Fire safety in timber buildings

Technical guideline for Europe

SP Report 2010:19

Excerpt of chapters 5-7 on Structural fire design

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Technical guideline for Europe

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Guideline Summary

Fire Safety in Timber Buildings is the very first Europe-wide technical guideline on the fire safe use of wood products and timber structures in buildings. It is one of the main outcomes of the collaborative European WoodWisdom-Net FireInTimber research project with partners from Austria, Estonia, Finland, Germany, France, Norway, Sweden, Switzerland and UK.

The guideline has been developed with the needs of architects, engineers, appropriate university departments, authorities and building industries in mind, providing information on and showcasing the fire-safe use of timber and wood products in buildings. It aims to provide the latest scientific knowledge on the fire safety of timber structures at European level. The guideline covers the use of design codes (such as the fire part of Eurocode 5), European fire standards and classifications and principles of performance-based design, as well as practical case studies and worked examples.

The guideline focus on structural fire protection in timber structures by providing detailed guidance on load-bearing and separating functions of timber structures under standard fire exposure. New design methods are presented: they have been developed recently, and will be potential input to the next revision of Eurocode 5. Representing state-of-the art knowledge, they have been included here for designers to trial and benefit from. The guideline also includes information on reaction to fire performance of wood products according to the latest European standards. The importance of proper detailing in building design and of quality of workmanship and inspection at building sites is stressed, and practical solutions offered. Means of active fire protection are introduced, and their contribution to meeting fire safety objectives explained.

Exciting new possibilities have recently been noted in timber buildings, with multi-storey applications and large-scale timber façade solutions being increasingly used throughout Europe. Whilst these applications are founded on a long history and tradition, only increased new knowledge of fire safety design has made these advances possible.

Chapter 1, Timber buildings, provides a short introduction to the long-term use of timber buildings and their renaissance in recent years as a sustainable solution to achieving environmental goals. It also describes the legacy of history and traditions and new opportunities to build multi-storey timber buildings based on new knowledge for fire safety design.

Chapter 2, Fire safety in buildings, gives an overview of the basic concepts of fire safety in buildings. It presents information on fire behaviour, fire loads, fire scenarios and fire safety objectives. Means of fulfilling the fire safety objectives are described, for use in all buildings and as a basis for the design solutions in this guideline.

Chapter 3, European system for construction products, presents an overview of the European system for fire safety in buildings, based on the Construction Products Directive (CPD) and its essential requirements. These requirements are mandatory for all European countries. They include the classification systems for reaction to fire of building products, fire resistance of structural elements, external fire performance of roofs, fire protection ability of claddings and structural Eurocodes. Descriptions of how these requirements are applied to wood products and timber structures are given in the following chapters.

Chapter 4, Wood products as linings, floorings, claddings and façades, presents the reaction-to-fire performance of wood products according to the European classification system. A wide range of products is included: wood-based panels, structural timber, glued laminated timber (glulam), solid wood panelling and wood flooring. A new system for the durability of the reaction-to-fire performance of wood products is explained and put into context, and K class performance is presented for wood claddings with fire protection ability.

In addition to reaction-to-fire performance, some countries have further requirements for façade claddings, for which at present no European harmonised solution exists. Best practice and state-of-the art information on fire scenarios for facades are presented.
Chapter 5, **Separating timber structures**, presents the basic requirements, calculation methods based on component additive design and the Eurocode 5 design method. It also presents an improved design method from recent research as potential input for future revisions of Eurocode 5, and practical examples on how to use the method.

Chapter 6, **Load-bearing timber structures**, introduces the design methods for verification of the structural stability of timber structures in the event of fire, applying the classification for Criterion R for fire resistance (load-bearing function). Reference is made to Eurocode 5 with respect to charring and strength and stiffness parameters. Alternative design models are presented, as well as design methods for new timber structures, outside the present scope of Eurocode 5.

Chapter 7, **Timber connections**, overviews the basic requirements for timber connections. The calculation methods in Eurocode 5 are complemented with state-of-the-art design methods, the result of recent research. Both timber-to-timber and steel-to-timber connections are included. The models are described and worked examples presented.

Chapter 8, **Fire stops, service installations and detailing in timber structures**, deals with the need for adequate detailing in the building structure to prevent fire spread within the building elements to other parts of the building. Special attention is paid to basic principles, fire stops, element joints and building services installations. Several practical examples of detailing in timber structures are included.

Chapter 9, **Novel products and their implementation**, is aimed primarily at product developers. It describes guidelines for introducing novel structural materials and products. The basic performance requirements and potential solutions for insulating materials, encasing claddings and board materials, thin thermal barriers and fire-retardant wood products are included. The innovation process from idea to approved product ready for the market is outlined.

Chapter 10, **Active fire protection**, describes how such protection is used to achieve a more flexible fire safety design of buildings and an acceptable level of fire safety in large and/or complex buildings. The chapter introduces common active fire protection systems, including fire detection and alarm systems, fire suppression and smoke control systems. Sprinkler installation provides special benefits for an increased use of wood in buildings especially on visible surfaces, e.g. internal linings and external facade claddings.

Chapter 11, **Performance-based design**, describes the basic principles of performance-based design, requirements and verification. Fire risk assessment principles are described in terms of objectives, fire safety engineering design, design fires, calculation/simulation methods and statistics. A case study of a probabilistic approach for structural fire safety is also included.

Chapter 12, **Quality of construction workmanship and inspection**, describes the need for execution and control of workmanship to ensure that the planned fire safety precautions are built in. It also emphasises the need for fire safety at building sites, when all fire safety measures are not yet in place.

The guideline summary is published also as a separate document, SP Info 2010:15, with extended information and illustrations. That summary document is available in several languages: English, Estonian, Finnish, French, German, Italian and Swedish.
5 Separating timber structures

Separating structures are used to limit the spread of fire from one fire cell, e.g. from one compartment, to another. This chapter presents the basic requirements, calculation methods based on component additive design and the Eurocode 5 design method. It also presents an improved design method from recent research as potential input for future revisions of Eurocode 5 and practical examples on how to use the method.

5.1 General

This section gives guidance for the fire design of separating timber structures. Reference is made to EN 1995-1-2 [5.1], which gives a calculation method for the verification of the separation function of timber structures (see Annex E of EN 1995-1-2). In addition, it presents a new design method developed from recent research. The calculation method given in Annex E of EN 1995-1-2 is informative, and may not be applicable in all European states. Depending on national regulations, the new design method given in the following may need agreement by national authorities. Hence the content of this section should be regarded as the state of the art, and the new design method as potential input for future revisions of EN 1995-1-2 [5.1].

5.2 Basic requirements for fire compartmentation

The main objective of structural fire safety measures is to restrict the spread of fire to the room of origin by guaranteeing the load-carrying capacity of the structure (Requirement on Mechanical Resistance R) and the separating function of walls and floors (Requirement on Insulation I and Integrity E) for the required period of time. The required period of time is normally expressed in terms of fire resistance, using fire exposure of the standard temperature-time curve, and is specified by the building regulations. While fire tests are still widely used for the verification of the fire resistance of timber structures, calculation models are becoming more and more common.

Concerning the basic requirements for fire compartmentation, EN 1995-1-2 [5.1] states:

“Where fire compartmentation is required, the elements forming the boundaries of the fire compartment shall be designed and constructed in such a way that they maintain their separating function during the relevant fire exposure. This shall include, when relevant, ensuring that:

– integrity failure does not occur (Criterion E),
– insulation failure does not occur (Criterion I), and
– thermal radiation from the unexposed side is limited.”

Criterion I (insulation) may be assumed to be satisfied where the average temperature rise over the whole of the non-exposed surface is limited to 140K, and the maximum temperature rise at any point on that surface does not exceed 180K (for fire exposure of the standard temperature-time curve), thus preventing ignition of objects in the neighbouring compartment. Criterion E (integrity) may be assumed to be satisfied when no sustained flaming or hot gases to ignite a cotton pad on the side of the construction not exposed to fire occur, or no cracks or openings in excess of certain dimensions exist. There is no risk of fire spread due to thermal radiation when Criterion I (insulation) is satisfied.

Criterion I (insulation) is clearly defined, and could be verified by heat transfer calculations instead of by testing if thermal material properties could be found as a function of temperature (conductivity and heat transfer). On the other hand, Criterion E (integrity) is mostly determined by testing, because calculations are still quite impossible (crack formation, dynamics of hot gases, etc.). For example, premature integrity failure may occur due to sudden failure of claddings or opening of gaps, which are often dependent on construction details such as fixings. However, extensive experience of full-scale testing of wall and floor assemblies made it possible to define some rules about detailing of wall and
Chapter 5 - Separating timber structures – Excerpt for JRC and CEN TC250/SC5

floor assemblies that have been included (for example) in EN 1995-1-2 [5.1]. Thus EN 1995-1-2 assumes that Criterion E (integrity) is satisfied where Criterion I (insulation) has been satisfied and panels remain fixed to the timber structure on the side not exposed to fire.

5.3 Calculation methods

In timber buildings, walls and floors are mostly built up by adding different layers to form an assembly. For the calculation of fire resistance with regard to the separation function of timber assemblies, component additive methods can be used. These methods are thus called component additive because they determine the fire resistance of a layered construction by adding the contribution of the different layers to obtain the fire resistance. Reference [5.2] presents and reviews calculation methods for verification of the separating performance of wall and floor assemblies as used in the United Kingdom [5.3], Canada [5.4] and Sweden [5.5, 5.6], as well as according to ENV 1995-1-2 [5.7]. The Swedish component additive method builds upon that described in ENV 1995-1-2 and the Canadian method by taking into account the influence of adjacent materials on the fire performance of each layer, and therefore describes the real fire performance more accurately.

The EN 1995-1-2 (Annex E) design method is based on modification of the Swedish component additive method by extending it to floors, including the effect of joints in claddings that are not backed by members, battens or panels, and applying some of the position coefficients to further test results that became available during the drafting of EN 1995-1-2. The EN 1995-1-2 design method is capable of considering claddings made of one or two layers of wood-based panels and gypsum plasterboard, and also voids or insulation-filled cavities. The insulation may be made of mineral wool.

