_0 = 7 mm

$d_{char,n}$ according to expression (3.2) or calculated according to the rules given in 3.3.3

k_0 according to table 4.1 and (3), see figure 4.2a.

NOTE: It is assumed that the reduction of strength and stiffness properties of the material close to the char line is allocated to the layer of thickness k_0 d_0, while the strength and stiffness properties of the remaining effective cross section are assumed to be unreduced.



Key

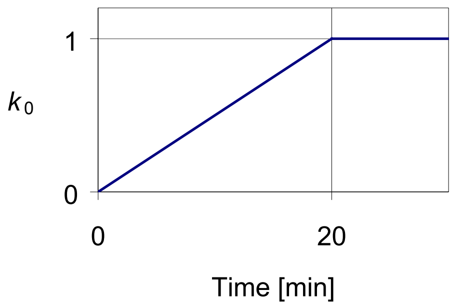
1 Initial surface of member
2 Border of residual cross section
3 Border of effective cross section

Figure 4.1 — Definition of residual cross section and effective cross section

Table 4.1 — Determination of k_0 for unprotected surfaces with t in minutes (see figure 4.1a)

	k_0
t < 20 minutes	t/20
$t \geq$ 20 minutes	1,0

(2) For protected surfaces with t_{ch} > 20 minutes or t_f > 20 minutes, it should be assumed that k_0 varies linearly from 0 to 1 during the time interval from t = 0 to $t = t_{ch}$ or $t = t_f$, whichever is the smallest, see figure 4.2b. For protected surfaces with $t_{ch} \leq$ 20 minutes or $t_f \leq$ 20 minutes table 4.1 applies.



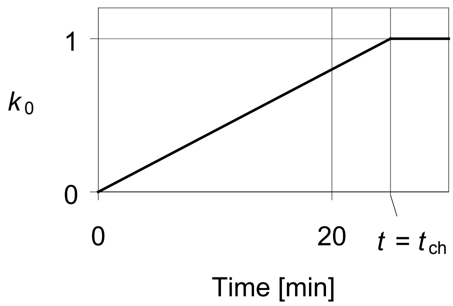


Figure 4.2 — Variation of k_0: a) for unprotected members, b) for protected members (shown for $t_{ch} < t_f$)

(3) The design strength and modulus of elasticity respectively of the effective cross section should be taken according to expressions (2.1)-(2.2) with $k_{mod,fi}$ = 1,0

4.2.3 Reduced properties method

(1) The following rules should be applied to rectangular cross sections of softwood exposed to fire on three or four sides and round cross sections exposed along its whole perimeter.

(2) The residual cross section should be determined according to 3.3.

(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:

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

– for compressive strength:

$$k_{mod,fi} = 1{,}0 - \frac{1}{125}\frac{p}{A_r} \qquad (4.3)$$

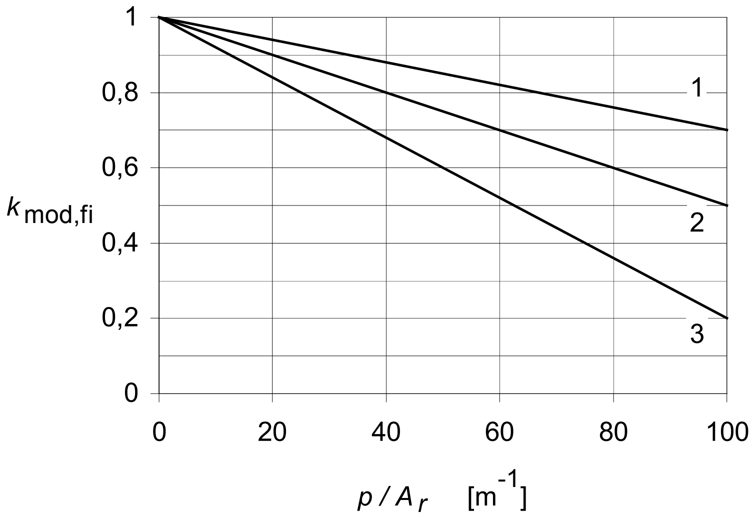
– for tensile strength and modulus of elasticity:

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

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 $0 \le t \le 20$ min 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 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, lateral buckling should be considered as for an unbraced member.

4.3.3 Columns

(1) Where bracing fails during the relevant fire exposure, buckling should be considered as for an unbraced member.

(2) More favourable boundary conditions compared to 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 completely fixed at both ends, in the top storey the column may be assumed as completely fixed at its lower end, see figure 4.4. The column length should be taken as the system length L of the storey.

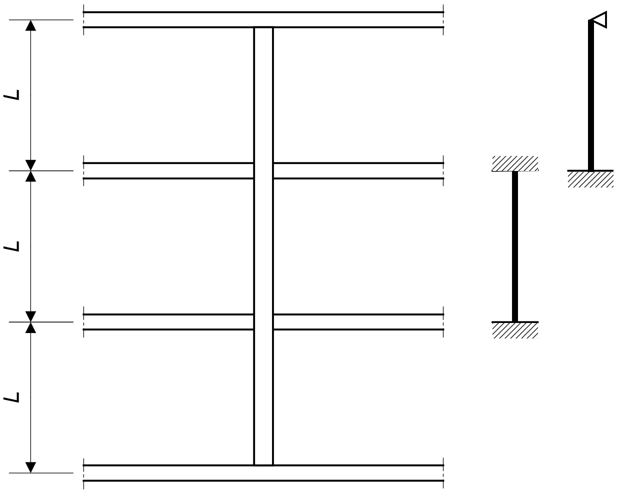


Figure 4.4 — Continuous column

4.3.4 Mechanically jointed members

(1)P For mechanically jointed members, the reduction of 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

$$K_{fi} = K_u \, \eta_f \tag{4.5}$$

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 coefficient according to table 4.2.

Table 4.2 — Conversion factor η_f

Nails	0,2
Bolts, dowels, 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) The bracing may be assumed not to fail if the residual width and area is 60 % of its initial width and area that are required with respect to normal temperature design, and is fixed with nails, screws, dowels or bolts.

4.4 Advanced calculation methods

4.4.1 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.

4.4.2 Thermal response

(1) Advanced calculation methods for thermal response should be based on the theory of heat transfer.

(2) The thermal response model should take into account:

– the variation of the thermal properties of the material with the 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.

(4) The influence of any moisture content of wood and of protection made of gypsum plasterboard should be taken into account.

4.4.3 Structural response

(1) General calculation methods should take into account the changes of mechanical properties with temperature and, where relevant, also of moisture.

(2) 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.

(3) For materials other than timber or wood-based materials, the effects of thermally induced strains and stresses both due to temperature rise and due to temperature gradients, should be taken into account.

(4) The structural response model should take into account the effects of non-linear material properties.

Section 5 Design procedures for wall and floor assemblies

5.1 General

(1) The rules in this subclause apply to load bearing (R), separating (EI), and load bearing and separating (REI) constructions. For the separating function the rules apply for a maximum standard fire resistance not more than 60 minutes.

5.2 Analysis of load bearing function

(1) For assemblies with void cavities, the rules of section 3 and 4 should be used.

NOTE: A design method for wall and floor assemblies with insulation in the cavities is given in annex C (informative)

(2)P Non-separating load-bearing constructions shall be assumed to be exposed to fire on both sides at the same time.

(3) Where wood-based panels or wood panelling are used for stiffening or bracing the load bearing timber frame, they should have a residual thickness of at least 60 % of the thickness required for normal temperature design; else the frame should be analysed as unbraced, see 4.3.5.

5.3 Analysis of separating function

5.3.1 General

(1)P The fixing of the panel on the unexposed side of the assembly shall be secured into unburnt timber.

(2) The centre-line of the fastener should be at least at a distance of 5 mm from the char-line.

(3) Requirements with respect to insulation (criterion I) are assumed to be satisfied provided that detailing is carried out according to subclause 7.1.

(4) Requirements with respect to integrity (criterion E) are assumed to be satisfied where the requirements with respect to insulation (criterion I) are satisfied provided that detailing is carried out according to subclause 7.1. It should also be ensured, that panels remain fixed to the timber frame on the unexposed side.

(5) 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 prEN 520. For other materials, integrity should be determined by testing.

NOTE: See Note 1 of 3.3.3(7).

(6) For separating members it should be verified that

$$t_{ins} \geq t_{req} \tag{5.1}$$

where

t_{ins} is the time to reach the temperature increase on the unexposed side given in 2.1.2(3);
t_{req} is the required time of fire resistance for the fire separating function of the assembly.

