prEN 1995-1-2
Eurocode 5 – Design of timber structures
Part 1-2: General rules – Structural fire design
Final Draft - October 2001
Stage 34
Clean version
Contents

Contents 1
Foreword 3
  Background of the Eurocode programme 3
  Status and field of application of Eurocodes 4
  National Standards implementing Eurocodes 4
  Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products 5
  Additional information specific to EN 1995-1-2 5
  National Annex for EN 1995-1-2 8
Section 1  General 9
  1.1  Scope 9
  1.2  Normative references 9
  1.3  ASSUMPTIONS 10
  1.4  Distinction between principles and application rules 10
  1.5  Definitions 10
  1.6  Symbols 11
Section 2  Basic principles and rules 14
  2.1  Performance requirements 14
    2.1.1  General 14
    2.1.2  Nominal fire exposure 14
    2.1.3  Parametric fire exposure 14
  2.2  Actions 15
  2.3  Design values of material properties and resistances 15
  2.4  Assessment methods 16
    2.4.1  General 16
    2.4.2  Member analysis 17
    2.4.3  Analysis of parts of the structure 18
    2.4.4  Global structural analysis 19
Section 3  Material properties 20
  3.1  Mechanical properties 20
  3.2  Thermal properties 20
  3.3  Charring 20
    3.3.1  General 20
    3.3.2  Unprotected surfaces 21
    3.3.3  Protected surfaces 23
  3.4  Adhesives 27
Section 4  Design procedures for mechanical resistance 29
  4.1  General 29
  4.2  Simplified rules for cross sectional resistance 29
    4.2.1  General 29
    4.2.2  Reduced cross section method 29
    4.2.3  Reduced properties method 30
  4.3  Simplified rules for analysis of structural members and components 32
    4.3.1  General 32
    4.3.2  Beams 32
    4.3.3  Columns 32
    4.3.4  Mechanically jointed members 32
    4.3.5  Bracings 33
  4.4  Advanced calculation methods 33
    4.4.1  General 33
    4.4.2  Thermal response 33
    4.4.3  Structural response 34
(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

5.2 Analysis of load bearing function

5.3 Analysis of separating function

5.3.1 General

5.3.2 Simplified method for the analysis of insulation

5.3.2.1 General

5.3.2.2 Basic insulation values, position coefficients and effect of joints

5.4 Advanced calculation methods

Section 6 Connections

6.1 General

6.2 Connections with side members of wood

6.2.1 Simplified rules

6.2.1.1 Unprotected connections

6.2.1.2 Protected connections

6.2.1.3 Additional rules for connections with internal steel plates

6.2.2 Reduced load method

6.2.2.1 Unprotected connections

6.2.2.2 Protected connections

6.3 Connections with external steel plates

6.3.1 Unprotected connections

6.3.2 Protected connections

6.4 Axially loaded screws

6.4.1 Simplified rules

6.4.3 Advanced method

Section 7 Detailing

7.1 Walls and floors

7.1.1 Dimensions and spacings

7.1.2 Detailing of panel connections

7.1.3 Insulation

7.2 Other elements

Annex A (Informative) Parametric fire exposure

A.1 General

A.2 Charring rates and charring depths

A.3 Mechanical resistance of members in edgewise bending

Annex B (informative) Thermal and mechanical material properties

B.1 Timber

B.1.1 Thermal properties

B.1.2 Mechanical properties

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

C.1 Residual cross section

C.2 Reduction of strength and stiffness parameters

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

D.1 Glued-in screws

D.2 Glued-in steel rods

Annex E (informative) Guidance for users of this Eurocode Part
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 agreement¹ 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:

<table>
<thead>
<tr>
<th>Code</th>
<th>Eurocode:</th>
</tr>
</thead>
<tbody>
<tr>
<td>EN 1990</td>
<td>Eurocode: Basis of Structural Design</td>
</tr>
<tr>
<td>EN 1991</td>
<td>Eurocode 1: Actions on structures</td>
</tr>
<tr>
<td>EN 1992</td>
<td>Eurocode 2: Design of concrete structures</td>
</tr>
<tr>
<td>EN 1993</td>
<td>Eurocode 3: Design of steel structures</td>
</tr>
<tr>
<td>EN 1994</td>
<td>Eurocode 4: Design of composite steel and concrete structures</td>
</tr>
<tr>
<td>EN 1995</td>
<td>Eurocode 5: Design of timber structures</td>
</tr>
<tr>
<td>EN 1996</td>
<td>Eurocode 6: Design of masonry structures</td>
</tr>
<tr>
<td>EN 1997</td>
<td>Eurocode 7: Geotechnical design</td>
</tr>
<tr>
<td>EN 1998</td>
<td>Eurocode 8: Design of structures for earthquake resistance</td>
</tr>
<tr>
<td>EN 1999</td>
<td>Eurocode 9: Design of aluminium structures</td>
</tr>
</tbody>
</table>

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 No. 1 – Mechanical resistance and stability – and Essential Requirement No. 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 referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards. 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.

\(^4\) 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.
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) 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) This Part 1-2 of EN 1995 deals only with passive methods of fire protection. Active methods are not covered.

(3) 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) 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) 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) 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) 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
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) The rules in EN 1990 clause 1.4 apply.