5.4 The EN 1995-1-2 design method

The analysis method for the separating function of wall and floor assemblies is presented in EN 1995-1-2 Annex E, which is informative: this means that the method shall be used according to the National Annex in the country concerned.

EN 1995-1-2 requires verification that the time \( t_{\text{ins}} \) that it takes for the temperature to increase (starting from room temperature) by 140K/180K on the side of the member that is not exposed to fire is equal to or greater than the required fire resistance period \( t_{\text{req}} \) for the separating function of the member.

\[
t_{\text{ins}} \geq t_{\text{req}} \quad \text{[min]} \quad (5.1)
\]

The insulation time \( t_{\text{ins}} \) depends on the fire behaviour of the layers used in the assemblies, as well as the positions and joint configurations of the layers. For simplicity, the time \( t_{\text{ins}} \) can be calculated as the sum of the contributions of the individual layers to fire resistance, considering different heat transfer paths (see Figure 5.1).

\[
t_{\text{ins}} = \sum_{i=1}^{n} t_{\text{ins},i} \quad \text{[min]} \quad (5.2)
\]
These contributions depend firstly on the inherent insulation property of each layer, as given by the basic insulation value, and secondly on the position of the respective layer and the materials backing or preceding that layer (in the direction of the heat flux), as given by the position coefficient. Further, a joint coefficient is used in order to take into account the influence of joint configurations on the insulation time of layers with joints. Thus the contribution of each layer $t_{\text{ins},\text{i}}$ is calculated using the basic insulation value ($t_{\text{ins},\text{0},\text{i}}$), the position coefficient ($k_{\text{pos},\text{i}}$) and the joint coefficient ($k_{j,i}$).

$$t_{\text{ins},\text{i}} = t_{\text{ins},\text{0},\text{i}} \cdot k_{\text{pos},\text{i}} \cdot k_{j,i} \quad [\text{min}]$$

The basic insulation value corresponds to the contribution of a single layer to fire resistance without the influence of adjacent materials, and depends on the material and the thickness of the layer. EN 1995-1-2 gives equations for calculating the basic insulation values for the following materials:

**Panels:**
- Plywood ($\rho \geq 450 \text{ kg/m}^3$)
- Wood panelling ($\rho \geq 400 \text{ kg/m}^3$)
- Particleboard and fibreboard ($\rho \geq 600 \text{ kg/m}^3$)
- Gypsum plasterboard, types A, F, R and H

**Cavity insulations:**
- Stone wool ($26 \text{ kg/m}^3 \leq \rho \leq 50 \text{ kg/m}^3$)
- Glass wool ($15 \text{ kg/m}^3 \leq \rho \leq 26 \text{ kg/m}^3$)

The position coefficient considers the position of the layer within the assembly (in direction of the heat flux), because the layers preceding and backing the layer under consideration have an influence on its fire behaviour. EN 1995-1-2 gives tabulated data for position coefficients for wall and floor assemblies with claddings made of one or two layers of wood-based panels and gypsum plasterboards, and void or insulation-filled cavities. The position coefficients were determined based on testing of non-load bearing wall assemblies, both in full scale and in small scale. This means that the position coefficients that are given are limited to a small number of timber constructions.
5.5 Improved design method for separating function of timber constructions

5.5.1 Introduction

The design method for the verification of the separating function of wall and floor assemblies that is given in EN 1995-1-2 is based on input data that were deduced from a limited number of fire tests of wall assemblies, and covers therefore only a limited area of timber structures. For this reason, a research project on the separating function of timber assemblies was recently completed in Switzerland. As a final result, an improved design method for determining the separating function of timber structures has been developed, based on extensive experimental results and finite-element thermal analysis [5.8, 5.9]. The design method is capable of considering timber assemblies with an unlimited number of layers made of gypsum plasterboards, wood panels or combinations thereof. The cavity may be void or filled with mineral wool insulation. The design method considers the following materials:

- Solid timber with characteristic density ≥ 290 kg/m³
- Cross-laminated timber with characteristic density ≥ 290 kg/m³
- Laminated Veneer Lumber (LVL) with characteristic density ≥ 480 kg/m³
- Oriented Strand Board (OSB) according to EN 300 [5.10] with characteristic density ≥ 550 kg/m³
- Particleboards according to EN 312 [5.11] with characteristic density ≥ 500 kg/m³
- Fibreboards according to EN 622-2 [5.12], EN 622-3 [5.13] or EN 622-5 [5.14] with characteristic density ≥ 400 kg/m³
- Plywood according to EN 636 [5.15] with characteristic density ≥ 400 kg/m³
- Gypsum plasterboards Type A, H and F according to EN 520 [5.16]
- Gypsum fibre boards according to EN 15283-2 [5.17]
- Mineral wool insulation according to EN 13162 [5.18].

The developed design method is based on the additive component method given in EN 1995-1-2. The total fire resistance is therefore taken as the sum of the contributions from the different layers (claddings, void or insulated cavities), considering different heat transfer paths (see Figure 5.1) and according to their function and interaction (see Figure 5.2):

\[ t_{ins} = \sum_{i=1}^{i=n-1} t_{prot,i} + t_{ins,n} \] (5.4)

with \[ \sum_{i=1}^{i=n-1} t_{prot,i} \] Sum of the protection times \( t_{prot,i} \) of the layers (in the direction of the heat flux) preceding the last layer of the assembly on the side not exposed to fire [min]

\[ t_{ins,n} \] Insulation time \( t_{ins,n} \) of the last layer of the assembly on the side not exposed to fire [min]
Protection and insulation times of the layers can be calculated according to the following general equations, taking into account the basic values of the layers, the coefficients for the position of the layers in the assembly and the coefficients for the joint configurations:

\[
t_{\text{prot},i} = (t_{\text{prot},0,i} \cdot k_{\text{pos},\text{exp},i} \cdot k_{\text{pos},\text{unexp},i} + \Delta t_i) \cdot k_{j,i}
\]  
\[
t_{\text{ins},n} = (t_{\text{ins},0,n} \cdot k_{\text{pos},\text{exp},n} + \Delta t_n) \cdot k_{j,n}
\]

with
- \(t_{\text{prot},0,i}\) Basic protection value [min] of layer i (see Figure 5.2 and Table 5.1)
- \(t_{\text{ins},0,n}\) Basic insulation value [min] of the last layer n of the assembly on the side not exposed to fire (see Figure 5.2 and Table 5.1)
- \(k_{\text{pos},\text{exp},i}\), \(k_{\text{pos},\text{unexp},i}\) Position coefficient that takes into account the influence of layers preceding the layer considered (see Table 5.2)
- \(k_{\text{pos},\text{exp},n}\), \(k_{\text{pos},\text{unexp},n}\) Position coefficient that takes into account the influence of layers backing the layer considered (see Table 5.3)
- \(\Delta t_i\), \(\Delta t_n\) Correction time [min] for layers protected by Type F gypsum plasterboards as well as gypsum fibreboards (see Table 5.4)
- \(k_{j,i}\), \(k_{j,n}\) Joint coefficient (see Table 5.5)

The coefficients of the design method (basic values, correction time and position coefficients) were calculated by extensive finite-element thermal simulations based on physical models for heat transfer through separating multi-layered construction [5.9, 5.19]. The coefficients given by general equations permit replacement of the tabulated data given in EN 1995-1-2. The material properties used for the finite element thermal simulations were calibrated and validated by fire tests performed on unloaded specimens at Empa (Swiss Laboratories for Materials Testing and Research) in Duebendorf, using fire exposure of the standard temperature-time curve. The design method was verified with full-scale fire tests [5.20-5.24], in addition to 27 full-scale fire tests recently performed in Austria [5.25] on slabs and walls using current EN testing standards. The comparison between test results and the design method shows that the improved model is able to predict the fire resistance of timber assembly safely. All details with regard to the development and validation of the design method can be found in [5.9]. The developed design method significantly improves the EN 1995-1-2 design method, and permits verification of the separating function of a large number of common timber assemblies. The coefficients of the design method are explained below.
5.5.2 Basic values

The **basic insulation value** $t_{\text{ins},0}$ corresponds to the fire resistance of a single layer without the influence of adjacent materials, i.e. the average temperature rise over the whole of the non-exposed surface is limited to 140K, and the maximum temperature rise at any point of that surface does not exceed 180K (for fire exposure of the standard temperature-time curve). These temperature criteria safely prevent ignition of objects in the neighbouring compartment. The temperature of the layer at the beginning of the fire on the side exposed to fire, as well as on the non-exposed side, is assumed to be 20 °C. The basic insulation value can be assessed by tests in accordance with (for example) EN 1364-1 [5.26] or FE (finite element) thermal analysis. It should be noted that, for FE thermal analysis, only the 160 °C average temperature criterion is used (see Figure 5.3).

![Figure 5.3. Definition of the basic insulation value $t_{\text{ins},0}$ according to EN 1995-1-2.](image)

Single-layer wall and floor assemblies are only a limited application area for timber assemblies. Most structures consist of assemblies having two or more layers. The contribution of each preceding layer to the separating function of the construction is mainly protection of the following layers. It therefore seems more appropriate to introduce a **basic protection value** $t_{\text{prot},0}$, defined as the time until loss of the fire protective function, in a similar manner as for evaluation of fire-protective claddings of load-bearing timber structures in accordance with EN 13501-2 [5.27]. The testing method for fire-protective claddings in accordance with EN 14135 [5.28] is performed with 19 mm particleboard backing for the studied layer. The contribution to the fire protection of the cladding may be assumed to be satisfied where the average temperature rise over the whole exposed surface of the particleboard is limited to 250K, and the maximum temperature rise at any point on that surface does not exceed 270K. In the same way as for EN 13501-2, the definition of the basic protection value $t_{\text{prot},0}$ is illustrated in Figure 5.4. The temperature of the layer at the beginning of the analysis on the fire-exposed side as well as on the unexposed side is assumed to be 20 °C. For FE thermal analysis, only the average temperature criterion of 270 °C is used. It should be noted that EN 1995-1-2 gives rules for calculation of the start of charring $t_{\text{ch}}$ of timber surfaces protected by fire-protective claddings made of wood-based panels or wood panelling, as well as gypsum plasterboards, by assuming that charring starts at a temperature of 300 °C. Although the EN 13501-2 temperature criteria of 270/290 °C are slightly lower than 300 °C (i.e. conservative), the basic protection value $t_{\text{prot},0}$ has the same significance as the start of charring $t_{\text{ch}}$ defined by EN 1995-1-2.