5.3.2 Simplified method for the analysis of insulation

5.3.2.1 General

(1) The value of t_{ins} may be calculated as the sum of contributions of the individual layers used in the construction, according to

$$t_{ins} = \sum_{i} t_{ins,0,i}\, k_{pos}\, k_{j} \tag{5.2}$$

where

$t_{ins,0,i}$ is the basic insulation value of layer “i” in minutes, see 5.3.2.2;

k_{pos} is a position coefficient, see 5.3.2.2;

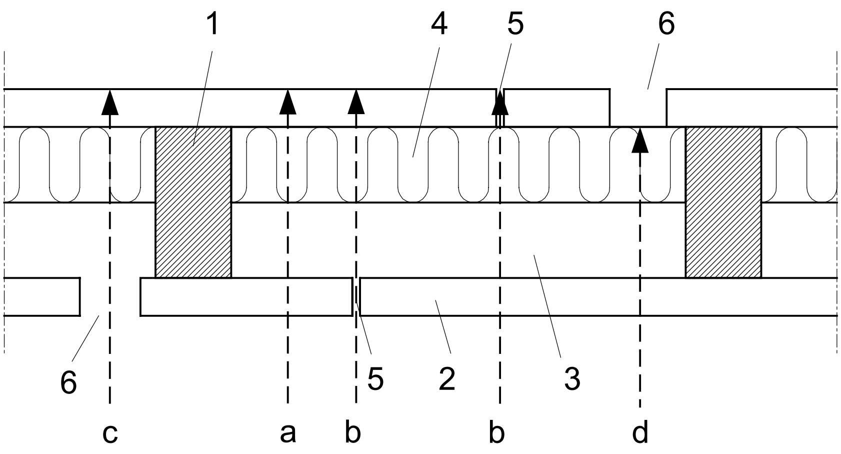
k_{j} is a joint coefficient, see 5.3.2.2(8) - (10).

The relevant number of layers should be taken according to table 5.1 and figure 5.2.

NOTE: A joint does not have an effect on the performance if it is backed with a batten or a structural element, which will prevent the travel of hot gases into the structure.

Table 5.1 — Heat transfer path through layer to be taken into account

	Temperature rise on unexposed side °C	**Heat transfer path according to figure 5.1**
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 5.1 — Illustration of heat transfer paths through separating construction

5.3.2.2 Basic insulation values, position coefficients and effect of joints

(1) The values given in this subclause may be applied for verification of fire resistance times up to 60 minutes.

(2) Basic insulation values of panels should be determined from the following expressions:
– for plywood with a characteristic density of 450 kg/m^3

$$t_{\text{ins},0} = 0{,}95\ h_{\text{p}} - 0{,}3\ t_{\text{ins},0} = 0{,}95\ h_{\text{p}} \tag{5.3}$$

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

$$t_{\text{ins},0} = 1{,}1\ h_{\text{p}} + 0{,}4\ t_{\text{ins},0} = 1{,}1\ h_{\text{p}} \tag{5.4}$$

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

$$t_{\text{ins},0} = 0{,}5\ h_{\text{p}} + 0{,}2\ t_{\text{ins},0} = 0{,}5\ h_{\text{p}} \tag{5.5}$$

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

$$t_{\text{ins},0} = 1{,}4\ h_{\text{p}} + 0{,}4\ t_{\text{ins},0} = 1{,}4\ h_{\text{p}} \tag{5.6}$$

where
$t_{\text{ins},0}$ is the basic insulation value in minutes
h_{p} is the panel thickness in millimetres.

(3) Where cavities are partially or completely filled with insulation made of glass or rock fibre, basic values of the insulation should be determined as:
– for rock fibre

$$t_{ins,0,i} = 0{,}2\, h_{ins}\, k_{dens} \tag{5.7}$$

– for glass fibre

$$t_{ins,0,i} = 0{,}1\, h_{ins}\, k_{dens} \tag{5.8}$$

where
h_{ins} is the insulation thickness in millimetres
k_{dens} should be taken from table 5.2.

(4) For void cavities of depth between 45 and 200 mm the basic insulation value should be taken as $t_{ins,0}$ = 5,0 min.

(5) For walls with single layered claddings, position coefficients for panels on the exposed side of walls should be taken from table 5.3, and for panels on the unexposed side of walls from table 5.4, with following expressions:

$$k_{pos} = \min \begin{cases} 0{,}02\, h_p + 0{,}54 \\ 1 \end{cases} \tag{5.9}$$

$$k_{pos} = 0{,}07\, h_p - 0{,}17 \tag{5.10}$$

The position coefficients for voids and insulation layers is 1,0.

(6) For walls with double layered claddings, see figure 5.2, position coefficients should be taken from table 5.5.

(7) For floors exposed from below, the position coefficients for the exposed panels given in tables 5.3 and 5.5 should be multiplied by 0,8.

(8) The joint coefficient k_j should be taken as

$$k_j = 1 \tag{6.11}$$

for the following:
— panel joints fixed to a battens of at least the same thickness or 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 (5.5).

(9) For panel joints not fixed to a batten, the joint coefficient k_j should be taken from tables 5.6 and 5.7.

(10) For butt jointed insulation batts or insulation batts with a density of greater than 30 kg/m^3 butted against the timber frame member, the joint coefficient may be taken as k_j = 1, otherwise it should be taken as k_j= 0,5.

Table 5.2 — Values of k_{dens} for cavity insulation materials

Cavity material	Density kg/m^3	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		

Table 5.3 — Position coefficients k_{pos} for single layered panels on the exposed side

Panel	Density kg/m^3	Thickness mm	Position coefficient for panels backed by	
			rock or glass fibre	void
Plywood	≥ 450	9 - 25	Expression (5.9)	0,8
Particleboard, fibreboard	≥ 600	9 - 25	Expression (5.9)	0,8
Wood panelling	≥ 400	15 - 19	Expression (5.9)	0,8 0,8
Gypsum plasterboard type H type A type F	 ≥ 740 ≥ 680 ≥ 830	9 - 15	Expression (5.9)	0,8

Table 5.4 — Position coefficients k_{pos} for single layered panels on the unexposed side

Panel	Density kg/m^3	Thickness of panel on unexposed side mm	Position coefficient for panels preceded by					
			Glass fibre	Rock fibre of thickness				Void
				45	95	145	195	
Plywood	≥ 450	9 -25	Expression (5.10)	1,5	1,5	3,9	4,9	0,6
Particleboard, fibreboard	≥ 600	9 -25	Expression (5.10)	1,5	1,5	3,9	4,9	0,6
Wood panelling	≥ 400	15 19	0,45 0,67	1,5	1,5	3,9	4,9	0,6
Gypsum plasterboard type H type A type F	 ≥ 740 ≥ 680 ≥ 830	 9 12,5 15	 0,46 0,74 0,88	1,5	1,5	3,0	4,9	0,7

Table 5.5 — 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

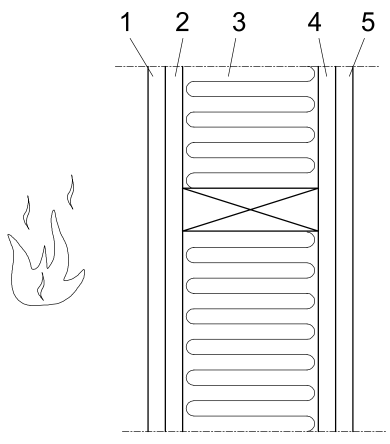


Figure 5.2 — Definition of layer numbers

Table 5.6 — Joint coefficient k_j taking into account the effect of joints in wood-based panels not being backed by battens

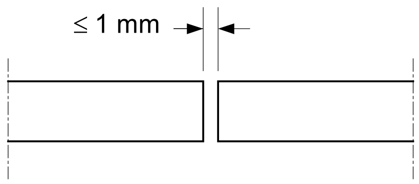
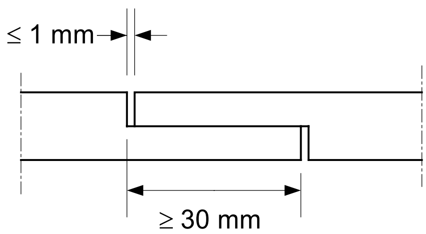
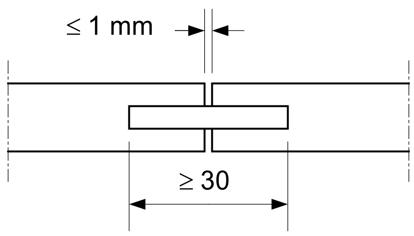
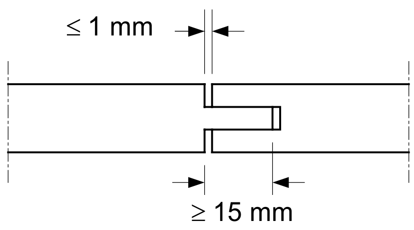
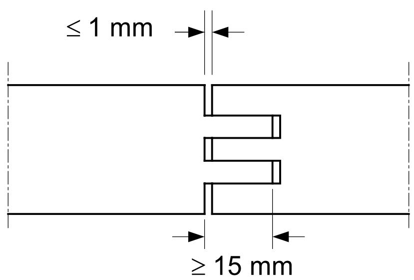
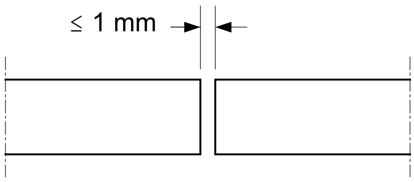
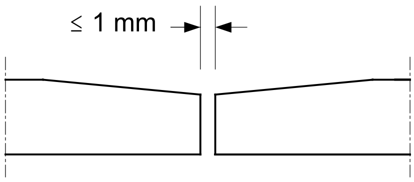
	Joint type	k_j
a		0,2
b		0,3
c		0,4
d		0,4
e		0,6

Table 5.7 — Joint coefficient k_j taking into account the effect of joints in panels of gypsum plasterboard not being backed by battens

	Joint type	Type	k_j
1		A, H, F	0,2
2		A, H,F	0,15

5.4 Advanced calculation methods

(1) For advanced calculation methods for the analysis of the load-bearing function of wall and floor assemblies, clause 4.4 applies.