(2) 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,
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
Width
Parameter
Parameter
Specific heat
Diameter of fastener
Depth of layer with assumed zero strength and stiffness
Charring depth for one dimensional charring
Notional charring depth
Effective charring depth
Gap depth
20 % fractile strength at normal temperature
Design strength in fire
Characteristic strength
Characteristic shear strength
Depth
Weighted average of heights of all vertical openings
Insulation thickness
Fire protective panel thickness
Parameter
Coefficient
Insulation coefficient
Post-protection coefficient
Coefficient
Heat flux coefficient for fasteners
Panel thickness coefficient
Joint coefficient
Notional cross section coefficient
Modification factor
Modification factor for fire
Modification factor for modulus of elasticity in the fire situation
Modification factor for bending strength in the fire situation
Position coefficient
Density coefficient
Temperature dependent reduction factor for local strength or stiffness property
Span of the panel
Anchorage length of fastener
Minimum anchorage length of fastener
Length of fastener
Perimeter of the fire exposed residual cross section
Design fire load density related to the total area of floors, walls and ceilings which enclose the fire compartment
Time of fire exposure
Thickness of the side member
Time of start of charring of protected members (delay of start of charring due to protection)
Failure time of protection
Time of the fire resistance of the unprotected connection
Time of temperature increase on the unexposed
Basic insulation value of layer “i”
Minimum thickness of panel
Time of fire resistance with respect to the load-bearing function
Required time of fire resistance
Co-ordinate
Co-ordinate
Greek upper case letters

$\Theta$  
Temperature

Greek lower case letters

$\beta_0$  
Basic charring rate for one-dimensional charring

$\beta_n$  
Notional charring rate

$\beta_{par}$  
Charring rate during heating phase of parametric fire curve

$\eta_{conn}$  
Conversion factor for the reduction of the load-bearing capacity in fire

$\eta_f$  
Conversion coefficient

$\gamma_{GA}$  
Partial factor for permanent actions in accidental design situations

$\gamma_M$  
Partial factor for a material property, also accounting for model uncertainties and dimensional variations

$\gamma_{M,fi}$  
Partial factor for timber in fire

$\gamma_{D,1}$  
Partial factor for variable action 1

$\lambda$  
Thermal conductivity

$\rho$  
Density

$\rho_c$  
Characteristic density

$\omega$  
Moisture content

$\psi_{1,1}$  
Combination factor for frequent value of a variable action

$\psi_{2,1}$  
Combination factor for quasi-permanent value of a variable action

$\psi_f$  
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) 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) 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) 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) 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 strength and stiffness parameters
shall be determined from

\[ f_{d,fi} = k_{mod,fi} \frac{f_{20}}{\gamma_{M,fi}} \quad (2.1) \]

\[ E_{d,fi} = k_{mod,fi} \frac{E_{20}}{\gamma_{M,fi}} \quad (2.2) \]

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

\[ F_{Rd,fi} = \eta_{conn} \frac{F_{R20}}{\gamma_{M,fi}} \quad (2.3) \]

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 (\( k_{mod} = 1 \));
- \( \eta_{conn} \) 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

\[
\begin{align*}
f_{20} &= k_{fi} f_k \quad \text{(2.4)} \\
E_{k20} &= k_{fi} E_{0.05} \quad \text{(2.5)}
\end{align*}
\]

where \( k_{fi} \) should be taken from table 2.1.

<table>
<thead>
<tr>
<th>Table 2.1 — Values of ( k_{fi} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid timber</td>
</tr>
<tr>
<td>Glued-laminated timber</td>
</tr>
<tr>
<td>Wood-based panels</td>
</tr>
<tr>
<td>LVL</td>
</tr>
<tr>
<td>Connections with side members of wood and wood-based panels</td>
</tr>
<tr>
<td>Connections with side members of steel</td>
</tr>
</tbody>
</table>

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

\[
F_{R,20} = k_{fi} F_{R,k} \quad \text{(2.6)}
\]

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) The model of the structural system adopted for design shall reflect the performance of the structure in the fire situation.

(2) It shall be verified for the required duration of fire exposure \( t \):

\[
E_{d,fi} \leq R_{d,t,fi} \quad \text{(2.7)}
\]

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

\( R_{d,t,fi} \) 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) 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

\[
E_{d,fi} = \eta_{fi} E_d
\]

where

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

\( \eta_{fi} \) is the reduction factor for the design load in the fire situation.

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

\[
\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}}
\]

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

\[
\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}}
\]

\[
\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\xi G_k + \gamma_{Q,1} Q_{k,1}}
\]

where

\( Q_{k,1} \) is the characteristic value of the principle variable action;

\( G_k \) is the characteristic value of a permanent action;

\( \gamma_G \) is the partial factor for permanent actions;

\( \gamma_{Q,1} \) is the partial factor for variable action 1;
$\psi_f$ is the combination factor for frequent values of variable actions, see EN 1991-1-2

$\xi$ is a reduction factor for unfavourable permanent actions $G$.

NOTE 1: An example of the variation of the reduction factor $\eta_f$ versus the load ratio $Q_{k,1}/G_k$ for different values of the combination factor $\psi_f$ according to expression (2.9) is shown in figure 2.1 with the following assumptions: $\gamma_{GA} = 1.0$, $\gamma_G = 1.35$ and $\gamma_Q = 1.5$. Partial factors are specified in the relevant National annexes of EN 1990. Expressions (2.9a) and (2.9b) give slightly higher values.