![Figure 5.4. Definition of the basic protection value $t_{\text{prot},0}$ according to EN 13501-2.](image)
Table 5.1 gives the equations for calculation of the basic insulation value $t_{\text{ins,0},i}$ as well as the basic protection value $t_{\text{prot,0},i}$ for different materials that were systematically calculated using finite-element numerical simulations and verified with fire tests [5.8, 5.9]. Only the basic protection value $t_{\text{prot,0},i}$ is given for mineral wool insulation, as wall and floor assemblies with the insulation as the last layer of the assembly are rarely used in buildings.

Table 5.1. Basic insulation value $t_{\text{ins,0},i}$ and basic protection value $t_{\text{prot,0},i}$ for different materials. For mineral wool insulation, only the basic insulation value $t_{\text{ins,0},i}$ and basic protection value $t_{\text{prot,0},i}$ is given, as wall and floor assemblies with the insulation as last layer of the assembly are rarely used in buildings.

<table>
<thead>
<tr>
<th>Material</th>
<th>Basic insulation value $t_{\text{ins,0},i}$ [min]</th>
<th>Basic protection value $t_{\text{prot,0},i}$ [min]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gypsum plasterboard, gypsum fibre board</td>
<td>$24 \cdot \left( \frac{h_i}{15} \right)^{1.4}$</td>
<td>$30 \cdot \left( \frac{h_i}{15} \right)^{1.2}$</td>
</tr>
<tr>
<td>Solid timber, cross-laminated timber, LVL</td>
<td>$19 \cdot \left( \frac{h_i}{20} \right)^{1.4}$</td>
<td>$30 \cdot \left( \frac{h_i}{20} \right)^{1.1} \leq \frac{h_i}{\beta_0}$</td>
</tr>
<tr>
<td>Particleboard, fibreboard</td>
<td>$22 \cdot \left( \frac{h_i}{20} \right)^{1.4}$</td>
<td>$33 \cdot \left( \frac{h_i}{20} \right)^{1.1} \leq \frac{h_i}{\beta_0}$</td>
</tr>
<tr>
<td>OSB, plywood</td>
<td>$16 \cdot \left( \frac{h_i}{20} \right)^{1.4}$</td>
<td>$23 \cdot \left( \frac{h_i}{20} \right)^{1.1} \leq \frac{h_i}{\beta_0}$</td>
</tr>
<tr>
<td>Stone wool insulation with $\rho \geq 26$ kg/m$^3$</td>
<td>0</td>
<td>$0.3 \cdot h_i \left(0.75\log(\rho_i) - \rho_i/400\right)$ for $h_i &lt; 40$ mm: 0</td>
</tr>
<tr>
<td>Glass wool insulation with $\rho \geq 15$ kg/m$^3$</td>
<td>0</td>
<td>for $h_i \geq 40$ mm: $(0.0007 \cdot \rho_i + 0.046) \cdot h_i + 13 \leq 30$</td>
</tr>
</tbody>
</table>

Where $h_i$: Thickness of the layer considered [mm]
$\rho_i$: Density of the layer considered [kg/m$^3$]
$\beta$: One-dimensional charring rate ($\beta_0 = 0.65$ mm/min)

The fire behaviour of cross-laminated timber panels is influenced by the behaviour of the adhesive used for bonding the panels [5.29]. For cross-laminated timber panels with gaps less than 2 mm, where the char layer does not fall off when the char front has reached a bonded connection, the fire resistance with regard to the separation function can be calculated in the same way as for solid timber panels, considering only the total thickness of the cross-laminated timber panels. If the char layer falls off when the char front has reached a bonded connection, then an increased charring rate must be considered (see Chapter 6.4.5). In this case, for simplicity, the fire resistance with regard to the separation function can be calculated considering the single layers of the cross-laminated timber panels.
5.5.3 Position coefficients

The position coefficient considers the position of the layer within the assembly (in the direction of the heat flux), because the layers preceding and backing the layer under consideration affect its fire behaviour. The physical meaning of the position coefficient can be explained by looking at the fire behaviour of the timber assembly with three layers as shown in Figure 5.5. For simplicity, it is assumed that each layer has the same thickness and density, and the influence of joints is neglected. In this case, the basic protection value for each layer is the same ($t_{prot,0,1} = t_{prot,0,2} = t_{prot,0,3}$). The first layer is directly exposed to fire and backed by the second layer. When the fire starts, the temperature of both sides of all layers is 20 °C (see 5.5a). The contribution of the first layer to the total fire resistance is defined as $t_{prot,1}$. The position coefficient $k_{pos,1}$ of the first layer can be described as the ratio $t_{prot,1}$ to $t_{prot,0,1}$, and depends on the layer backing the first layer.

The second layer is protected by the first layer. It is conservatively assumed that, after failure of protection by the first layer (temperature of 270 °C at the interface) at time $t = t_{prot,1}$, the second layer is directly exposed to fire. The main difference in comparison with the initially unprotected first layer is that the temperature of the second layer on the fire-exposed side is 270 °C (as defined previously), and the temperature on the unexposed side is equal to or greater than 20 °C, depending on the thickness of the second layer and the material preceding and backing the layer (see Figure 5.5b). In addition, the temperature in the fire compartment is already at a high level, while no protective layer exists to reduce the effect of the temperature. For these reasons, the contribution of the second layer to the total fire resistance is lower than the contribution of the first layer, i.e. $t_{prot,2} < t_{prot,1}$. The position coefficient $k_{pos,2}$ of the second layer can be described as the ratio $t_{prot,2}$ to $t_{prot,0,2}$ and is $< 1.0$. For the same physical reasons, EN 1995-1-2 assumes that, after failure of a cladding, charring of initially unprotected surfaces takes place at an increased rate. The third layer is the last layer in the assembly. For this layer, the 140K/180K temperature criteria should therefore be applied, and an insulation value ($t_{ins,3}$) must be calculated.

![Diagram showing temperature distribution of timber assembly with three layers at different times](image)

- **a)** Start of fire: $t = 0$
- **b)** Second layer exposed to fire: $t = t_{prot,1}$
- **c)** Third layer exposed to fire: $t = t_{prot,1} + t_{prot,2}$

*Figure 5.5. Temperature distribution of timber assembly with three layers at different times.*

The influence of the layers preceding and backing the layer considered was analysed separately. The position coefficient $k_{pos,exp}$ considers the influence of the layer preceding the layer studied, while the influence of the layer backing the layer studied is considered by $k_{pos,unexp}$.

Finite-element thermal simulations showed that the influence of preheating is small. The position coefficient $k_{pos,exp}$ is mainly influenced by the time when the layer considered is exposed directly to fire and the material and thickness of the layer considered. It was therefore possible to determine the position coefficient $k_{pos,exp}$ as a function of the sum of the protection times of the layers preceding the layer considered ($\sum t_{prot,i-1}$), and the basic protection value $t_{prot,0,1}$ or basic insulation value $t_{ins,0,n}$ as
relevant for the layer considered, making calculation of the position coefficient $k_{\text{pos,exp}}$ easier for the designer (see Table 5.2).

Results of fire tests supported by finite-element thermal simulations showed that the influence of the layer backing the layer under consideration is small if the backing layer is made of gypsum or wood. Thus, for simplicity, it is assumed that $k_{\text{pos,unexp}} = 1.0$ for these cases (see Table 5.3). On the other hand, insulating batts backing the layer caused the layer to heat up more rapidly, reducing the protection time of the layer. For the different materials, this effect is allowed for by introducing the position coefficient $k_{\text{pos,unexp}}$ (see Table 5.3).

Table 5.2 and 5.3 give the position coefficient $k_{\text{pos,exp}}$ and $k_{\text{pos,unexp}}$, that were systemically calculated using finite-element numerical simulations and verified with fire tests [5.8, 5.9]. For the finite-element numerical simulations, it was assumed that a layer fails (i.e. falls off) when the temperature on the unexposed side of the layer reaches 270 °C.
### Table 5.2: Position coefficient \( k_{\text{pos,exp,i}} \) and \( k_{\text{pos,exp,n}} \)

For mineral wool insulation, only the position coefficient \( k_{\text{pos,exp,i}} \) for the protection value \( t_{\text{prot,i}} \) is given, as wall and floor assemblies with the insulation as last layer of the assembly are rarely used in buildings.

<table>
<thead>
<tr>
<th>Material</th>
<th>Position coefficient ( k_{\text{pos,exp,n}} ) for ( t_{\text{ins,n}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cladding (gypsum, timber)</strong></td>
<td>[ 1 - 0.6 \cdot \frac{\sum t_{\text{prot,n-1}}}{t_{\text{ins,0,n}}} ] for ( \sum t_{\text{prot,n-1}} \leq \frac{t_{\text{ins,0,n}}}{2} )</td>
</tr>
<tr>
<td></td>
<td>[ 0.5 \cdot \sqrt{\frac{t_{\text{ins,0,n}}}{\sum t_{\text{prot,n-1}}}} ] for ( \sum t_{\text{prot,n-1}} &gt; \frac{t_{\text{ins,0,n}}}{2} )</td>
</tr>
<tr>
<td><strong>Stone wool insulation</strong></td>
<td>[ 1 - 0.6 \cdot \frac{\sum t_{\text{prot,i-1}}}{t_{\text{prot,0,i}}} ] for ( \sum t_{\text{prot,i-1}} \leq \frac{t_{\text{prot,0,i}}}{2} )</td>
</tr>
<tr>
<td></td>
<td>[ 0.5 \cdot \sqrt{\frac{t_{\text{prot,0,i}}}{\sum t_{\text{prot,i-1}}}} ] for ( \sum t_{\text{prot,i-1}} &gt; \frac{t_{\text{prot,0,i}}}{2} )</td>
</tr>
<tr>
<td><strong>Glass wool insulation</strong></td>
<td>[ 1 - 0.8 \cdot \frac{\sum t_{\text{prot,i-1}}}{t_{\text{prot,0,i}}} ] for ( \sum t_{\text{prot,i-1}} \leq \frac{t_{\text{prot,0,i}}}{4} )</td>
</tr>
<tr>
<td>for ( h_i \geq 40 \text{ mm} )</td>
<td>[ (0.001 \cdot \rho_i + 0.27) \left( \frac{t_{\text{prot,0,i}}}{\sum t_{\text{prot,i-1}}} \right)^{(0.75 - 0.002 \rho_i)} ] for ( \sum t_{\text{prot,i-1}} &gt; \frac{t_{\text{prot,0,i}}}{4} )</td>
</tr>
</tbody>
</table>