Section 6 Connections

6.1 General

(1) This section applies to connections between members at standard fire exposure, made with nails, bolts, dowels, and ring and shear plate connectors according to EN 912 and glued-in rods. Where not stated otherwise, the rules apply to fire resistances of not more than 60 minutes.

(2) The rules are valid for symmetrical three-member connections with laterally loaded fasteners (see figures 8.2 g-k of EN 1995-1-1), and glued-in rods.

6.2 Connections with side members of wood

6.2.1 Simplified rules

6.2.1.1 Unprotected connections

(1) For unprotected wood-to-wood joints with spacings, distances and side member dimensions complying with minimum requirements given in EN 1995-1-1 section 8, times of fire resistance may be taken from table 6.1.

Table 6.1 — Time of fire resistance of unprotected connections with side members of wood

	Time of fire resistance $t_{fi,d}$ min	**Provisions**[a]
Smooth 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] t_1 is the thickness of the side member		

(2) For fire resistance periods greater than those given in table 6.1, but not more than 30 minutes, and using connections with dowels, nails or screws with non-projecting heads, then
— the thickness of side members;
— the length and width of the side members
— the end and edge distance to fasteners;
should be increased by a_{fi} (see figure 6.1) given as

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

where

β_0 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 the time of standard fire resistance

$t_{fi,d}$ is the time of the fire resistance of the unprotected connection according to table 6.1

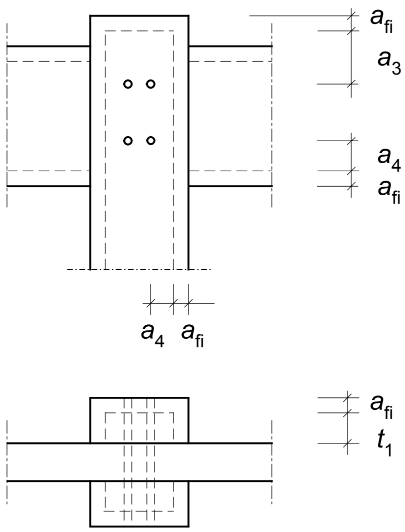


Figure 6.1 — Extra thickness and extra end and edge distances of connections

(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 sheets of wood panels, wood-based panels or gypsum plasterboard type A or H according to EN 520, the additional fire resistance of the additional member protection should satisfy

$$t_{ch} \geq t_{req} - 0{,}5\, t_{fi,d} \quad (6.2)$$

where

t_{ch} is the time of start of charring according to 3.3.3;

t_{req} is required the time of standard fire resistance;

$t_{fi,d}$ is the inherent fire resistance of the unprotected connection according to table 6.1 loaded with design effect of actions $E_{d,fi}$.

(2) When the connection is protected by the addition of gypsum plasterboard type F according to EN 520, the additional fire resistance of the additional protection protective should satisfy

$$t_{ch} \geq t_{req} - 1{,}2\, t_{fi,d} \quad (6.4)$$

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

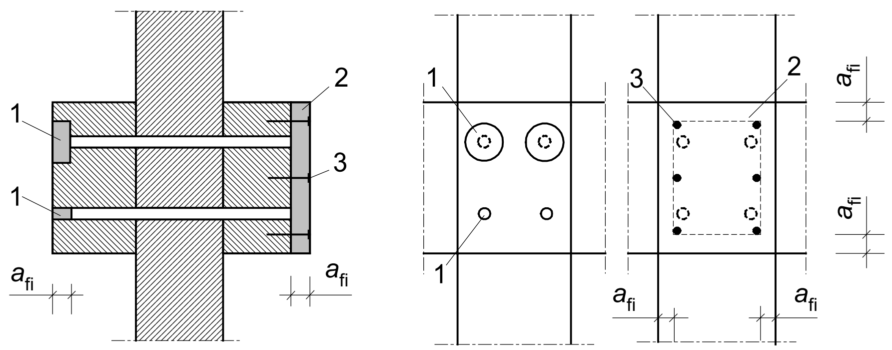
(4) The additional protection should be fixed such that its premature failure is prevented. Additional protection using wood-based panels or gypsum plasterboard should remain in

place until charring starts of the member ($t = t_{ch}$). Additional protection using gypsum plasterboard type F should remain in place during the required time of fire resistance ($t = t_{req}$).

(5) For protection of connections with bolts the bolt heads should be protected by a protection of thickness h_{fi}, see figure 6.3.

(6) For fastening of the additional protection with nails or screws
— the distance between fasteners should be at least 100 mm along edges and at least 300 mm remote from edges;
— the edge distance of fasteners should be at least equal to according to expression (6.1), see figure 6.2.

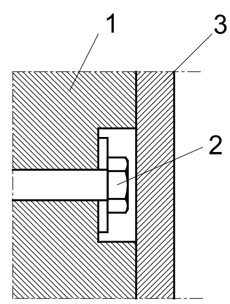
(7) The penetration depth of fasteners for fastening of the additional protection made of wood, wood-based panels or gypsum plasterboard type A or Hshould be at least 6d . For gypsum plasterboard type F, the penetration length into unburned wood (that is beyond the charring depth) should be at least 10 mm, see figure 7.1b.



Key:

1 Glued-in plugs
2 Additional protection using panels
3 Fastener for fixing of additional protection using panels

Figure 6.2 — Examples of additional protection by glued-in plugs and protection made by wood-based panels or gypsum plasterboard (the protection of edges of side and middle members is not shown)



Key:
1 Member
2 Bolt
3 Protection

Figure 6.3 — Example of protection of bolt head

6.2.1.3 Additional rules for connections with internal steel plates

(1) For joints with steel plates as middle members with a thickness equal or greater than 2 mm, and where the steel plates do not project beyond the timber surface, the widths 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) Edges of steel plates with a width smaller than the width of the timber member may be considered as protected in the following cases (see figure 6.3):

- 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 of R 30 and greater than 60 mm for a fire resistance of R 60
- For joints with glued-in strips or protective wood-based boards where the gap depth d_g or the panel thickness h_p is greater than 10 mm for a fire resistance of R 30 and greater than 30 mm for a fire resistance of R 60

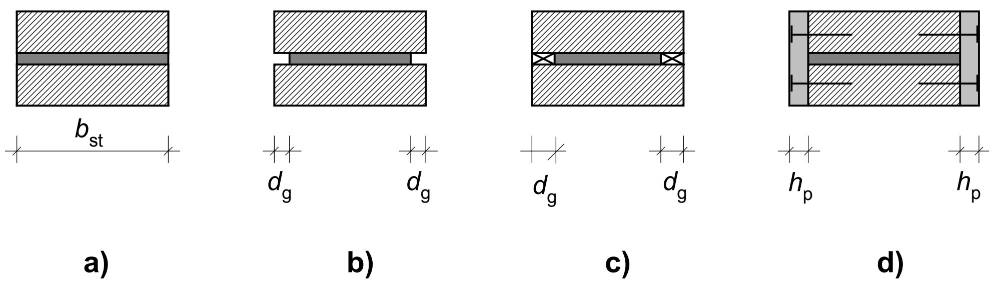


Figure 6.3 — Protection of edges of steel plates (fasteners not shown): a) unprotected, b) protected by gaps, c) protected by glued-in strips, d) protected by panels

6.2.2 Reduced load method

6.2.2.1 Unprotected connections

(1) For standard fire exposure, the characteristic mechanical resistance of a connection exposed to fire should be calculated as

$$F_{Rk,fi} = \eta_{conn} F_{Rk} \tag{6.5}$$

with

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

where

η_{conn} is a conversion function of *t* for the reduction of the mechanical resistance of connections in the fire situation;

k is a parameter given in table 6.2

$t_{fi,d}$ is the design fire resistance of the unprotected connection in minutes.