![Figure 2.1 – Examples of reduction factor $\eta_f$ versus load ratio $Q_{k,1}/G_k$ according to expression (2.9)](image)

NOTE 2: As a simplification, the recommended value is $\eta_f = 0.6$, 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 $\eta_f = 0.7$. 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) 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) 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 and the design charring depth should be calculated as (see figure 3.1)

\[ d_{\text{char},0} = \beta_0 t \]  

(3.1)

where

- \( d_{\text{char},0} \) is the design charring depth for one dimensional charring;
- \( \beta_0 \) 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

\[ d_{\text{char},n} = \beta_n t \]  

(3.2)

where

- \( d_{\text{char},n} \) is the notional design charring depth, including the effect of corner roundings;
- \( \beta_n \) 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 \( d_{\text{char},n} \)](image)

(3) For unprotected surfaces of timber design charring rates \( \beta_0 \) and \( \beta_n \) 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 $\beta_0$ and $\beta_n$ 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 $\rho_k$ and thicknesses $h_p$ of panels the charring rate should be calculated as

$$\beta_{0,\text{ch}} = \beta_0 \cdot k_\rho \cdot k_h$$

with

$$k_\rho = \sqrt[\rho_k]{\frac{450}{\rho_k}}$$

$$k_h = \max\left\{\sqrt[\rho_k]{\frac{20}{h_p}}, 1,0\right\}$$

where

- $\rho_k$ 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 $\beta_0$ and $\beta_n$ of timber, LVL, wood panelling and wood-based panels

<table>
<thead>
<tr>
<th></th>
<th>$\beta_0$</th>
<th>$\beta_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm/min</td>
<td>mm/min</td>
<td></td>
</tr>
<tr>
<td>a) Softwood and beech</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glued laminated timber with a characteristic density of $\geq 290$ kg/m$^3$</td>
<td>0,65</td>
<td>0,7</td>
</tr>
<tr>
<td>Solid timber with a characteristic density of $\geq 290$ kg/m$^3$</td>
<td>0,65</td>
<td>0,8</td>
</tr>
<tr>
<td>b) Hardwood</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Solid or glued laminated hardwood with a characteristic density of $\geq 290$ kg/m$^3$</td>
<td>0,65</td>
<td>0,7</td>
</tr>
<tr>
<td>Solid or glued laminated hardwood with a characteristic density of $\geq 450$ kg/m$^3$</td>
<td>0,50</td>
<td>0,55</td>
</tr>
<tr>
<td>c) LVL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>with a characteristic density of $\geq 500$ kg/m$^3$</td>
<td>0,65</td>
<td>0,7</td>
</tr>
<tr>
<td>d) Panels$^a$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wood panelling</td>
<td>0,9</td>
<td>–</td>
</tr>
<tr>
<td>Plywood</td>
<td>1,0</td>
<td>–</td>
</tr>
<tr>
<td>Wood-based panels other than plywood</td>
<td>0,9</td>
<td>–</td>
</tr>
</tbody>
</table>

$^a$ 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.
(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 $d_{\text{char,n}}$ 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.
Figure 3.2 — Illustration of charring depth vs. time for \( t_{\text{ch}} = t_f \)

Figure 3.3 — Illustration of charring depth vs. time for \( t_{\text{ch}} < t_f \) 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

\[
t_f = \frac{h_p}{\beta_0} - 4
\]

where
- \( t_f \) is the failure time in minutes;
- \( \beta_0 \) is the basic charring rate of the panel according to table 3.1 in mm/minute;
- \( h_p \) is the total cladding thickness 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

\[
t_{ch} = 2.8 h_p - 14
\]

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

\[
t_{ch} = 2.8 h_p - 23
\]

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

\[
k_2 = 1 - 0.018 h_p
\]

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

\[
t_{ch} = 0.07(h_{ins} - 20) \sqrt[3]{\rho_{ins}}
\]

Expression (3.10) applies also for two layers of gypsum plasterboard, where the outer layer is type F and the inner layer is type A or H.

\[
k_2 = 1 - 0.018 h_p
\]

where

- \( h_p \) is the layer thickness in millimetres.

3.4 Adhesives

(1) 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)

\[ d_{ef} = d_{char,n} + k_0 d_0 \]  \hspace{1cm} (4.1)

with

\[ d_0 = 7 \text{ mm} \]

\[ d_{char,n} \] 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.

![Figure 4.1 — Definition of residual cross section and effective cross section](image-url)
Table 4.1 — Determination of $k_0$ for unprotected surfaces with $t$ in minutes (see figure 4.1a)

<table>
<thead>
<tr>
<th>$t$ (min)</th>
<th>$k_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t &lt; 20$</td>
<td>$t/20$</td>
</tr>
<tr>
<td>$t \geq 20$</td>
<td>1,0</td>
</tr>
</tbody>
</table>

(2) For protected surfaces with $t_{ch} > 20$ minutes or $t_i > 20$ 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 $t = t_i$, whichever is the smallest, see figure 4.2b. For protected surfaces with $t_{ch} \leq 20$ minutes or $t_i \leq 20$ minutes table 4.1 applies.

(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 $t \geq 20$ minutes, the modification factor for fire $k_{mod,fi}$ see 2.3 (1)P, should be taken as follows (see figure 4.3):

- for bending strength:
  
  $$k_{mod,fi} = 1,0 - \frac{1}{200} \frac{p}{A_t}$$  

(4.2)

- for compressive strength:

  $$k_{mod,fi} = 1,0 - \frac{1}{125} \frac{p}{A_t}$$  

(4.3)
− for tensile strength and modulus of elasticity:

\[ k_{\text{mod,fi}} = 1.0 - \frac{1}{330} \frac{p}{A_r} \]  

(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 m²

(4) For unprotected and protected members, for time \( t = 0 \) the modification factor for fire should be taken as \( k_{\text{mod,fi}} = 1 \). For unprotected members, for \( 0 \leq t \leq 20 \) 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](image)

4.3.4 Mechanically jointed members

(1) 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

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

\( K_{fi} \) is the slip modulus in the fire situation in N/mm

\( K_u \) is the slip modulus at normal temperature for the ultimate limit state according to EN 1995-1-1 2.2.2(2) in N/mm

\( \eta_f \) is a conversion coefficient according to table 4.2.