With \( \rho_i \): Density of the layer considered [kg/m\(^3\)]
Table 5.3. Position coefficient $k_{pos,unexp}$

<table>
<thead>
<tr>
<th>Material of the layer considered</th>
<th>$k_{pos,unexp,i}$ for layers backed by cladding made of gypsum or timber</th>
<th>$k_{pos,unexp,i}$ for layers backed by insulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gypsum plasterboard,</td>
<td>1.0</td>
<td>0.5·$h_i^{0.15}$</td>
</tr>
<tr>
<td>gypsum fibre board</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Solid timber, cross-laminated</td>
<td>1.0</td>
<td>0.35·$h_i^{0.21}$</td>
</tr>
<tr>
<td>timber, LVL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Particleboard, fibreboard</td>
<td>1.0</td>
<td>0.41·$h_i^{0.18}$</td>
</tr>
<tr>
<td>OSB, plywood</td>
<td>1.0</td>
<td>0.5·$h_i^{0.15}$</td>
</tr>
<tr>
<td>Stone wool insulation</td>
<td>1.0</td>
<td>0.18·$h_i^{(0.001\rho_i+0.08)}$</td>
</tr>
<tr>
<td>Glass wool insulation</td>
<td>1.0</td>
<td>$0.01·h_i \left(-\frac{h_i^2}{30000}+\rho_i^{0.09}\right)$ - 1.3</td>
</tr>
</tbody>
</table>

With $h_i$: Thickness of the layer considered [mm]

The position coefficients $k_{pos,exp}$ given in Table 5.2 were calculated assuming that the layers fall off when the temperature of 270 °C is reached on the unexposed side of the layers. Fire tests showed that this assumption is conservative for Type F gypsum plasterboards and gypsum fibre boards [5.9]. The protection or insulation times of layers protected by Type F gypsum plasterboards or gypsum fibre board can therefore be increased using respective correction times of $\Delta t_i$ and $\Delta t_n$. Table 5.4 shows the values of these correction times that were systemically calculated using finite-element numerical simulations. It was assumed that, for floor assemblies, the Type F gypsum plasterboards or gypsum fibre boards do not fall off until the temperature on the unexposed side of the board reaches 400 °C, while the corresponding temperature for wall assemblies was assumed to be a temperature of 600 °C [5.9].
Table 5.4. Correction time $\Delta t_i$ and $\Delta t_n$ of protection and insulation times $t_{prot,i}$ and $t_{ins,n}$ of layers protected by Type F gypsum plasterboards and gypsum fibre boards.

<table>
<thead>
<tr>
<th>Material</th>
<th>Floor assemblies</th>
<th>Wall assemblies</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cladding (gypsum, timber)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Delta t_n$ for $t_{ins,n}$ [min]</td>
<td>$0,06 \cdot t_{prot,n-1} + 1,1 \cdot t_{ins,0,n} - 5,0$</td>
<td>$0,03 \cdot t_{prot,n-1} + 0,9 \cdot t_{ins,0,n} - 2,3$</td>
</tr>
<tr>
<td>for $t_{ins,0,n} &lt; 8$ min</td>
<td>for $t_{ins,0,n} &lt; 12$ min</td>
<td></td>
</tr>
<tr>
<td>$0,1 \cdot t_{prot,n-1} - 0,035 \cdot t_{ins,0,n} + 1,2$</td>
<td>$0,22 \cdot t_{prot,n-1} - 0,1 \cdot t_{ins,0,n} + 4,7$</td>
<td></td>
</tr>
<tr>
<td>for $t_{ins,0,n} \geq 8$ min</td>
<td>for $t_{ins,0,n} \geq 12$ min</td>
<td></td>
</tr>
<tr>
<td>$\Delta t_i$ for $t_{prot,i}$ [min]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0,06 \cdot t_{prot,i-1} + 1,1 \cdot t_{prot,0,i} - 5,0$</td>
<td>$0,03 \cdot t_{prot,i-1} + 0,9 \cdot t_{prot,0,i} - 2,3$</td>
<td></td>
</tr>
<tr>
<td>for $t_{prot,0,i} &lt; 8$ min</td>
<td>for $t_{prot,0,i} &lt; 12$ min</td>
<td></td>
</tr>
<tr>
<td>$0,1 \cdot t_{prot,i-1} - 0,035 \cdot t_{prot,0,i} + 1,2$</td>
<td>$0,22 \cdot t_{prot,i-1} - 0,1 \cdot t_{prot,0,i} + 4,7$</td>
<td></td>
</tr>
<tr>
<td>for $t_{prot,0,i} \geq 8$ min</td>
<td>for $t_{prot,0,i} \geq 12$ min</td>
<td></td>
</tr>
<tr>
<td><strong>Insulation (mineral wool insulation)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Delta t_i$ for $t_{prot,i}$ [min]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0,1 \cdot t_{prot,i-1} - 0,035 \cdot t_{prot,0,i}$</td>
<td>$0,1 \cdot t_{prot,i-1} + t_{prot,0,i} - 1,0$</td>
<td></td>
</tr>
<tr>
<td>for $t_{prot,0,i} &lt; 6$ min</td>
<td>for $t_{prot,0,i} &lt; 6$ min</td>
<td></td>
</tr>
<tr>
<td>$0,22 \cdot t_{prot,i-1} - 0,1 \cdot t_{prot,0,i} + 3,5$</td>
<td>for $t_{prot,0,i} \geq 6$ min</td>
<td></td>
</tr>
</tbody>
</table>
5.5.4 Joint coefficient

The joint coefficient considers the influence of joints in panels (claddings) not backed by battens or structural members or panels, and their influence on the protection and insulation time of these layers. EN 1995-1-2 does not permit the use of joints with a width greater than 2 mm. Results of the fire tests showed that the influence of joints with a width less than 2 mm, and backed by a layer, is small [5.30]. Thus, for simplicity, the design method considers the influence of joints only for the last layer of the assembly on the unexposed side and for the layer preceding a void cavity (see Table 5.5). For all other layers, it is assumed that \( k_{j,i} = 1.0 \).

Table 5.5. Joint coefficient \( k_{j,i} \)

<table>
<thead>
<tr>
<th>Material</th>
<th>Joint type</th>
<th>( k_{j,n} ) for ( t_{\text{ins,n}} )</th>
<th>( k_{j,i} ) for ( t_{\text{prot,i}} )</th>
<th>Layer backed by a void cavity</th>
<th>Layer backed by battens or panels or structural members or insulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cladding (timber)</td>
<td>( H \leq 2 \text{mm} )</td>
<td>0.3</td>
<td>0.3</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( &gt; 30 \text{mm} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \leq 2 \text{mm} ), ( \geq 15 \text{mm} )</td>
<td>0.4</td>
<td>0.4</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \geq 30 \text{mm} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \leq 2 \text{mm} ), ( \geq 15 \text{mm} )</td>
<td>0.6</td>
<td>0.6</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \geq 30 \text{mm} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>no joint</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Gypsum plasterboard,</td>
<td>( H \leq 2 \text{mm} )</td>
<td>0.8</td>
<td>0.8</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>gypsum fibre board</td>
<td>( H &gt; 2 \text{mm} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>filled joint</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>no joint</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Insulation (mineral wool</td>
<td>( H \leq 0 \text{mm} )</td>
<td>-</td>
<td>0.8</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>insulation)</td>
<td>no joint</td>
<td>-</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>
5.5.5 Void cavities

The influence of void cavities between two layers is considered in the design method by modifying the position coefficient $k_{pos,\text{exp}}$ for the layer on the side of the cavity not exposed to fire and the position coefficient $k_{pos,\text{unexp}}$ for the layer on the side of the cavity that is exposed to fire (see Table 5.6).

**Table 5.6. Modification of position coefficient $k_{pos,\text{exp}}$ and $k_{pos,\text{unexp}}$ for void cavities.**

<table>
<thead>
<tr>
<th>Material</th>
<th>Layer on the exposed side of the cavity</th>
<th>Layer on the unexposed side of the cavity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cladding (gypsum, timber)</td>
<td>$k_{pos,\text{unexp},i}$ according to Table 5.3, column 3</td>
<td>$1,6 \times k_{pos,\text{exp},i}$ according to Table 5.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$3 \times \Delta t_i$ (or $3 \times \Delta t_n$) according to Table 5.4</td>
</tr>
<tr>
<td>Insulation (mineral wool insulation)</td>
<td>$k_{pos,\text{unexp},i} = 1,0$ according to Table 5.2</td>
<td>$1,6 \times k_{pos,\text{exp},i}$ according to Table 5.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\Delta t_i$ (or $\Delta t_n$) according to Table 5.4</td>
</tr>
</tbody>
</table>

5.5.6 Detailing

The same rules as applied in EN 1995-1-2 must be satisfied in order to avoid a premature failure (i.e. fall off) of cladding and insulation. Insulating layers that are taken into account in the calculation should be tightly fitted and fixed to the timber frame such that premature failure or slumping is prevented (see Section 6.6.3.1).

Edge distances strongly influence the fire behaviour of cladding. Increasing the edge distances in comparison with those specified in the rules for normal temperature design is beneficial with respect to failure of claddings. For Type F gypsum plasterboards and gypsum fibre boards, the penetration length of fasteners into the residual cross-section should not be less than 10 mm. In addition, it must be verified that the failure time of Type F gypsum plasterboards and gypsum fibre boards with respect to pull-out failure of fasteners exceeds the protection time calculated according to Equation 5.5.

5.5.7 Other materials

Coefficients of the design method (basic values, correction time and position coefficients) for specific products can be evaluated by means of fire tests and FE-thermal simulations (see e.g. [5.31]) and found in data sheets from the producers.
5.6 Examples

The following (Sections 5.6.1 - 5.6.3) present three examples of verification of the separation function of timber structures using the improved design method.