NOTE: The design load-bearing capacity is calculated according to 2.3 (2)P

(2) The design fire resistance of the unprotected connection loaded with the design effect of actions should be taken as

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

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);

γ_M is the partial factor for the connection, see EN 1995-1-1, subclause 2.2.2;

k_{fi} is a value according to 2.3 (4);

$\gamma_{M,fi}$ is the partial safety factor for timber in fire.

Table 6.3 — Parameters *k*

Connection with	***k***	**Maximum time of validity for unprotected connection** min
Smooth nails	0,08	20
Bolts wood-to-wood with $d \geq 12$ mm	0,065	25
Bolts steel-to-wood with $d \geq 12$ mm	0,085	25
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	25
Connectors	0,065	25
[a] The values for dowels are for connections with up to 20 % bolts to avoid separation of side members		

(3) For dowels projecting more than 5 mm, values of *k* should be taken as for bolts.

(4) For connections made of both bolts and dowels, the load-bearing capacity should be calculated as the sum of the load-bearing capacities of respective fastener.

(5) For connections with nails or screws with non-projecting heads, for fire resistances greater than given by expression (6.6) 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

$$a_{fi} = \beta_0 \left(t_{req} - t_{fi,d}\right) \tag{6.8}$$

where

β_0 is the charring rate according to table 3.1

t_{req} is the required standard fire resistance

$t_{fi,d}$ is the fire resistance of the unprotected connection loaded with the design effect of actions

(6) For greater fire resistances than 30 minutes, see 6.2.1.1 (3).

6.2.2.2 Protected connections

(1) Subclause 6.2.1.2 applies, however with $t_{fi,d}$ calculated according to expression (6.7).

(2) As an alternative 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 applies also for butted middle members.

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 according to EN 1993-1-2, steel surfaces in close contact with wood may be taken as protected.

6.3.2 Protected connections

(1) Steel plates used as side members may be considered as protected if they are totally covered by timber or wood-based panels with a minimum thickness of a_{fi} according to expression (6.1) with $t_{d,0}$ = 5 min.

Steel plate edges should be protected accordingly.

(2) The effect of other fire protections should be calculated according to EN 1993-1-3.

6.4 Axially loaded screws

(1) For axially loaded screws which are protected from direct fire exposure, the following rules apply.

6.4.1 Simplified rules

(1) The design resistance of the screws should be calculated according to expression (2.3).

(2) For connections according to figure 6.4 with

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

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

where a_1, a_2 and a_3 are distances in millimetres, the conversion function η_{conn} for the reduction of the mechanical resistance of the screw in the fire situation should be taken as

$$\eta_{conn} = 0 \qquad \text{for } a_1 \leq 0{,}6\, t_{fi,d} \tag{6.11a}$$

$$\eta_{conn} = \frac{0,44\ a_1 - 0,264\ t_{fi,d}}{0,2\ t_{fi,d} + 5} \quad \text{for } 0,6\ t_{fi,d} \le a_1 \le 0,8\ t_{fi,d} + 5 \qquad (6.11b)$$

$$\eta_{conn} = \frac{0,56\ a_1 - 0,36 \cdot t_{fi,d} + 7,32}{0,2\ t_{fi,d} + 23} \quad \text{for } 0,8\ t_{fi,d} + 5 \le a_1 \le t_{fi,d} + 28 \qquad (6.11c)$$

$$\eta_{conn} = 1,0 \quad a_1 \ge t_{fi,d} + 28 \qquad (6.11d)$$

where

a_1 is the side cover in mm, see figure 6.4

$t_{fi,d}$ is the required time of fire resistance in minutes

(3) The conversion function η_{conn} for side covers $a_2 = a_1$ and $a_3 \ge a_1 + 20$ mm should be calculated according to expression (6.11) where $t_{fi,d}$ is replaced by 1,25 $t_{fi,d}$.

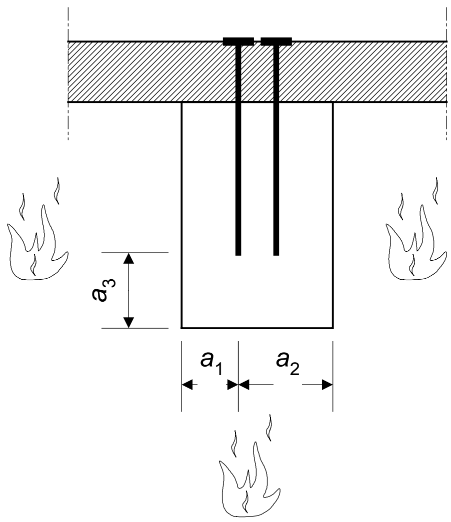


Figure 6.4 — Cross section and definition of distances[8]

6.4.3 Advanced method

(1) Advanced methods for the determination of the mechanical resistance should take into account the following:

— ….
— ….
— ….

NOTE: A method is given in annex D (informative).

6.5

[8] **Drafting note: (symbols to be changed since they interfere with spacings and end and edge distances in Part 1-1)**

Section 7 Detailing

7.1 Walls and floors

7.1.1 Dimensions and spacings

(1) The spacing of wall studs and floor joists should not be greater than 625 mm.

(2) For walls, individual panels should have a minimum thickness of

$$t_{p,min} = \max \begin{cases} \dfrac{l_p}{70} \\ 8 \end{cases} \qquad (7.1)$$

where

$t_{p,min}$ is the minimum thickness of panel in millimetres

l_p is the span of the panel (spacing of timber frame members or battens) in millimetres.

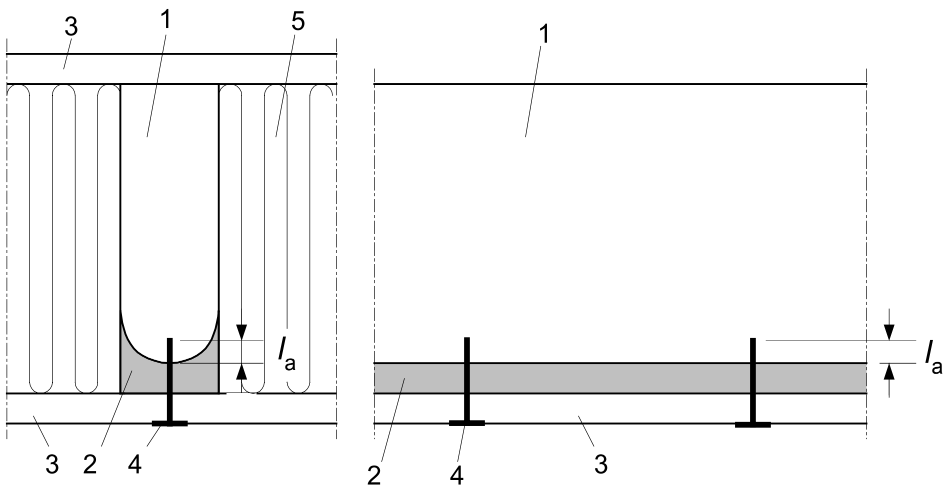
(3) Wood-based panels in constructions with a single layer on each side should have a characteristic density of at least 350 kg/m^3.

7.1.2 Detailing of panel connections

(1) Panels should be fixed to the timber frame or battens. For wood-based and wood panelling fixed with nails, the maximum spacing should be 150 mm. The minimum penetration depth should be eight times the fastener diameter for load bearing panels and six times the fastener diameter for non load bearing panels, or such. When the panels are fixed with screws the maximum spacing should be 250 mm.

(2) For gypsum plasterboard of type A and H it is sufficient to observe the rules for normal temperature design with respect to penetration depth, spacings and edge distances. For screws, however, the spacing along edges should not be greater than 200 mm and remote from edges not greater than 300 mm.

(3) For gypsum plasterboard type F panels, the penetration length of fasteners into unburned timber should not be less than 10 mm, see figure 7.1.



Key

1 Unburned 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 unburned timber: a) Timber frame assembly with insulation in cavity, b) Wide timber member in general

(4) 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. For multiple layers this requirement applies to the outer layer.

(5) 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 fixed to the timber frame such that premature failure or slumping is prevented.

7.2 Other elements

(1) Fire protective panels protecting members such as beams and columns should be fixed 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. Spacings of fasteners should not be greater than 200 mm. With reference to fastener length, 7.1.2(1) applies, see figure 7.1 b.