Table 4.2 — Conversion factor \( \eta_f \)

<table>
<thead>
<tr>
<th>Connections</th>
<th>( \eta_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nails</td>
<td>0.2</td>
</tr>
<tr>
<td>Bolts, dowels, connectors</td>
<td>0.67</td>
</tr>
</tbody>
</table>

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) 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) 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

\[ t_{\text{ins}} \geq t_{\text{req}} \]  \hspace{1cm} (5.1)

where

- \( t_{\text{ins}} \): is the time to reach the temperature increase on the unexposed side given in 2.1.2(3);
- \( t_{\text{req}} \): 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 $t_{\text{ins}}$ may be calculated as the sum of contributions of the individual layers used in the construction, according to

$$t_{\text{ins}} = \sum_i t_{\text{ins},0,i} k_{\text{pos}} k_j$$  \hspace{1cm} (5.2)

where

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

$k_{\text{pos}}$ 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>
<thead>
<tr>
<th>Temperature rise on unexposed side</th>
<th>Heat transfer path according to figure 5.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>General construction</td>
<td>140</td>
</tr>
<tr>
<td>Joints</td>
<td>180</td>
</tr>
<tr>
<td>Services</td>
<td>180</td>
</tr>
</tbody>
</table>
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/m³
     \[ t_{\text{ins,0}} = 0.95 \, h_p - 0.3 \, t_{\text{ins,0}} = 0.95 \, h_p \] (5.3)
   – for particleboard and fibreboard with a characteristic density greater or equal 600 kg/m³
     \[ t_{\text{ins,0}} = 1.1 \, h_p + 0.4 \, t_{\text{ins,0}} = 1.1 \, h_p \] (5.4)
   – for wood panelling with a characteristic density greater or equal 400 kg/m³
     \[ t_{\text{ins,0}} = 0.5 \, h_p + 0.2 \, t_{\text{ins,0}} = 0.5 \, h_p \] (5.5)
   – for gypsum plasterboard of type A, F, R and H
     \[ t_{\text{ins,0}} = 1.4 \, h_p + 0.4 \, t_{\text{ins,0}} = 1.4 \, h_p \] (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

---

**Figure 5.1 — Illustration of heat transfer paths through separating construction**

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
\[ t_{\text{ins,0},i} = 0,2 h_{\text{ins}} k_{\text{dens}} \]  

– for glass fibre \[ t_{\text{ins,0},i} = 0,1 h_{\text{ins}} k_{\text{dens}} \]

where
\( h_{\text{ins}} \) is the insulation thickness in millimetres
\( k_{\text{dens}} \) 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 \( t_{\text{ins,0}} = 5,0 \text{ min.} \)

(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:
\[ k_{\text{pos}} = \min \left( 0,02 h_p + 0,54, 1 \right) \]  
\[ k_{\text{pos}} = 0,07 h_p - 0,17 \]

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 \( k_j = 1 \)

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 \( t_{\text{ins,0}} \) 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 $k_{\text{dens}}$ for cavity insulation materials

<table>
<thead>
<tr>
<th>Cavity material</th>
<th>Density kg/m$^3$</th>
<th>$k_{\text{dens}}$ $^a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glass fibre</td>
<td>15</td>
<td>0,9</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>1,0</td>
</tr>
<tr>
<td></td>
<td>26</td>
<td>1,2</td>
</tr>
<tr>
<td>Rock fibre</td>
<td>26</td>
<td>1,0</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>1,1</td>
</tr>
</tbody>
</table>

$^a$ For intermediate densities, linear interpolation may be applied.

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

<table>
<thead>
<tr>
<th>Panel</th>
<th>Density kg/m$^3$</th>
<th>Thickness mm</th>
<th>Position coefficient for panels backed by</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>rock or glass fibre</td>
</tr>
<tr>
<td>Plywood</td>
<td>≥ 450</td>
<td>9 - 25</td>
<td>Expression (5.9)</td>
</tr>
<tr>
<td>Particleboard, fibreboard</td>
<td>≥ 600</td>
<td>9 - 25</td>
<td>Expression (5.9)</td>
</tr>
<tr>
<td>Wood panelling</td>
<td>≥ 400</td>
<td>15 - 19</td>
<td>Expression (5.9)</td>
</tr>
<tr>
<td>Gypsum plasterboard</td>
<td>≥ 740</td>
<td>9 - 15</td>
<td>Expression (5.9)</td>
</tr>
<tr>
<td>type H</td>
<td>≥ 680</td>
<td></td>
<td></td>
</tr>
<tr>
<td>type A</td>
<td>≥ 830</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 5.4 — Position coefficients $k_{pos}$ for single layered panels on the unexposed side