5.6.1 Worked example 1

A timber floor structure consists of joists and claddings made of timber boards and gypsum plasterboards (see Figure 5.6). The cavities of the timber floor are completely filled with stone wool insulation, with a density of 30 kg/m³. The joints of the decking (20 mm thick solid timber panels) are single tongued and grooved with a maximum gap width of 2 mm. The required fire resistance is EI 30.

![Figure 5.6. Cross-section of the timber frame floor assembly.](Image)

The insulation time $t_{ins}$ of the timber floor should be calculated considering different heat transfer paths. However, for this example, only Heat Transfer Path 3 will be analysed (see Figure 5.6).

An insulation time must be calculated for the last layer of the floor assembly on the unexposed side (solid timber panel), while for the other layers with protective function a protection time must be calculated.

### Protection time of gypsum plasterboard Type A (Layer 1)

\[
t_{prot,0,1} = 30 \cdot \left( \frac{h_1}{15} \right)^{1.2} = 30 \cdot \left( \frac{12.5}{15} \right)^{1.2} = 24.1 \text{ min}
\]

- $k_{pos,exp,1} = 1.0$ (no layer preceding the gypsum plasterboard)
- $k_{pos,un,exp,1} = 1.0$ (OSB backing the gypsum plasterboard)
- $k_{j,1} = 1.0$ (OSB backing the gypsum plasterboard)

\[
t_{prot,1} = (t_{prot,0,1} \cdot k_{pos,exp,1} \cdot k_{pos,un,exp,1} + \Delta t_1) \cdot k_{j,1} = 24.1 \cdot 1.0 \cdot 1.0 + 0.0 = 24.1 \text{ min}
\]

### Protection time of OSB (Layer 2)

\[
t_{prot,0,2} = 23 \cdot \left( \frac{h_2}{20} \right)^{1.1} = 23 \cdot \left( \frac{12}{20} \right)^{1.1} = 13.1 \text{ min}
\]
\[
\sum_{i=1}^{n} t_{\text{prot},i} > \frac{t_{\text{prot},0.2}}{2} \rightarrow k_{\text{pos},\exp,2} = 0.5 \cdot \frac{t_{\text{prot},0.2}}{\sum_{i=1}^{n} t_{\text{prot},i}} \Rightarrow 24.1 > \frac{13.1}{2} \rightarrow k_{\text{pos},\exp,2} = 0.5 \cdot \frac{13.1}{24.1} = 0.37
\]

\[
k_{\text{pos, \exp,2}} = 0.5 \cdot h_2^{0.15} = 0.5 \cdot 12^{0.15} = 0.73 \quad \text{(insulation backing the OSB)}
\]

\[
k_{1,2} = 1.0 \quad \text{(insulation backing the OSB)}
\]

\[
t_{\text{prot},2} = (t_{\text{prot},0.2} \cdot k_{\text{pos, \exp,2}} \cdot k_{\text{pos, \exp,2}} + \Delta t_2) \cdot k_{1,2} = (3.1 \cdot 0.37 \cdot 0.73 + 0 > 1.0 = 3.5 \text{min})
\]

**Protection time of stone wool insulation (Layer 3)**

\[
t_{\text{prot},0.3} = 0.3 \cdot h_3^{0.75 \log \left( \frac{P_i}{P_0} \right)} = 0.3 \cdot 80^{0.75 \log (\frac{30}{400})} = 27.7 \text{ min}
\]

\[
\sum_{i=1}^{n_2} t_{\text{prot},i} > \frac{t_{\text{prot},0.3}}{2} \rightarrow k_{\text{pos, \exp,3}} = 0.5 \cdot \frac{t_{\text{prot},0.3}}{\sum_{i=1}^{n_2} t_{\text{prot},i}} \Rightarrow 24.1 + 3.5 = 27.6 > \frac{27.7}{2} \rightarrow k_{\text{pos, \exp,3}} = 0.5 \cdot \frac{27.7}{27.6} = 0.5
\]

\[
k_{\text{pos, \exp,3}} = 1.0 \quad \text{(solid timber panel backing the insulation)}
\]

\[
k_{1,3} = 1.0 \quad \text{(solid timber panel backing the insulation)}
\]

\[
t_{\text{prot},3} = (t_{\text{prot,0.3}} \cdot k_{\text{pos, \exp,3}} \cdot k_{\text{pos, \exp,3}} + \Delta t_3) \cdot k_{1,3} = (7.7 \cdot 0.5 \cdot 1.0 + 0 > 1.0 = 13.9 \text{ min})
\]

It should be noted that the calculated protection time of stone wool insulation can be taken into account in the calculation only if the stone wool insulation is tightly fitted and fixed to the timber frame such that premature failure (i.e. fall-off) or slumping is prevented.

**Insulation time of solid timber panel (Layer 4, last layer)**

\[
t_{\text{ins,0,4}} = 19 \cdot \left( \frac{h_4}{20} \right)^{1.4} = 19 \cdot \left( \frac{20}{20} \right)^{1.4} = 19.0 \text{ min}
\]

\[
\sum_{i=1}^{n_3} t_{\text{prot},i} > \frac{t_{\text{ins,0.4}}}{2} \rightarrow k_{\text{pos, \exp,4}} = 0.5 \cdot \frac{t_{\text{ins,0.4}}}{\sum_{i=1}^{n_3} t_{\text{prot},i}} \Rightarrow 24.1 + 3.5 + 13.9 = 41.5 > \frac{19.0}{2} \rightarrow k_{\text{pos, \exp,4}} = 0.5 \cdot \frac{19.0}{41.5} = 0.34
\]

\[
k_{1,4} = 0.4 \quad \text{(solid timber panel with single tongued and grooved joints)}
\]

\[
t_{\text{ins,4}} = (t_{\text{ins,0.4}} \cdot k_{\text{pos, \exp,4}} + \Delta t_4) \cdot k_{1,4} = (9.0 \cdot 0.34 + 0 > 9.4 = 2.6 \text{ min})
\]

**Insulation time \( t_{\text{ins}} \) (fire resistance) of the timber floor**

\[
t_{\text{ins}} = \sum_{i=1}^{n_3} t_{\text{prot},i} + t_{\text{ins},4} = t_{\text{prot,1}} + t_{\text{prot,2}} + t_{\text{prot,3}} + t_{\text{ins},4} = 24.1 + 3.5 + 13.9 + 2.6 = 44.1 \text{ min} \geq t_{\text{req}} = 30 \text{ min}
\]
5.6.2 Worked example 2

A timber wall consists of studs and claddings made of Type F gypsum plasterboards (see Figure 5.7). The cavities of the timber wall are voids. The required fire resistance is EI 60.

![Figure 5.7. Cross-section of the timber frame wall assembly.](image)

The insulation time $t_{\text{ins}}$ of the timber wall should be calculated for different heat transfer paths. However, for this example, only Heat Transfer Path 2 will be analysed (see Figure 5.7).

An insulation time must be calculated for the last layer of the wall assembly on the unexposed side, while a protection time must be calculated for the first layer with protective function.

**Protection time of gypsum plasterboard type F (Layer 1)**

$$t_{\text{prot},0,1} = 30 \cdot \left( \frac{h_1}{15} \right)^{1,2} = 30 \cdot \left( \frac{15}{15} \right)^{1,2} = 30,0 \text{ min}$$

$$k_{\text{pos,exp},1} = 1,0 \quad \text{(no layer preceding the gypsum plasterboard)}$$

$$k_{\text{pos,un,exp},1} = 0,5 \cdot h_1^{0,15} = 0,5 \cdot 15^{0,15} = 0,75 \quad \text{(void cavity backing the gypsum plasterboard)}$$

$$k_{j,1} = 1,0 \quad \text{(gypsum plasterboard without joints)}$$

$$t_{\text{prot}} = (t_{\text{prot},0,1} \cdot k_{\text{pos,exp},1} \cdot k_{\text{pos,un,exp},1} + \Delta t_1) \cdot k_{j,1} = (0,0 \cdot 1,0 \cdot 0,75 + 0 \cdot 1,0) = 22,5 \text{ min}$$

**Insulation time of gypsum plasterboard type F (Layer 2, last layer)**

$$t_{\text{ins},1,2} = 24 \cdot \left( \frac{h_2}{15} \right)^{1,4} = 24 \cdot \left( \frac{15}{15} \right)^{1,4} = 24,0 \text{ min}$$

$$\sum_{i=1}^{t_{\text{ins},1,2}} t_{\text{prot},i} > \frac{t_{\text{ins},1,2}}{2} \rightarrow k_{\text{pos,exp},2} = 1,6 \cdot 0,5 \cdot \frac{t_{\text{ins},1,2}}{\sum_{i=1}^{t_{\text{ins},1,2}} t_{\text{prot},i}} \Rightarrow \frac{22,5 > \frac{24,0}{2}}{22,5} \rightarrow k_{\text{pos,exp},2} = 1,6 \cdot 0,5 \cdot \frac{24,0}{22,5} = 0,83$$

$$t_{\text{ins},1,2} \geq 12 \text{ min} \rightarrow 3 \cdot \Delta t_2 = 3 \cdot \left[ 22 \cdot t_{\text{prot},1} - 0,1 \cdot t_{\text{ins},0,2} + 4,7 \right] = 3 \cdot 0,22 \cdot 22,5 - 0,1 \cdot 24,0 + 4,7 = 21,8 \text{ min}$$

$$k_{j,2} = 1,0 \quad \text{(gypsum plasterboard without joints)}$$

$$t_{\text{ins},2} = (t_{\text{ins},0,2} \cdot k_{\text{pos,exp},2} + 3 \cdot \Delta t_2) \cdot k_{j,2} = (4,0 \cdot 0,83 + 21,8 \cdot 1,0) = 41,7 \text{ min}$$

**Insulation time $t_{\text{ins}}$ (fire resistance) of the timber wall**

$$t_{\text{ins}} = \sum_{i=1}^{t_{\text{ins},1,2}} t_{\text{prot},i} + t_{\text{ins},2} = t_{\text{prot},1} + t_{\text{ins},2} = 22,5 + 41,7 = 64,2 \text{ min} \geq t_{\text{req}} = 60 \text{ min}$$

73
5.6.3 Worked example 3

A timber floor consists of joists and claddings made of timber boards and gypsum plasterboards (see Figure 5.8). The cavities of the timber floor are completely filled with glass wool insulation with a density of 20 kg/m³. The joints of the decking (20 mm thick solid timber panels) are single tongued and grooved with a maximum gap width of 2 mm. The required fire resistance is EI 30.