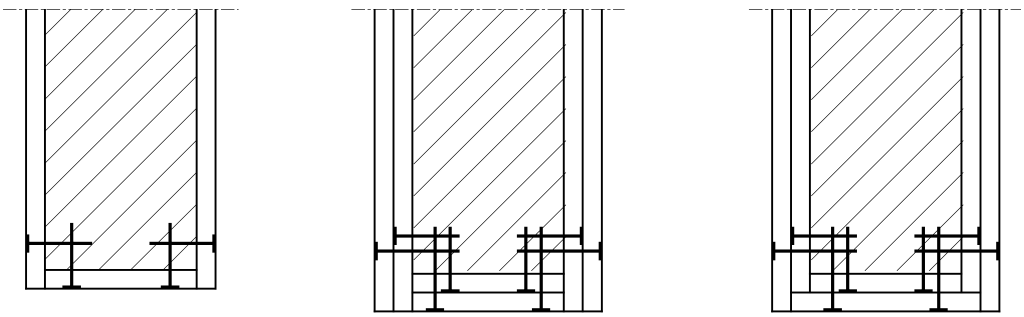


Figure 7.2 — Examples of fixing of fire protective panels

Annex A (Informative) Parametric fire exposure

A.1 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, annex A.

A.2 Charring rates and charring depths

(1) For unprotected softwood the relation between the charring rate and time t according to figure A.1 should be used. The charring rate β_{par} during the heating phase of a parametric fire curve is given by

$$\beta_{par} = 1{,}5\ \beta_n \frac{5\ O k_b - 0{,}04}{4\ O k_b + 0{,}08} \tag{A.1}$$

with

$$O = \frac{A}{A_t}\sqrt{h_{eq}} \tag{A.2}$$

$$k_b = \frac{1160}{\sqrt{\lambda \rho c}} \tag{A.3}$$

and

$$h_{eq} = \sum \frac{A_i\ h_i}{A} \tag{A.4}$$

where

O is the opening factor in $m^{1/2}$;

β_n is the notional charring rate in mm/min;

A is the total area of vertical openings (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;

k_b is a factor accounting for the thermal properties of the boundaries of the enclosure of the compartment (more information on the determination of is given in EN 1991-1-2, annex A);

λ thermal conductivity of boundary of enclosure in $Wm^{-1}K^{-1}$;

ρ density of boundary enclosure in kg/m^3;

c specific heat of boundary of enclosure in $Jkg^{-1}K^{-1}$.

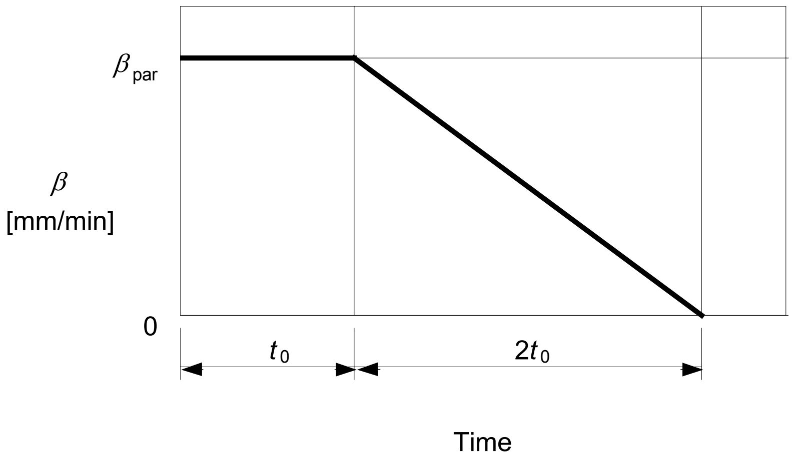


Figure A.1 — Relationship between charring rate and time

(2) The maximum charring depth during the total duration of fire exposure, i.e. the heating phase and the subsequent decay period should be taken as

$$d_{\mathrm{char}} = 2\,\beta_{\mathrm{par}}\,t_0 \qquad \text{(A.5)}$$

with

$$t_0 = 0{,}009\,\frac{q_{\mathrm{t,d}}}{O} \text{ (Changed corresponding to prEN 1991-1-2)} \qquad \text{(A.6)}$$

where

t_0 is in minutes;

$q_{\mathrm{t,d}}$ 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.

Equations (A.1), (A.5) and (A.6) should only be used for values of O between 0,02 and 0,20 $m^{1/2}$ and for

$$t_0 \le 40 \text{ min}$$

$$d_{\mathrm{char}} \le \frac{b}{4}$$

$$d_{\mathrm{char}} \le \frac{h}{4}$$

where

b is the width of the cross section;

h is the depth of the cross section.

A.3 Mechanical resistance of members in edgewise bending

(1) For members in edgewise bending with an initial width b of 130 mm or more exposed to fire on three sides the lowest mechanical resistance during the complete fire endurance 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.4).

(2) For softwood timber the modification factor for fire $k_{mod,fi}$ should be calculated according to the following:

- For $t \leq t_{in} + 3t_0$ the modification factor for fire should be calculated according to expression (4.2)
- for $t = t_{in} + 5t_0$ as

$$k_{mod,fi} = 1,0 - 3,2 \frac{d_{char,n}}{b} \tag{A.7}$$

where

$d_{char,n}$ is the notional charring depth;

b is the width of the member.

For $t_{in} + 3t_0 \leq t \leq t_{in} + 5t_0$ 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 $t \leq t_{in} + 3t_0$ the modification factor for fire can be derived from the reduced cross section method as

$$k_{mod,fi} = \frac{W_{ef}}{W_r} \tag{A.8}$$

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) Thermal and mechanical material properties

B.1 Timber

B.1.1 Thermal properties

(1) Values of thermal conductivity, specific heat and the ratio of density to dry density of softwood may be taken as given in figures B.1 to B.3 and tables B.1 and B.2.

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 for these effects.

NOTE 2: Depending on the model used for calculation, modification of thermal properties given here may be necessary.

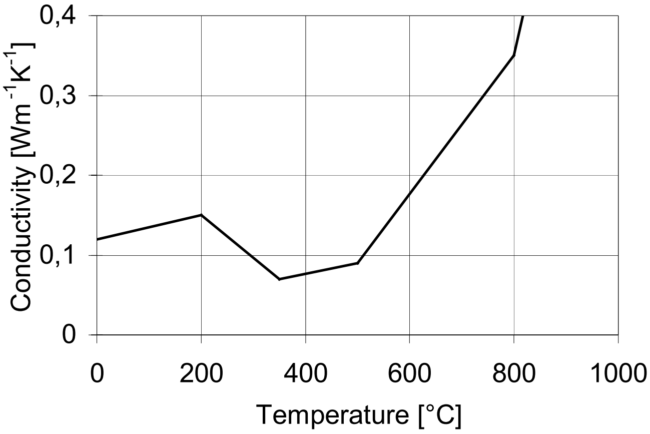


Figure B.1 – Temperature-conductivity relationship for wood and the char layer

Table B.1 – Temperature-conductivity relationship for wood and the char layer

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

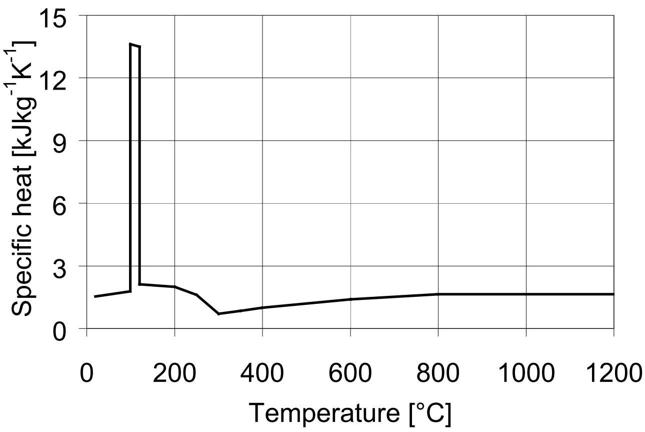


Figure B.2 – Temperature-specific heat relationship for wood and charcoal

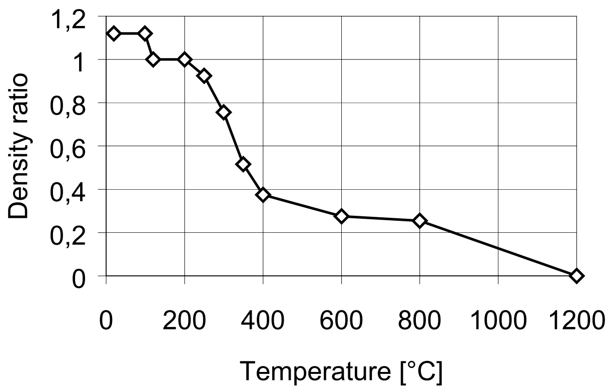


Figure B.3 – Temperature-density ratio relationship for softwood with an initial moisture content of 12 %

Table B.2 – 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}	Density ratio –
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

B.1.2 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 B.4 and B.5.

NOTE: The relationships include the effects of transient creep of timber.