<table>
<thead>
<tr>
<th>Panel</th>
<th>Density $\text{kg/m}^3$</th>
<th>Thickness of panel on unexposed side $\text{mm}$</th>
<th>Position coefficient for panels preceded by Glass fibre</th>
<th>Rock fibre of thickness</th>
<th>Void</th>
</tr>
</thead>
<tbody>
<tr>
<td>Painted panels</td>
<td></td>
<td></td>
<td>Expression (5.10)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plywood</td>
<td>$\geq 450$</td>
<td>9 -25</td>
<td>1,5</td>
<td>1,5</td>
<td>3,9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4,9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0,6</td>
</tr>
<tr>
<td>Particleboard, fibreboard</td>
<td>$\geq 600$</td>
<td>9 -25</td>
<td>1,5</td>
<td>1,5</td>
<td>3,9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4,9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0,6</td>
</tr>
<tr>
<td>Wood panelling</td>
<td>$\geq 400$</td>
<td>15</td>
<td>0,45</td>
<td>1,5</td>
<td>3,9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>19</td>
<td>0,67</td>
<td></td>
<td>4,9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0,6</td>
</tr>
<tr>
<td>Gypsum plasterboard type H</td>
<td>$\geq 740$</td>
<td>9</td>
<td>0,46</td>
<td>1,5</td>
<td>3,0</td>
</tr>
<tr>
<td></td>
<td>$\geq 680$</td>
<td>12,5</td>
<td>0,74</td>
<td></td>
<td>4,9</td>
</tr>
<tr>
<td></td>
<td>$\geq 830$</td>
<td>15</td>
<td>0,88</td>
<td></td>
<td>0,7</td>
</tr>
</tbody>
</table>
Table 5.5 — Position coefficients $k_{pos}$ for walls with double layered panels

<table>
<thead>
<tr>
<th>Construction:</th>
<th>Layer number and material</th>
<th>Layer number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>1, 2, 4, 5, 3</td>
<td>Wood-based panel</td>
<td>0,7</td>
</tr>
<tr>
<td>1, 2, 4, 5, 3</td>
<td>Gypsum plasterboard type A or H</td>
<td>1,0</td>
</tr>
<tr>
<td>1, 5, 2, 4, 3</td>
<td>Gypsum plasterboard type A or H</td>
<td>1,0</td>
</tr>
<tr>
<td>1, 5, 2, 4, 3</td>
<td>Wood-based panel</td>
<td>1,0</td>
</tr>
<tr>
<td>1, 2, 4, 5, 3</td>
<td>Rock fibre batts</td>
<td>0,7</td>
</tr>
<tr>
<td>1, 2, 4, 5, 3</td>
<td>Gypsum plasterboard type A or H</td>
<td>1,0</td>
</tr>
<tr>
<td>1, 5, 2, 4, 3</td>
<td>Gypsum plasterboard type A or H</td>
<td>1,0</td>
</tr>
<tr>
<td>1, 5, 2, 4, 3</td>
<td>Wood-based panel</td>
<td>1,0</td>
</tr>
<tr>
<td>1, 5, 2, 4, 3</td>
<td>Gypsum plasterboard type A or H</td>
<td>1,0</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Joint type</th>
<th>$k_j$</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>( \leq 1 \text{ mm} )</td>
</tr>
<tr>
<td>b</td>
<td>( \leq 1 \text{ mm} )</td>
</tr>
<tr>
<td>c</td>
<td>( \leq 1 \text{ mm} )</td>
</tr>
<tr>
<td>d</td>
<td>( \leq 1 \text{ mm} )</td>
</tr>
<tr>
<td>e</td>
<td>( \leq 1 \text{ mm} )</td>
</tr>
</tbody>
</table>
Table 5.7 — Joint coefficient $k_j$ taking into account the effect of joints in panels of gypsum plasterboard not being backed by battens

<table>
<thead>
<tr>
<th>Joint type</th>
<th>Type</th>
<th>$k_j$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>≤ 1 mm</td>
<td>A, H, F</td>
</tr>
<tr>
<td>2</td>
<td>≤ 1 mm</td>
<td>A, H, F</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Provisionsa</th>
<th>Time of fire resistance</th>
<th>Provisionsa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth nails</td>
<td>15</td>
<td>$d \geq 2,8 \text{ mm}$</td>
</tr>
<tr>
<td>Screws</td>
<td>15</td>
<td>$d \geq 3,5 \text{ mm}$</td>
</tr>
<tr>
<td>Bolts</td>
<td>15</td>
<td>$t_1 \geq 45 \text{ mm}$</td>
</tr>
<tr>
<td>Dowels</td>
<td>20</td>
<td>$t_1 \geq 45 \text{ mm}$</td>
</tr>
<tr>
<td>Connectors according to EN 912</td>
<td>15</td>
<td>$t_1 \geq 45 \text{ mm}$</td>
</tr>
</tbody>
</table>

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

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

where

$\beta_0$ 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

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

(3) The factor $k_{\text{flux}}$ should be taken as $k_{\text{flux}} = 1.5$

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

$$t_{\text{ch}} \geq t_{\text{req}} - 0.5 t_{\text{fi,d}}$$

(6.2)

where

- $t_{\text{ch}}$ is the time of start of charring according to 3.3.3;
- $t_{\text{req}}$ is required the time of standard fire resistance;
- $t_{\text{fi,d}}$ is the inherent fire resistance of the unprotected connection according to table 6.1 loaded with design effect of actions $E_{d,\text{fr}}$.