The insulation time \( t_{ins} \) of the timber floor should be calculated for different heat transfer paths. However, for this example, only Heat Transfer Path 3 will be analysed (see Figure 5.8).

An insulation time must be calculated for the last layer of the floor assembly on the unexposed side (solid timber panel), while a protection time must be calculated for the other layers with protective functions.

Protection time of Type F gypsum plasterboard (Layer 1)

\[
t_{prot,0,1} = 30 \cdot \left( \frac{h_1}{15} \right)^{1,2} = 30 \cdot \left( \frac{15}{15} \right)^{1,2} = 30 \text{ min}
\]

\( k_{pos,exp,2} = 1,0 \) (no layer preceding the gypsum plasterboard)

\( k_{pos,un,exp,1} = 1,0 \) (Type A gypsum plasterboard backing the Type F gypsum plasterboard)

\( k_{j,1} = 1,0 \) (Type A gypsum plasterboard backing the Type F gypsum plasterboard)

\[
t_{prot,1} = (t_{prot,0,1} \cdot k_{pos,exp,1} \cdot k_{pos,un,exp,1} + \Delta t_i) \cdot k_{j,1} = 0 \cdot 1,0 \cdot 1,0 + 0 \geq 1,0 = 30 \text{ min}
\]

Protection time of gypsum plasterboard Type A (Layer 2)

\[
t_{prot,0,2} = 30 \cdot \left( \frac{h_2}{15} \right)^{1,2} = 30 \cdot \left( \frac{12,5}{15} \right)^{1,2} = 24,1 \text{ min}
\]

\[
\sum_{i=1}^{t_{prot,0,2}} \frac{t_{prot,0,2}}{2} \rightarrow k_{pos,exp,2} = 0,5 \cdot \sqrt{\frac{t_{prot,0,2}}{\sum_{i=1}^{t_{prot,0,2}} t_{prot,i}}} \Rightarrow 30 > \frac{24,1}{2} \rightarrow k_{pos,exp,2} = 0,5 \cdot \sqrt{\frac{24,1}{30}} = 0,45
\]

\( k_{pos,un,exp,2} = 0,5 \cdot h_2^{0,15} = 0,5 \cdot 12,5^{0,15} = 0,73 \) (insulation backing the Type A gypsum plasterboard)

\[
t_{prot,0,2} \geq 8 \text{ min} \rightarrow \Delta t_2 = 0,1 \cdot t_{prot,1} - 0,035 \cdot t_{prot,0,2} + 1,2 = 0,1 \cdot 30 - 0,035 \cdot 24,1 + 1,2 = 3,3 \text{ min}
\]

\( k_{j,2} = 1,0 \) (insulation backing the Type A gypsum plasterboard)
Protection time of glass wool insulation (Layer 3)

\[ t_{\text{prot,3}} = (t_{\text{prot,3}} \cdot k_{\text{pos,exp,3}} \cdot k_{\text{pos,un,exp,3}} + \Delta t_3) \cdot k_{j,3} = 0.0107 \cdot 0.046 + 13 = 0.0107 \cdot 0.046 + 13 = 17.8 \min \]

\[ \sum_{i=1}^{4} t_{\text{prot,i}} > \frac{t_{\text{prot,3}}}{4} \rightarrow k_{\text{pos,exp,3}} = (0.001 \cdot \rho_3 + 0.27) \cdot \left[ \frac{t_{\text{prot,3}}}{\sum_{i=1}^{4} t_{\text{prot,i}}} \right]^{0.75 - 0.002 \rho_3} \]

\[ 30 + 11.2 > 41.2 \rightarrow k_{\text{pos,exp,3}} = (0.001 \cdot 20 + 0.27) \cdot \left[ \frac{17.8}{41.2} \right]^{0.75 - 0.002 \cdot 20} = 0.16 \]

\[ k_{\text{pos,un,exp,3}} = 1.0 \quad \text{(solid timber panel backing the insulation)} \]

\[ k_{j,3} = 1.0 \quad \text{(solid timber panel backing the insulation)} \]

Insulation time of solid timber panel (Layer 4, last layer)

\[ t_{\text{ins,0,4}} = 19 \cdot \left( \frac{h_4}{20} \right)^{1.4} = 19 \cdot \left( \frac{20}{20} \right)^{1.4} = 19 \min \]

\[ \sum_{i=1}^{4} t_{\text{ins,i}} > \frac{t_{\text{ins,0,4}}}{2} \rightarrow k_{\text{pos,exp,4}} = 0.5 \cdot \sqrt[4]{\frac{t_{\text{ins,0,4}}}{\sum_{i=1}^{4} t_{\text{ins,i}}}} \Rightarrow 30 + 11.2 + 2.8 > 44 \rightarrow k_{\text{pos,exp,4}} = 0.5 \cdot \sqrt[4]{44} = 0.33 \]

\[ k_{j,4} = 0.4 \quad \text{(solid timber panel with single tongued and grooved joints)} \]

\[ t_{\text{ins,4}} = (t_{\text{ins,0,4}} \cdot k_{\text{pos,exp,4}} + \Delta t_4) \cdot k_{j,4} = 0.0107 \cdot 0.33 + 0 \cdot 0.4 = 2.5 \min \]

Insulation time \( t_{\text{ins}} \) (fire resistance) of the timber floor

\[ t_{\text{ins}} = \sum_{i=1}^{4} t_{\text{prot,i}} + t_{\text{ins,4}} = t_{\text{prot,1}} + t_{\text{prot,2}} + t_{\text{prot,3}} + t_{\text{ins,4}} = 30 + 11.2 + 2.8 + 2.5 = 46.5 \min \geq t_{\text{req}} = 30 \min \]
5.7 References


6 Load-bearing timber structures

This chapter presents design methods for the verification of structural stability of timber structures in the event of fire, applying the classification for criterion R for fire resistance (load-bearing function). Reference is made to Eurocode 5, EN 1995-1-2, with respect to charring and strength and stiffness parameters. Alternative design models are presented, as well as design methods for new timber structures, outside the present scope of Eurocode 5.

6.1 General

This section gives guidance for structural fire design of timber structures. Reference is made to EN 1995-1-2 [6.1], including Corrigenda [6.2][6.3] and other parts of Structural Eurocodes and, where new knowledge is available, to other references. Some of the design rules given in informative annexes to Eurocodes may not be applicable in all European States, see National annexes to Eurocodes. Depending on national regulations, some of the new design methods given in the following may need agreement by the Competent Authority. Hence the content of this section should be regarded as the state of the art, and new items as potential input for future revisions of EN 1995-1-2 [6.1]. Fire scenarios other than standard fire exposure are outside the scope of this chapter.

6.2 Structural stability

The model of the structural system adopted for the design must reflect the performance of the structure in a fire situation. EN 1995-1-2 [6.1] provides the following alternatives for verification of the structural performance of the building:

- Member analysis
- Analysis of parts of the structure
- Global structural analysis.

The structural system may be different in a fire situation, e.g. where a structural member is braced at ambient temperature and the bracing fails in the fire situation, the member must be regarded as unbraced in the structural fire design, see also 6.6.3.1. Elements that are used for the stabilisation of the building, e.g. wood-based panels or gypsum plasterboard in wall or floor diaphragms, often lose their racking resistance in a fire situation unless they are protected from the fire. This effect on the global structural system must therefore be taken into account. In redundant structural systems it may be advantageous to allow for premature failure if an alternative load path is possible, e.g. a column in a fire compartment.

Unlike steel and concrete, thermal expansion of timber need not be taken into account.

6.3 Materials

6.3.1 Timber and wood-based materials

The main properties of timber and wood-based materials relevant in structural fire design are charring and the reduction of strength and stiffness properties due to elevated temperature. For simplified design, it is sufficient to consider charring, see 6.4 and simplified model strength verification, see 6.5.1. For advanced calculations (see 6.5.2), thermo-mechanical properties of softwoods (solid timber, glued-laminated timber and LVL) are given in EN 1995-1-2 Annex B. Thermal properties of OSB, plywood and wood fibreboard are given in 6.5.2.
6.3.2 Gypsum plasterboard

Some properties of gypsum plasterboards and gypsum fibreboards are given in EN 520 [6.4] and EN 15283-2 [6.5] respectively. With respect to performance in fire of these boards, these European standards give insufficient information, such as thermal properties for heat transfer calculations and mechanical properties in fire. The latter are important with respect to failure of gypsum plasterboard claddings due to thermal degradation. Failure times (i.e. fall-off times) of gypsum plasterboards are given in EN 1995-1-2 [6.1] for gypsum plasterboards Type A and H. For Type F, they must be determined on the basis of tests. Some data on failure times of gypsum plasterboards are given in [6.6], see 6.4.4.5.

For thermal properties of gypsum plasterboards, see 6.5.2.

6.3.3 Insulation materials

6.3.3.1 Mineral wool

EN 13162 [6.7] gives product characteristics of mineral wool (i.e. stone wool and glass wool) such as thermal conductivity, density and other. However, this European standard does not classify mineral wool in terms needed for structural fire design. The designer knows, for example, that stone wool performs better than glass wool when directly exposed to fire, however no relevant test method exists to quantify the difference in terms of product properties. It is also known that glass wool and stone wool perform equally when protected from direct flames, i.e. by gypsum plasterboard.

Both for stone wool and glass wool there is a weak relationship between fire performance and density. Density is therefore not sufficient to characterise the fire performance of the insulation. This can be determined by fire testing. The terminology used below is therefore traditional and recognized by the designer. Where density requirements are given, they refer to the requirements given in EN 1995-1-2 [6.1]. For better specification of mineral wool, a new classification of mineral wool is needed with respect to its performance in fire. Such classification should permit the inclusion of new types of mineral wool in accordance with EN 13162, such as one that has recently been developed and introduced on the marked. See also 9.1.1.

For thermal properties of mineral wool for thermal analyses, see 6.5.2.