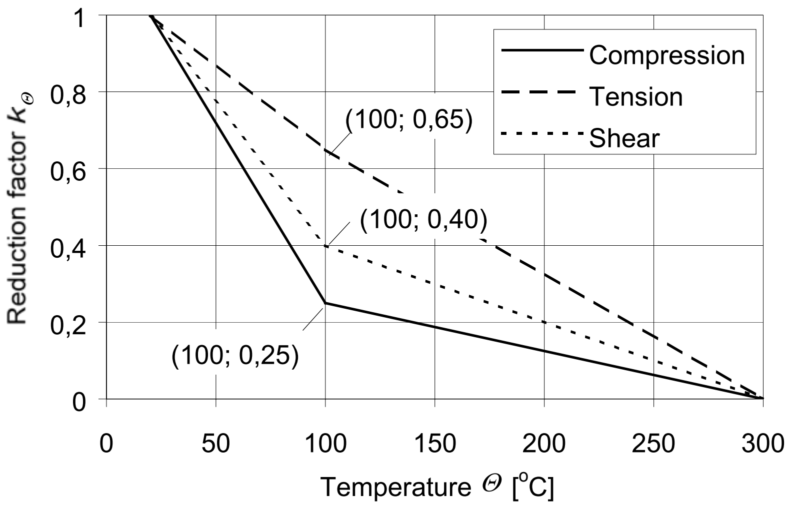


Figure B.4 – Reduction factor for strength parallel to grain of softwood

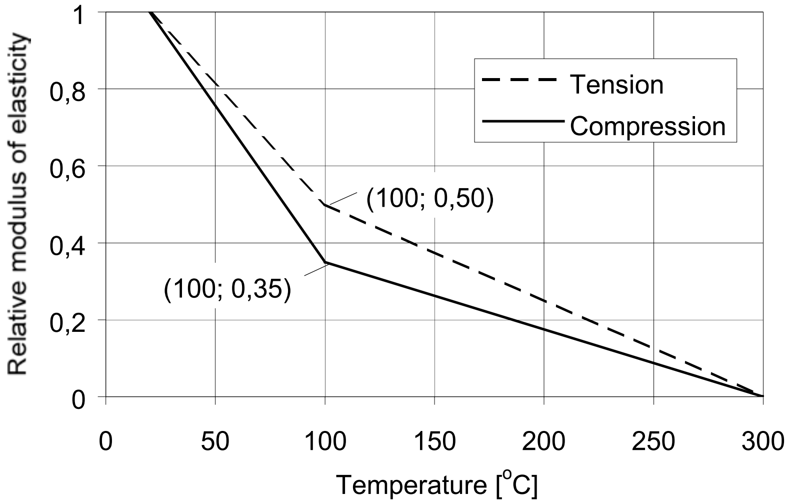


Figure B.5 – Effect of temperature on modulus of elasticity parallel to grain of softwood

(2) For compression perpendicular to grain, the same reduction of strength may be applied as for compression parallel to grain.

(3) For shear parallel to grain, the same reduction of strength may be applied as for compression parallel to grain.

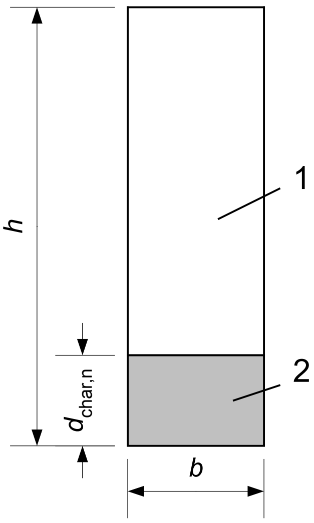
Annex C (Informative) Load-bearing floor joists and wall studs

C.1 Residual cross section

(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 assumptions apply:

- the cavities are filled with insulation made of rock or glass fibre;
- the studs are braced against buckling in the plane of the wall and against torsional buckling by means of panels on the unexposed side or by noggins;
- for walls, the cladding may also be fixed to steel channels with a maximum depth 25 mm which are perpendicular to the direction of the timber joists;
- the separating function is verified according to 5.3.

(2) The notional residual cross section may be determined according to figure C.1 where the notional charring depth is given by expression (3.2).



Key:
1 Notional residual cross section
2 Notional char layer

Figure C.1 — Notional residual cross section of timber frame member protected by cavity insulation

(3) For timber members protected by claddings on the fire-exposed side, the charring rate may be calculated as

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

$$\beta_n = k_s\, k_3\, k_n\, \beta_0 \qquad \text{for } t \ge t_{bf} \qquad (C.2)$$

with

$$k_n = 1{,}5$$

where

β_n is the notional charring rate in millimetres per minute;

k_s is the cross section factor;

k_2 is the insulation factor;

k_3 is the post-protection factor;

k_n is a factor to convert the irregular residual cross section into a notional rectangular cross section;

β_0 is the basic charring rate for one-dimensional charring, see 3.3.2 table 3.1;

t is the time of fire exposure;

t_{ch} is the time of start of charring of the timber frame member;

t_f is the failure time of the cladding.

(4) The cross section factor may be taken from table C.1.

Table C.1 — Cross section factor for different widths of timber frame member

b mm	k_s
38	1,4
45	1,3
60	1,1

(5) 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

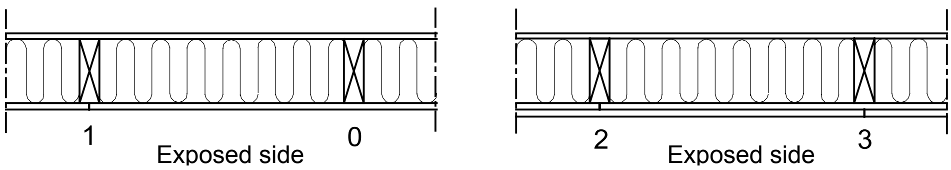
– for joint configurations 0 and 2, see figure C.2:

$$k_2 = 1{,}05 - 0{,}0073\, h_p \tag{C.3}$$

– for joint configurations 1 and 3, see figure C.2:

$$k_2 = 0{,}86 - 0{,}0037\, h_p \tag{C.4}$$

where h_p is the total thickness of panels in millimetres.



Key

0: No joint
1: Joint in single layer
2: Joint in inner board layer
3: Joint in outer board layer

Figure C.2 — Joint configurations of linings with one or two layers

(6) 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 may be calculated as

$$k_3 = 0{,}036\, t_f + 1 \tag{C.5}$$

where t_f is the failure time of the lining in minutes.

(7) Where the cavity insulation is made of glass fibre, failure of the member should be assumed to take place at the time t_f.

(8) For claddings made of wood-based panels, the time of start of charring t_{ch} should be determined as

$$t_{ch} = t_f \tag{C.6}$$

where the failure time t_f should be calculated according to 3.3.3(6).

(9) For claddings made of gypsum plasterboard of type A, H or F, the time of start of charring may be determined according to 3.3.3(7).

(10) For claddings made of gypsum plasterboard type A or H, the failure time should be taken as $t_f = t_{ch}$.

(11) 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 unburned wood.

(12) 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.

(13) The failure time t_f of panels with respect to pull-out failure of fasteners may be calculated as

$$t_f = t_{ch} + \frac{l_f - l_{a,min} - h_p}{k_s\, k_2\, k_n\, k_j\, \beta_0} \tag{C.7}$$

with

$$k_j = 1{,}0 \quad \text{for joint configurations 0 and 2} \tag{C.8}$$

$$k_j = 1{,}15 \quad \text{for joint configurations 1 and 3} \tag{C.9}$$

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 unburned wood;

h_p is the total thickness of the panels;

k_s is the cross section factor;

k_2 is the insulation factor;

k_n is a factor to convert the irregular residual cross section into a notional rectangular cross section;

β_0 is the basic charring rate for one-dimensional charring, see 3.3.2 table 3.1;

(14) Where panels are fixed to steel channels, see figure C.3, the failure time of the steel channels may be calculated according to expression (C.7) where h_p is replaced by the thickness t_s of the steel channel and $k_j = 1{,}0$.

(15) 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

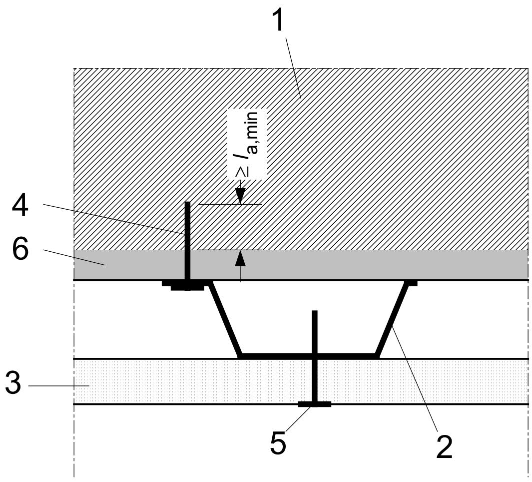
$$t_{sf} = t_f + \frac{l_f - l_{a,min} - k_s\, k_2\, k_n\, \beta_0\, (t_f - t_{ch}) - t_s}{k_s\, k_3\, k_n\, \beta_0} \qquad (C.10)$$

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.