(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

$$t_{\text{ch}} \geq t_{\text{req}} - 1.2 t_{\text{fi,d}}$$

(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 H should 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>
<thead>
<tr>
<th></th>
<th>( b_{st} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unprotected edges in general</td>
<td>R 30 ( \geq 200 \text{ mm} )</td>
</tr>
<tr>
<td></td>
<td>R 60 ( \geq 280 \text{ mm} )</td>
</tr>
<tr>
<td>Unprotected edges on one or two sides</td>
<td>R 30 ( \geq 120 \text{ mm} )</td>
</tr>
<tr>
<td></td>
<td>R 60 ( \geq 280 \text{ mm} )</td>
</tr>
</tbody>
</table>

(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

\[ F_{Rk,fi} = \eta_{\text{conn}} F_{Rk} \]  

with \[ \eta_{\text{conn}} = e^{-k_{l,d}} \]  

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
where

\( \eta_{\text{conn}} \) 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

\( t_{\text{fi,d}} \) 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

\[
t_{\text{fi,d}} = - \frac{1}{k} \ln \frac{\eta_{\text{fi}} \gamma_{M,\text{fi}}}{\gamma_{M,\text{fi}}} \tag{6.7}
\]

where

\( k \) is a parameter given in table 6.3

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

\( \gamma_{M,\text{fi}} \) is the partial factor for the connection, see EN 1995-1-1, subclause 2.2.2;

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

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

### Table 6.3 — Parameters \( k \)

<table>
<thead>
<tr>
<th>Connection with</th>
<th>( k )</th>
<th>Maximum time of validity for unprotected connection min</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth nails</td>
<td>0.08</td>
<td>20</td>
</tr>
<tr>
<td>Bolts wood-to-wood with ( d \geq 12 \text{ mm} )</td>
<td>0.065</td>
<td>25</td>
</tr>
<tr>
<td>Bolts steel-to-wood with ( d \geq 12 \text{ mm} )</td>
<td>0.085</td>
<td>25</td>
</tr>
<tr>
<td>Dowels wood-to-wood(^a) with ( d \geq 12 \text{ mm} )</td>
<td>0.04</td>
<td>40</td>
</tr>
<tr>
<td>Dowels steel-to-wood(^a) with ( d \geq 12 \text{ mm} )</td>
<td>0.085</td>
<td>25</td>
</tr>
<tr>
<td>Connectors</td>
<td>0.065</td>
<td>25</td>
</tr>
</tbody>
</table>

\(^a\) 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 \( a_{\text{fi}} \) (see figure 6.1) which should be taken as

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

where
\( \beta_0 \) is the charring rate according to table 3.1
\( t_{\text{req}} \) is the required standard fire resistance
\( t_{\text{fi,d}} \) 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 \( t_{\text{fi,d}} \) 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 \( a_{f_i} \) according to expression (6.1). For fire resistances greater than 30 minutes, however, the end distances should be increased by \( 2a_{f_i} \). 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 \( a_{f_i} \) according to expression (6.1) with \( t_{d,0} = 5 \text{ min.} \)

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
\[
\begin{align*}
a_2 & \geq a_1 + 40 \\
a_3 & \geq a_1 + 20
\end{align*}
\]
where \( a_1, a_2 \) and \( a_3 \) are distances in millimetres, the conversion function \( \eta_{\text{conn}} \) for the reduction of the mechanical resistance of the screw in the fire situation should be taken as

\[
\eta_{\text{conn}} = 0 \quad \text{for } a_1 \leq 0.6 \ t_{\text{fi,d}}
\]
\[ n_{\text{conn}} = \frac{0.44 a_1 - 0.264 t_{\text{fi,d}}}{0.2 t_{\text{fi,d}} + 5} \quad \text{for } 0.6 t_{\text{fi,d}} \leq a_1 \leq 0.8 t_{\text{fi,d}} + 5 \quad (6.11\text{b}) \]

\[ n_{\text{conn}} = \frac{0.56 a_1 - 0.36 \cdot t_{\text{fi,d}} + 7.32}{0.2 t_{\text{fi,d}} + 23} \quad \text{for } 0.8 t_{\text{fi,d}} + 5 \leq a_1 \leq t_{\text{fi,d}} + 28 \quad (6.11\text{c}) \]

\[ n_{\text{conn}} = 1.0 \quad a_1 \geq t_{\text{fi,d}} + 28 \quad (6.11\text{d}) \]

where

- \( a_1 \) is the side cover in mm, see figure 6.4
- \( t_{\text{fi,d}} \) is the required time of fire resistance in minutes

(3) The conversion function \( n_{\text{conn}} \) for side covers \( a_2 = a_1 \) and \( a_3 \geq a_1 + 20 \) mm should be calculated according to expression (6.11) where \( t_{\text{fi,d}} \) is replaced by 1.25 \( t_{\text{fi,d}} \).

---

**Figure 6.4 — Cross section and definition of distances**

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

\[
 t_{p,\text{min}} = \max \left\{ \frac{l_p}{70}, 8 \right\}
\]  

(7.1)

where

- \( t_{p,\text{min}} \) 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/m³.

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.
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 \( \beta_{\text{par}} \) during the heating phase of a parametric fire curve is given by

\[
\beta_{\text{par}} = 1.5 \beta_n \frac{5 O k_b - 0.04}{4 O k_b + 0.08}
\]  

(A.1)

with

\[
O = \frac{A}{A_t} \sqrt{h_{\text{eq}}}
\]  

(A.2)

\[
k_b = \frac{1160}{\sqrt{\lambda \rho c}}
\]  

(A.3)

and

\[
h_{\text{eq}} = \sum \frac{A_i h_i}{A}
\]  

(A.4)

where

- \( O \) is the opening factor in \( \text{m}^{1/2} \);
- \( \beta_n \) is the notional charring rate in \( \text{mm/min} \);
- \( A \) is the total area of vertical openings (windows etc.) in \( \text{m}^2 \);
- \( A_t \) is the total area of floors, walls and ceilings that enclose the fire compartment in \( \text{m}^2 \);
- \( A_i \) is the area of vertical opening "i" in \( \text{m}^2 \);
- \( h_{\text{eq}} \) is the weighted average of heights of all vertical openings (windows etc.) in metres;
- \( h_i \) is the height of vertical opening "i" in metres;
- \( k_b \) 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);
- \( \lambda \) is the thermal conductivity of boundary of enclosure in \( \text{Wm}^{-1}\text{K}^{-1} \);
- \( \rho \) is the density of boundary enclosure in \( \text{kg/m}^3 \);
- \( c \) is the specific heat of boundary of enclosure in \( \text{Jkg}^{-1}\text{K}^{-1} \).
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

\[ d_{\text{char}} = 2 \beta_{\text{par}} t_0 \]  