6.3.3.2 Cellulose insulation

The fire performance cellulose insulation must be determined by fire testing. For their use in timber structures, of special interest is the degree of their capability of providing protection of timber members, to resist smouldering, shrinkage and surface recession. See also 9.1.1.

6.3.3.3 Other insulation materials

For more information, see 9.1.1.

6.3.4 Adhesives

EN 1995-1-2 Clause 5.2 requires that “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. For bonding of wood to wood, wood to wood-based materials or wood-based materials to wood-based materials, adhesives of phenol-formaldehyde and aminoplastic type 1 adhesive according to EN 301 [6.8] may be used. For plywood and LVL, adhesives according to EN 314 may be used”. Furthermore, since 1994 one component polyurethane (PUR) adhesives have been classified as Type 1 adhesive according to EN 301 and additional requirements. In 2008 the adhesive classification standard for PUR adhesives, EN 15425 [6.9], was published.
EN 301 [6.8] does not require any testing at elevated temperatures for phenolic and aminoplastic adhesives. According to EN 15425 [6.9], PUR adhesives are required to be tested at 70°C, being held over two weeks under constant loading of the specimens. The intention of this scenario is to cover for elevated temperatures in timber structures e.g. caused by the impact of sustained sunlight. Therefore, it has been suggested that there is a need to establish a new classification system for all types of adhesives with respect to their performance in fire [6.10] [6.15] and to develop relevant test methods.

For effect on charring in laminated members, see 6.4.5, for effect on finger joint strength, see 6.6.4.5.

### 6.4 Charring of timber and wood-based panels

#### 6.4.1 General

Timber members exposed to fire exhibit charring unless they are protected during the relevant time of fire exposure. For calculation of the resistance of timber members, the original cross-section is reduced by the charring depth.

In the following, charring of timber members is divided into:

- One-dimensional charring as a physical property for a specific species, or timber of specific density or strength class, see 6.4.2 below.
- Two-dimensional charring, including the effects of cross-sectional dimensions and other effects, see 6.4.3 below.

The charring rates are applicable for any orientation of fire-exposed surfaces and direction of fire exposure, i.e. there is no distinction between vertical or horizontal surfaces. For example, for surfaces on floors with fire exposure from above, the same charring rates apply as for surfaces with fire exposure from below. For fire exposure from above, fall-off of fire protective claddings supported by a decking are not relevant and need not be considered.

#### 6.4.2 One-dimensional charring

As a basic value, the one-dimensional charring rate $\beta_0$ is the charring rate observed for one-dimensional heat transfer under standard fire exposure of an unprotected semi-infinite timber slab without any fissures or gaps. The conditions are similar in a slab of limited thickness, see Figure 6.1, or in wide timber cross-sections remote from corners.

The one dimensional charring depth $d_{\text{char},0}$ is expressed as

$$d_{\text{char},0} = \beta_0 t$$

(6.1)

where $t$ is the time of fire exposure and $\beta_0$ is the one-dimensional charring rate perpendicular to the grain, as shown in Table 3.1 of EN 1995-1-2 [6.1]. For charring in the direction of the grain, these charring rates should be doubled. The one-dimensional charring rate given for softwoods is valid for European species (0.65 mm/min); it may also be applicable to other species, e.g. radiata pine, while the charring rates of several North American softwoods may considerably deviate, see Schaffer [6.11]. The influence of density within European strength classes for softwoods (solid timber, glued-laminated timber and LVL) is small and therefore neglected.
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The one-dimensional charring rate of wood panelling and wood-based panels is given for a panel thickness of 20 mm and a density of 450 kg/m³. For other thicknesses and densities, \( \beta_0 \) should be multiplied by factors \( k_h \) and \( k_d \), respectively, see EN 1995-1-2 Subclause 3.4.2. The charring rates for wood panelling and wood-based panels as given in EN 1995-1-2 do not take into account the fact that the panels or wood panelling burn through much more quickly at joints. In addition, the charring rates given in Table 3.1 of [6.1] are not consistent with respect to the distinction made between wood panelling and timber. For example, tongued and grooved timber decking more than 30 mm thick would imply a charring rate for timber rather than for wood panelling, 0.65 mm/min rather than 0.9 mm/min. Here it would be more appropriate to calculate the burn-through time, or basic protection value \( t_{prot,0} \) as described in Section 5 clause 5.5.2.

6.4.3 Two-dimensional charring

Near corners of, for example, rectangular cross-sections, the heat flux is typically two-dimensional, giving rise to a rounded shape of the residual cross-section near corners. At first, the radius of the arris rounding is about equal to the one-dimensional charring depth, see Figure 6.2a. Finally, due to the superposition of rounding of the two opposite arrises, the charring depth on the narrow side of a rectangular cross section increases more than it does on the wide side, see Figure 6.2b. For timber members with rectangular cross sections exposed on three or four sides, and with normal load ratios relevant for structural fire design, increased charring of the narrow side has a very limited influence on the resistance of timber members and can therefore be neglected [6.12]. Consequently, this effect needs not be taken into account.

For some specific cases, e.g. timber studs or joists protected by cavity insulation on their wide sides, increased charring of the narrow side must be taken into account, see 6.6.3.2.

For simplicity, the residual cross-section as shown in Figure 6.2 is normally replaced by a rectangular cross-section, replacing the one-dimensional charring depth and arris rounding with an equivalent notional charring depth (see Figure 6.3), calculated as

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

(6.2)

where \( \beta_n \) is the notional charring rate. EN 1995-1-2, Table 3.1 gives notional charring rates for timber members with rectangular cross-sections exposed to fire on three or four sides as

\[
\begin{align*}
\beta_n &= 0.7 \text{ mm/min} & & \text{for glued-laminated timber and LVL (softwood)} \\
\beta_n &= 0.8 \text{ mm/min} & & \text{for solid timber (softwood).}
\end{align*}
\]

The notional charring rates given in EN 1995-1-2, Table 3.1 also take into account the effects of fissures. For this reason, different values are assumed for glued-laminated timber and solid timber.

Figure 6.1. One-dimensional charring of wide cross-section.
respectively. [6.13] states that fissures and gaps with a width greater than 4 mm should be taken into account, however no proposal to quantify this effect is given.

The notional charring rate can be expressed as

$$\beta_a = k_a \beta_0$$  \hspace{1cm} (6.3)

With $\beta_0 = 0.65$ mm/min, we get $k_a = 1.08$ and $k_a = 1.23$ for glued laminated and solid timber respectively.

![Diagram](image)

Figure 6.2. Effect of arris rounding on charring on wide and narrow sides of cross-section.

![Diagram](image)

Figure 6.3. Replacing the one-dimensional charring depth and corner rounding with an equivalent (notional) charring depth.

As an alternative to the simplification of using notional charring depths, it is possible to consider a residual cross-section with linear and rounded boundaries. The calculation of cross-sectional properties will become more complicated, but normally it is not worthwhile to consider it since the difference is negligible.

For other cases, see 6.6.2, 6.6.3 and 6.6.5.

6.4.4 Effect of protection

6.4.4.1 Large cross-sections

The formation of a char layer may provide effective protection against heat flux, especially in large cross-sections. If the structure also incorporates applied protection, e.g. in the form of wood-based panels, gypsum plasterboard, stone wool batt-type insulation or other materials, the start of charring is
delayed and, where the protection remains in place after the start of charring, the rate of charring is slowed down in comparison with the charring rate for initially unprotected cross-sections. Simplified relationships of charring phases with start of charring, charring rates and failure times of protection are illustrated in Figure 6.4 to 6.6 [6.1], where $t_{ch}$ is the time of start of charring, $t_f$ is the failure time of the cladding and $t_a$ is the time when charring depth is 25 mm.

Since the charring rate immediately after failure of the fire protection – i.e. after the protection has fallen off – is much greater than for initially unprotected timber (due to the combination of high temperature and absence of, or insufficient protection by, the char layer), some of the fire protection effect is lost some time after failure. Effective protection provided by the char layer requires a char layer thickness of about 25 mm. When the char layer has grown to that depth, the charring rate falls to the rate for initially unprotected surfaces. A lasting protection effect is therefore only possible when a char layer thickness of 25 mm can be built up during the phase of increased charring rate immediately after failure of the fire protection, see Curve 2a of Figure 6.4. With rapid failure of the protection there is some delay before the start of charring, but no lasting fire protection effect: see Figure 6.5. Applied protection remaining in place after some considerable time provides the most effective fire protection, especially for protection materials with low thermal conductivities at high temperature, e.g. some gypsum plasterboards type F [6.3] exhibiting long failure times. Fall-off of claddings protecting surfaces fire-exposed from above is not a relevant scenario to be taken into consideration.

**Figure 6.4. Charring depth vs. time when charring starts at the time of failure ($t_{ch} = t_f$).**

**Key:**
1 Relationship for members unprotected throughout the time of fire exposure for charring rate $\beta_0$ (or $\beta_0$).
3a, 3b Relationship for initially protected members after failure of the fire protection:
3a After the fire protection has fallen off, charring starts at an increased rate
3b After the char depth exceeds 25 mm, the charring rate falls to the rate for initially unprotected members.
Key:
1 Relationship for members unprotected throughout the time of fire exposure for charring rate $\beta_0$ ($\alpha_0$).
3a Relationship for initially protected members with rapid failure times of the fire protection $t_f$.

Figure 6.5. Charring depth vs. time for protection with rapid failure time.

Key:
1 Relationship for members unprotected throughout the time of fire exposure for charring rate $\beta_0$ ($\alpha_0$).
2, 3a, 3b Relationship for initially protected members where charring starts before failure of the fire protection:
2 Charring starts at $t_{ch}$ at a reduced rate when the fire protection is still in place.
3a After the fire protection has fallen off, charring starts at increased rate.
3b After the char depth exceeds 25 mm, the charring rate reduces to the rate for initially unprotected members.

Figure 6.6. Charring depth vs. time when charring takes place behind the fire protection ($t_{ch} < t_f$).
6.4.4.2 Small-sized timber frame members

In small-sized timber frame members, e.g. floor joists or wall studs in assemblies with void cavities (see Figure 6.7), increased charring takes place after failure of the cladding. However, the timber member will normally collapse before reaching the consolidation phase with a char depth of 25 mm. Such conditions are described in EN 1995-1-2 [6.1] Annex D. See also 6.6.3. (below).