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 C.3 — Example of use of steel channels for fixing panels in the ceiling

(16) For a fire resistance of not more than 60 min, a verification of the load-bearing capacity and stiffness of the steel channels need not be performed.

C.2 Reduction of strength and stiffness parameters

(1) The modification factor for fire for strength should be calculated as

$$k_{mod,fi,fm} = a_0 - a_1 \frac{d_{char,n}}{h} \qquad (C.11)$$

where

a_0, a_1 are values given in table C.1 and C.2;

$d_{char,n}$ 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 C.1 — Values[a] of a_0 and a_1 for reduction of strength for assemblies exposed to fire on one side

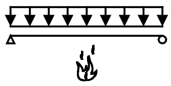
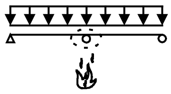
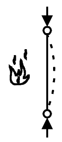
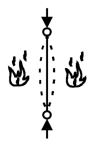
Case			*h* mm	a_0	a_1
1	Bending strength with exposed side in tension		95	0,60	1,04
			145	0,68	1,10
			195	0,73	1,14
			220	0,76	1,14
2	Bending strength with exposed side in compression		95	0,46	0,83
			145	0,55	0,89
			195	0,65	1,07
			220	0,67	1,05
3	Compressive strength		95	0,46	0,83
			145	0,55	0,89
			195	0,65	1,07
			220	0,67	1,05
[a] For intermediate values of *h*, linear interpolation should be applied					

Table C.2 — Values of a_0 and a_1 for reduction of compressive strength for walls exposed to fire on both sides

Case			*h* mm	a_0	a_1
1	Compressive strength		145	0,39	3,65

(2) The modification factor for modulus of elasticity should be calculated as

$$k_{\text{mod,fi,E}} = b_0 - b_1 \frac{d_{\text{char,n}}}{h} \qquad \text{(C.12)}$$

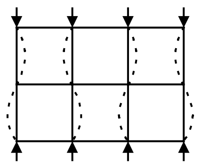
where

b_0, b_1 are values given in tables C.3 and C.4;

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

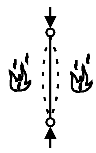
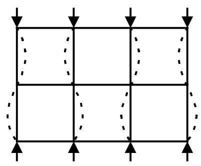
h is the depth of the joist..

Table C.3 — Values[a] of b_0 and b_1 for reduction of modulus of elasticity for walls exposed to fire on one side

Case			*h* mm	b_0	b_1
1	Buckling perpendicular to wall plane		95	0,50	1,77
			145	0,60	1,88
			195	0,68	1,73
2	Buckling in plane of wall		95	0,54	1,11
			145	0,66	1,23
			195	0,73	1,41

[a] For intermediate values of *h*, linear interpolation should be applied
NOTE: In the illustration to case 2 the studs are braced by noggins

Table C.4 — Values[a] of b_0 and b_1 for reduction of modulus of elasticity for walls exposed to fire on both sides

Case			*h* mm	b_0	b_1
1	Buckling perpendicular to wall plane		145	0,37	4,2
2	Buckling in plane of wall		145	0,44	4,9

Annex D (informative) Advanced methods for glued-in screws and steel rods

D.1 Glued-in screws

(1) The withdrawal shear strength of the timber should multiplied by a temperature dependent reduction factor given as (see figure 6.5)

$$k_{\Theta} = \frac{-0{,}70\ \Theta_{\mathrm{i}} + 114}{100} \quad \text{for } 20 \le \Theta_{\mathrm{i}} \le 100°\text{C} \tag{6.12}$$

$$k_{\Theta} = \frac{-0{,}22\ \Theta_{\mathrm{i}} + 66}{100} \quad \text{for } 100 \le \Theta_{\mathrm{i}} \le 300°\text{C} \tag{6.13}$$

where Θ is the local temperature in the timber in °C

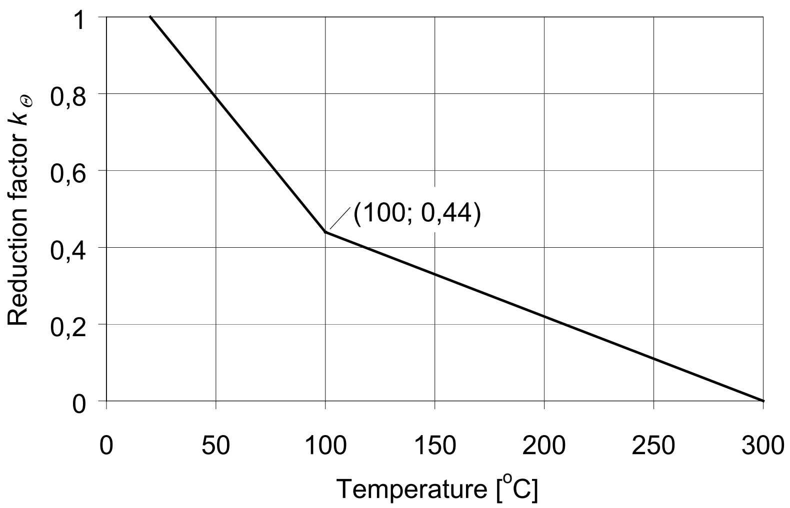


Figure 6.5 — Reduction factor for withdrawal shear strength

(2) The temperature around the screws depends on the section size and the position of the screws in the timber member. The influence of the heat flux from all sides with direct fire exposure should be taken into account.

(3) For one-dimensional heat transfer, the temperature profile along the fastener should be calculated according to:

$$\Theta = 20 + 280 \left(\frac{\beta_0\ t}{y} \right)^{\alpha} \tag{6.14}$$

with

$$\alpha = 0{,}025\ t + 1{,}75 \tag{6.15}$$

where

- Θ is the temperature in the timber in °C
- y is the distance in mm from the original surface of the timber
- t is the time in minutes

(4) For multi-dimensional heat transfer, e.g. two dimensional heat transfer as shown in figure 6.6, the temperature at a point P with co-ordinates y and z should be calculated according to:

$$\Theta = 20 + 280\,(\beta_0\, t)^{\alpha} \left\{ \left(\frac{1}{y}\right)^{\alpha} + \left(\frac{1}{b-y}\right)^{\alpha} + \left(\frac{1}{z}\right)^{\alpha} \right\} \tag{6.16}$$

where

- Θ is the temperature in a point P in °C
- y, z are the co-ordinates of point P in millimetres
- t is the time in minutes
- α is given by expression (6.15)

(5) The design mechanical resistance of the screw should be calculated according to:

$$F_{\mathrm{Rd,fi}} = \pi\, d\, \frac{k_{\mathrm{fi}}\, f_{\mathrm{v,k}}}{\gamma_{\mathrm{M,fi}}} \sum_{i=1}^{n} \left\{ k_{\Theta,i} \cdot \Delta \ell_{\mathrm{i}} \right\} \tag{6.17}$$

where

- Θ is the temperature of the element i in °C, see figure 6.6;
- $f_{\mathrm{v,k}}$ is the characteristic shear strength of timber;
- k_{fi} should be taken as for solid timber from table 2.1;
- $\gamma_{\mathrm{M,fi}}$ is the partial factor for timber in fire;
- $\Delta \ell_{\mathrm{i}}$ is the length of the element i of totally *n* elements into which the anchorage length l_{a} should be subdivided, see figure 6.6;
- $k_{\Theta,\iota,}$ is the reduction factor for the withdrawal shear strength of element *i* according to expression (6.12) and (6.13);
- d is the outer diameter measured on the threaded part of the screw.

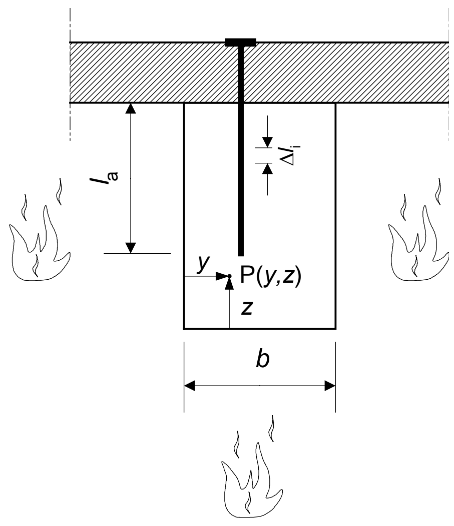


Figure 6.5 — Definitions of co-ordinates and dimensions

D.2 Glued-in steel rods

(1) For axially loaded glued-in steel rods, which are protected from direct fire exposure, the following rules apply.