(A.5)

with

\[ t_0 = 0.009 \frac{q_{l,d}}{O} \]  

(Changed corresponding to prEN 1991-1-2)  

(A.6)

where

\( t_0 \) is in minutes;

\( q_{l,d} \) is the design fire load density related to the total area of floors, walls and ceilings which enclose the fire compartment in MJ/m², 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

\[ t_0 \leq 40 \text{ min} \]

\[ d_{\text{char}} \leq \frac{b}{4} \]

\[ d_{\text{char}} \leq \frac{h}{4} \]

where

\( b \) is the width of the cross section;

\( h \) is the depth of the cross section.

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

$$k_{mod,fi} = 1.0 - 3.2 \frac{d_{char,n}}{b}$$

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

$$k_{mod,fi} = \frac{W_{ef}}{W_r}$$

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](image)

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

<table>
<thead>
<tr>
<th>Temperature (^{\circ})C</th>
<th>Conductivity (\text{Wm}^{-1}\text{K}^{-1})</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0,12</td>
</tr>
<tr>
<td>200</td>
<td>0,15</td>
</tr>
<tr>
<td>350</td>
<td>0,07</td>
</tr>
<tr>
<td>500</td>
<td>0,09</td>
</tr>
<tr>
<td>800</td>
<td>0,35</td>
</tr>
<tr>
<td>1200</td>
<td>1,50</td>
</tr>
</tbody>
</table>
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

<table>
<thead>
<tr>
<th>Temperature °C</th>
<th>Specific heat capacity kJ kg(^{-1}) K(^{-1})</th>
<th>Density ratio –</th>
<th>ω</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1,53</td>
<td>1 + ω</td>
<td></td>
</tr>
<tr>
<td>99</td>
<td>1,77</td>
<td>1 + ω</td>
<td></td>
</tr>
<tr>
<td>99</td>
<td>13,60</td>
<td>1 + ω</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>13,50</td>
<td>1,00</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>2,12</td>
<td>1,00</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>2,00</td>
<td>1,00</td>
<td></td>
</tr>
<tr>
<td>250</td>
<td>1,62</td>
<td>0,93</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>0,71</td>
<td>0,76</td>
<td></td>
</tr>
<tr>
<td>350</td>
<td>0,85</td>
<td>0,52</td>
<td></td>
</tr>
<tr>
<td>400</td>
<td>1,00</td>
<td>0,38</td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>1,40</td>
<td>0,28</td>
<td></td>
</tr>
<tr>
<td>800</td>
<td>1,65</td>
<td>0,26</td>
<td></td>
</tr>
<tr>
<td>1200</td>
<td>1,65</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

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).

![Figure C.1 — Notional residual cross section of timber frame member protected by cavity insulation](image)

Key:
1 Notional residual cross section
2 Notional char layer

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

\[
\beta_n = k_s k_2 k_n \beta_0 \quad \text{for } t_{ch} \leq t \leq t_f \quad (C.1)
\]

\[
\beta_n = k_s k_3 k_n \beta_0 \quad \text{for } t \geq t_{bf} \quad (C.2)
\]

with

\[k_n = 1.5\]

where

\(\beta_n\) 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;

$\beta_0$ 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**

<table>
<thead>
<tr>
<th>$b$ (mm)</th>
<th>$k_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>38</td>
<td>1.4</td>
</tr>
<tr>
<td>45</td>
<td>1.3</td>
</tr>
<tr>
<td>60</td>
<td>1.1</td>
</tr>
</tbody>
</table>

(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:

$$k_2 = 1.05 - 0.0073 h_p$$  \hspace{1cm} (C.3)

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

$$k_2 = 0.86 - 0.0037 h_p$$  \hspace{1cm} (C.4)

where $h_p$ is the total thickness of panels in millimetres.

![Figure C.2 — Joint configurations of linings with one or two layers](image_url)

**Key**

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

(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

$$k_3 = 0.036 t_f + 1$$  \hspace{1cm} (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

\[
t_{ch} = t_f
\]

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

\[
t_f = t_{ch} + \frac{l_f - l_{a,min} - h_p}{k_s k_2 k_n k_j \beta_0}
\]

with

\[
k_j = 1,0 \quad \text{for joint configurations 0 and 2}
\]

\[
k_j = 1,15 \quad \text{for joint configurations 1 and 3}
\]

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;
- \( \beta_0 \) 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

\[ t_{sf} = t_f + \frac{l_f - l_{a,min} - k_f k_e k_n \beta_0 (t_f - t_{ch}) - t_s}{k_f k_e k_n \beta_0} \]  

(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.