(2) The design resistance of axially loaded glued-in rods should be verified for the failure modes according to 8.11.2.1 of EN 1995-1-1 taking into account the effect of the fire situation on the mechanical properties of the steel rod, the wood, the adhesive and its bond to steel.

(3) The effect of the temperature on the withdrawal shear strength of softwood should be taken from figure 6.5a.

(4) The effect of the temperature on the shear strength of the adhesive and its bond to steel should be verified by tests.

(5) The effect of the temperature on the yield strength of the steel may be neglected.

(6) The temperature around the rod depends on the section size and the position of the glued-in rods in the timber member. The influence of the heat flux from all sides with direct fire exposure should be taken into account.

(7) For one-dimensional heat transfer, the temperature profile may be calculated according to expression (6.12).

(8) For multi-dimensional heat transfer, the temperature profile may be calculated according to expression (6.14).

Annex E (informative) Guidance for users of this Eurocode Part

(1) In this annex flow charts are given as guidance for users of this Part 1-1 of EN 1995, see figure E.1 and E.2.

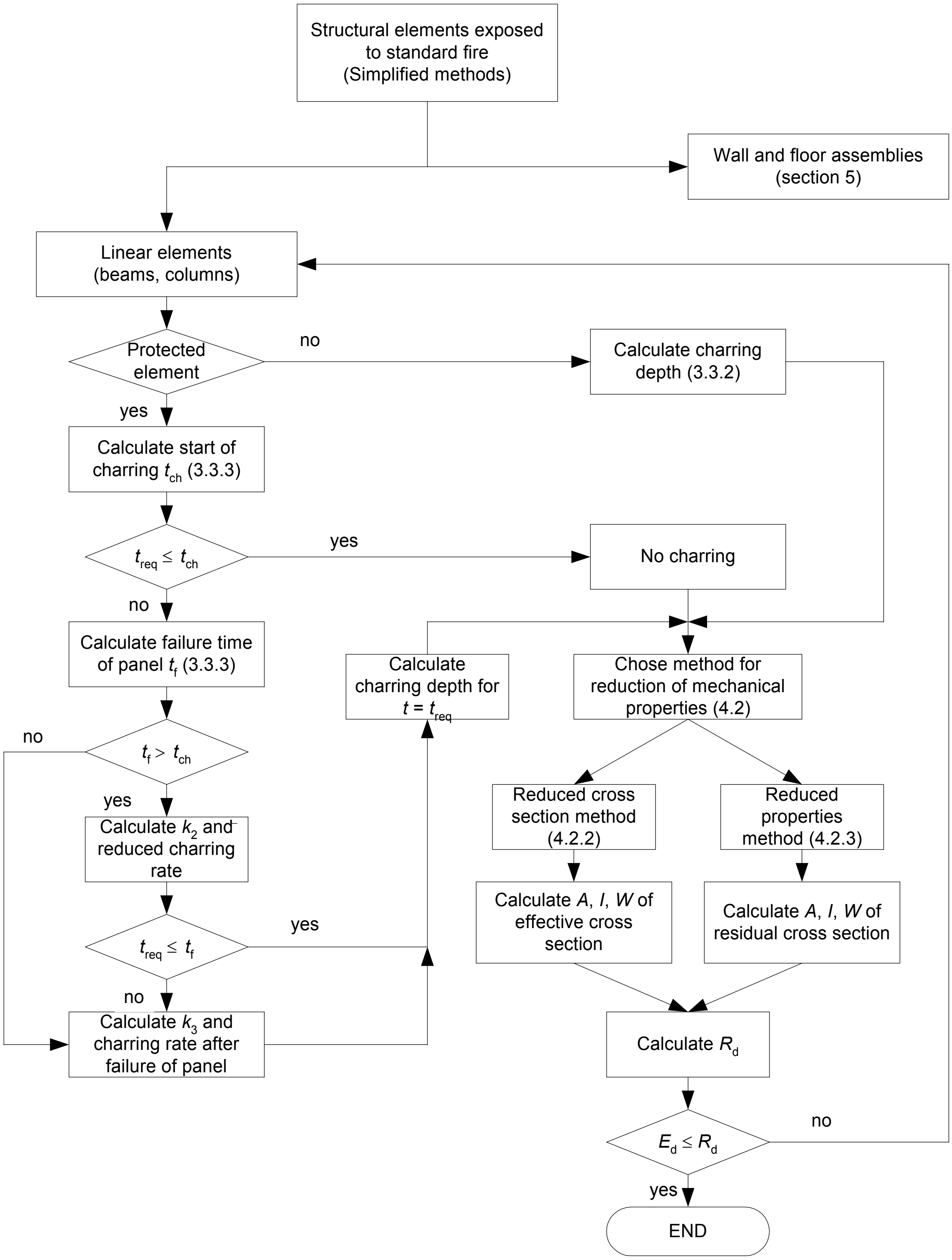


Figure E.1 — Flow chart for the design procedure of structural members

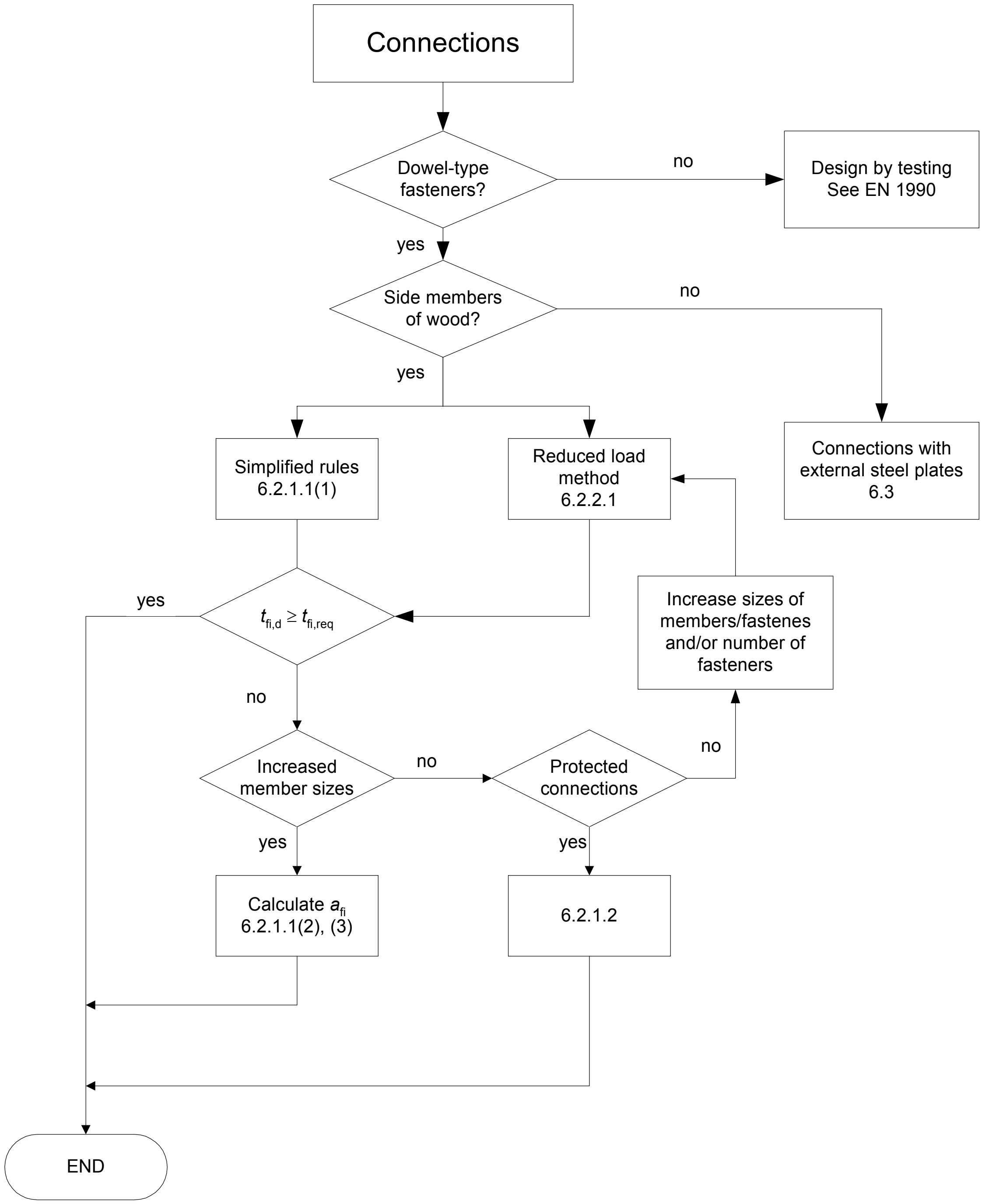


Figure E.2 — Flow chart for the design procedure of connections