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

\[ k_{mod,f,m} = a_0 - a_1 \frac{d_{\text{char},n}}{h} \]  

(C.11)

where

- \( a_0, a_1 \) are values given in table C.1 and C.2;
- \( d_{\text{char},n} \) is the notional charring depth according to expression (3.2) with \( \beta_i \) according to expression (C.1) and (C.2);
- \( h \) is the depth of the joist or the stud.
### Table C.1 — Values of \( a_0 \) and \( a_1 \) for reduction of strength for assemblies exposed to fire on one side

<table>
<thead>
<tr>
<th>Case</th>
<th>( h ) mm</th>
<th>( a_0 )</th>
<th>( a_1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Bending strength with exposed side in tension</td>
<td>95</td>
<td>0,60</td>
<td>1,04</td>
</tr>
<tr>
<td></td>
<td>145</td>
<td>0,68</td>
<td>1,10</td>
</tr>
<tr>
<td></td>
<td>195</td>
<td>0,73</td>
<td>1,14</td>
</tr>
<tr>
<td></td>
<td>220</td>
<td>0,76</td>
<td>1,14</td>
</tr>
<tr>
<td>2 Bending strength with exposed side in compression</td>
<td>95</td>
<td>0,46</td>
<td>0,83</td>
</tr>
<tr>
<td></td>
<td>145</td>
<td>0,55</td>
<td>0,89</td>
</tr>
<tr>
<td></td>
<td>195</td>
<td>0,65</td>
<td>1,07</td>
</tr>
<tr>
<td></td>
<td>220</td>
<td>0,67</td>
<td>1,05</td>
</tr>
<tr>
<td>3 Compressive strength</td>
<td>95</td>
<td>0,46</td>
<td>0,83</td>
</tr>
<tr>
<td></td>
<td>145</td>
<td>0,55</td>
<td>0,89</td>
</tr>
<tr>
<td></td>
<td>195</td>
<td>0,65</td>
<td>1,07</td>
</tr>
<tr>
<td></td>
<td>220</td>
<td>0,67</td>
<td>1,05</td>
</tr>
</tbody>
</table>

\( a \) 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

<table>
<thead>
<tr>
<th>Case</th>
<th>( h ) mm</th>
<th>( a_0 )</th>
<th>( a_1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Compressive strength</td>
<td>145</td>
<td>0,39</td>
<td>3,65</td>
</tr>
</tbody>
</table>

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

\[
k_{\text{mod,fi,E}} = b_0 - b_1 \frac{d_{\text{char,n}}}{h}
\]

(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 \( \beta_n \) according to expression (C.1) and (C.2);
- \( h \) is the depth of the joist.
Table C.3 — Values\textsuperscript{*} of $b_0$ and $b_1$ for reduction of modulus of elasticity for walls exposed to fire on one side

<table>
<thead>
<tr>
<th>Case</th>
<th>$h$ mm</th>
<th>$b_0$</th>
<th>$b_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Buckling perpendicular to wall plane</td>
<td>95</td>
<td>0,50</td>
<td>1,77</td>
</tr>
<tr>
<td></td>
<td>145</td>
<td>0,60</td>
<td>1,88</td>
</tr>
<tr>
<td></td>
<td>195</td>
<td>0,68</td>
<td>1,73</td>
</tr>
<tr>
<td>2 Buckling in plane of wall</td>
<td>95</td>
<td>0,54</td>
<td>1,11</td>
</tr>
<tr>
<td></td>
<td>145</td>
<td>0,66</td>
<td>1,23</td>
</tr>
<tr>
<td></td>
<td>195</td>
<td>0,73</td>
<td>1,41</td>
</tr>
</tbody>
</table>

* 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\textsuperscript{*} of $b_0$ and $b_1$ for reduction of modulus of elasticity for walls exposed to fire on both sides

<table>
<thead>
<tr>
<th>Case</th>
<th>$h$ mm</th>
<th>$b_0$</th>
<th>$b_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Buckling perpendicular to wall plane</td>
<td>145</td>
<td>0,37</td>
<td>4,2</td>
</tr>
<tr>
<td>2 Buckling in plane of wall</td>
<td>145</td>
<td>0,44</td>
<td>4,9</td>
</tr>
</tbody>
</table>
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)

\[
k_\phi = \frac{-0.70 \Theta_i + 114}{100} \quad \text{for } 20 \leq \Theta_i \leq 100^\circ C \quad (6.12)
\]

\[
k_\phi = \frac{-0.22 \Theta_i + 66}{100} \quad \text{for } 100 \leq \Theta_i \leq 300^\circ C \quad (6.13)
\]

where \( \Theta \) is the local temperature in the timber in °C

![Figure 6.5 — Reduction factor for withdrawal shear strength](image)

(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:

\[
\Theta = 20 + 280 \left( \frac{R_0}{y} \right) y \quad (6.14)
\]

with

\[
\alpha = 0.025 \, t + 1.75 \quad (6.15)
\]

where

- \( \Theta \) 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:

\[ \Theta = 20 + 280 \left( \beta_0 \right) t^y \left\{ \left( \frac{1}{y} \right)^a + \left( \frac{1}{b - y} \right)^a + \left( \frac{1}{z} \right)^a \right\} \]  

(6.16)

where

- \( \Theta \) is the temperature in a point P in \( ^\circ \text{C} \)
- \( y, z \) are the co-ordinates of point P in millimetres
- \( t \) is the time in minutes
- \( \alpha \) is given by expression (6.15)

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

\[ F_{Rd,fi} = \pi \cdot d \cdot k_{fi} \cdot f_{v,k} \cdot \gamma_{M,fi} \cdot \sum_{i=1}^{n} \left\{ k_{\Theta,i} \cdot \Delta l_i \right\} \]  

(6.17)

where

- \( \Theta \) is the temperature of the element i in \( ^\circ \text{C} \), see figure 6.6;
- \( f_{v,k} \) is the characteristic shear strength of timber;
- \( k_{fi} \) should be taken as for solid timber from table 2.1;
- \( \gamma_{M,fi} \) is the partial factor for timber in fire;
- \( \Delta l_i \) 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;
- \( k_{\Theta,i} \) 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.

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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