For small-sized timber frame members in assemblies with cavity insulation, charring mainly takes place on the narrow, fire-exposed side, see 6.6.3.2. Since there is a considerable heat flux through the insulation to the sides of the member during the stage after failure of the lining (provided that the cavity insulation remains in place), the effect of increasing arris rounding becomes dominant and no consolidation of the charring rate is possible.

Figure 6.7. Example of section of timber frame assembly.

Key:
1. Narrow side of timber beam initially protected then exposed to fire
2. Wide side of timber beam facing the cavity
3. Fire protective cladding (lining) on exposed side of timber frame floor assembly
4. Fire protective cladding (lining) on side of timber frame floor assembly not exposed to fire
Chapter 6 – Load-bearing structures – Excerpt for JRC and CEN TC250/SC5

**6.4.4.3 Determination of start of charring and failure times according to EN 1995-1-2**

EN 1995-1-2 provides limited information on determination of the start of charring behind applied protection (claddings, linings), and on failure times of protective layers. In general, the standard states that, unless expressions are given to calculate the start of charring, failure times of protective layers and charring rates of wood behind the protection where relevant, these *must be determined by testing*. For some specific materials, expressions are given for the calculation of:

- start of charring (wood-based panels and wood panelling, regular gypsum plasterboard type A, H and F [6.3], stone wool insulation)
- failure times (wood-based panels and wood panelling, regular gypsum plasterboard type A [6.3])
- charring rates of timber behind the protection (gypsum plasterboard type F [6.3], batt-type stone wool insulation).

Since gypsum plasterboard of types E, D, R and I have equal or better thermal and mechanical properties than gypsum plasterboard of type A and H, the expressions for the calculation of start of charring of gypsum plasterboard type A and H may be conservatively used for those types. Although not explicitly stated, the same applies to gypsum plasterboard type F.

EN 1995-1-2 also provides information on the start of charring where two layers of gypsum plasterboard are attached to the timber member. Where both layers are of Type A or Type H, the contribution of the inner layer is reduced by taking into account only 50% of its thickness since, after failure of the outer layer, the inner layer is already preheated and has partially calcined and is exposed to a higher temperature.
Where two layers of different quality, e.g. Type F and Type A are attached to the timber member, it is important that the better quality (Type F, in this example) is used as the outer layer, while the contribution of the inner layer (Type A or H) is reduced by taking into account only 80% of its thickness. If the outer layer is of Type A or H, and the inner layer of Type F, it should conservatively be assumed that both layers are of Type A or H.

Since the thermo-mechanical properties of gypsum plasterboard Type F are not part of the classification given in [6.3], failure times of different makes may vary considerably. No generic failure times for gypsum plasterboard are known; and so it is expected that the producer should declare failure times determined on the basis of tests, including information on spacing of joists, studs, battens etc. and edge distances and spacing of fasteners. Conservative values based on evaluation of a large number of full-scale fire tests are given in 6.4.4.5. For the time being, the European system of CE-marking does not include such information. It is important that the failure times of gypsum plasterboard should be related to thermo-mechanical degradation of the boards, i.e. issues such as position (horizontal or vertical), span and edge distances of fixings (screws, nails, staples). Pull-out failure of fasteners due to charring behind the cladding should be verified by the designer; expressions for this failure type are only given for screws: it is required that the minimum penetration length into uncharred wood is 10 mm.

Failure times of wood-based panels and wood panelling depend on the field of application. For beams and columns protected by these claddings, it is assumed that the cladding falls off at the time of start of charring \( t_{ch} \). For walls and floors, however, normally with greater distances between the fixings (the distance on centres of supporting studs, joists or resilient channels is normally 400 to 600 mm or more), it is assumed that the cladding falls off four minutes before the panel has burned through.

For gypsum plasterboard cladding, there is no corresponding distinction between beams and columns on the one hand and timber frame assemblies on the other hand. More appropriate values can be obtained from fire testing.

### 6.4.4.4 Start of charring and failure according to component additive method given in Chapter 5

Differing from the components additive method given by EN 1995-1-2 [6.1][6.2], the new components additive method presented in Section 5 is consistent with the needs of the designer to determine the start of charring and failure times of protective layers. The method given in Section 5 is therefore applicable and offers more precise solutions for a greater variety of materials. The start of charring can therefore, by modification of Expression (5.4), be calculated as:

\[
t_{ch} = \sum t_{prot,i}
\]  

(6.4)

i.e. the sum of protection times of \( i \) layers protecting the timber member, where \( t_{prot,i} \) is calculated according to Section 5, taking into account relevant position coefficients.

### 6.4.4.5 Start of charring and failure times from data base

More than 340 full-scale fire test reports with constructions including claddings made of gypsum plasterboards in accordance with EN 520 [6.4] and gypsum fibre boards in accordance with EN 15283-2 [6.52] were evaluated. Failure times and start of charring of timber studs or joists were, when recorded, collated in a data base [6.6]. The tested constructions were either timber frame assemblies, the great majority with solid timber members and some with I-joists, or in a few cases lightweight steel members. The studs or joists were placed maximum 600 mm on centres. Below pessimized expressions are given which give conservative results, see Table 6.1 and Table 6.2. The spread of data in the data base may be due to one or several of the following:
- Variation of mechanical properties (between manufacturers or batches);
- Variation of thermal properties;
- Insulated or void cavity behind cladding;
- Timber or steel studs;
- Fixing of boards with respect to edge distance and spacing;
- Fixing of boards with respect screw length;
- Distance between battens or resilient steel channels fixed to floor joists.

In some tests, premature failure of the claddings may have been caused by too short screws leading to fall-off of boards due to pull-out failure rather than thermal degradation of the boards.

The failure times of gypsum plasterboards attached to large timber members such as glued-laminated beams and columns or solid wood panels such as CLT may be considerably greater, especially when edge distances of screws are greater than possible in timber frame construction.

Since the values given in the tables are conservative, especially with regard to failure times, \( t_f \), producers may wish to determine values for their products and applications to be used by designers, see 6.4.4.3.

**Table 6.1. Start of charring behind gypsum plasterboards \( t_{ch} \) in minutes with outer board thickness \( h_p \) and total board thickness \( h_{p,tot} \) in millimetres.**

<table>
<thead>
<tr>
<th>Cladding</th>
<th>Walls</th>
<th>Floors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type A, F one layer</td>
<td>( 1.8h_p - 7 )</td>
<td>( 1.8h_p - 7 )</td>
</tr>
<tr>
<td></td>
<td>25,5</td>
<td>25,5</td>
</tr>
<tr>
<td>Type F two layers</td>
<td>( \min { 2.1h_{p,tot} - 7, 3.5h_p + 7 } )</td>
<td>( \min { 2.1h_{p,tot} - 7, 4h_p - 14 } )</td>
</tr>
<tr>
<td>Type F + Type A two layers</td>
<td>( 9 mm \leq h_p \leq 18 mm )</td>
<td>( 25 mm \leq h_{p,tot} \leq 31 mm )</td>
</tr>
<tr>
<td></td>
<td>( h_p &gt; 18 mm )</td>
<td>( 9 mm \leq h_p \leq 18 mm )</td>
</tr>
<tr>
<td>Type A two layers</td>
<td>( \min { 2.1h_{p,tot} - 7, 1.6h_p + 13 } )</td>
<td>( \min { 2.1h_{p,tot} - 7, 1.6h_p + 11 } )</td>
</tr>
<tr>
<td></td>
<td>( 18 mm \leq h_{p,tot} \leq 31 mm )</td>
<td>( 18 mm \leq h_{p,tot} \leq 31 mm )</td>
</tr>
<tr>
<td></td>
<td>( 9 mm \leq h_p \leq 18 mm )</td>
<td>( 9 mm \leq h_p \leq 18 mm )</td>
</tr>
</tbody>
</table>

Note: These values may be in conflict with EN 1995-1-2 [6.1].

For structures with claddings of two layers the important issue is the fall-off of outer layer. If charring behind the second layer starts before first layer has fallen off, the expressions for gypsum plasterboards type F and type A are similar.

If the charring starts after the outer layer has fallen off, then the expression for start of charring time depends on the failure time of outer layer.

Tables 6.1 and 6.2 show the safe design equations and their associated limitations of use. The limitations come from the available test data. No extrapolation is used. In some cases the limitations can result with bigger start of charring time compare to failure time of the same cladding. In this case the start time of charring should be taken equal to failure time of cladding, according to Figure 6.4.
Table 6.2. Failure times of gypsum plasterboards $t_f$ in minutes with board thickness $h_p$ and total board thickness $h_{p,tot}$ in millimetres.

<table>
<thead>
<tr>
<th>Cladding</th>
<th>Walls</th>
<th>Floors</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type F, one layer</strong></td>
<td>$4.5 h_p - 24$ (6.9)</td>
<td>$h_p + 10$ (6.10)</td>
</tr>
<tr>
<td></td>
<td>57</td>
<td><strong>12.5 mm ≤ $h_p$ ≤ 16 mm</strong></td>
</tr>
<tr>
<td></td>
<td>$h_p &gt; 18$ mm</td>
<td><strong>$h_p &gt; 16$ mm</strong></td>
</tr>
<tr>
<td><strong>Type F, two layers</strong></td>
<td>$4 h_{p,tot} - 40$ (6.11)</td>
<td>$2 h_{p,tot} - 3$ (6.12)</td>
</tr>
<tr>
<td></td>
<td>84</td>
<td><strong>25 mm ≤ $h_{p,tot}$ ≤ 31 mm</strong></td>
</tr>
<tr>
<td></td>
<td>$h_{p,tot} ≥ 31$ mm</td>
<td><strong>$h_{p,tot} ≥ 31$ mm</strong></td>
</tr>
<tr>
<td><strong>Type F + Type A</strong></td>
<td>81</td>
<td>$h_p ≥ 15$ mm $^b$</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>50</strong></td>
</tr>
<tr>
<td><strong>Type A, one layer</strong></td>
<td>$1.9 h_p - 7$ (6.13)</td>
<td>$1.8 h_p - 7$ (6.14)</td>
</tr>
<tr>
<td></td>
<td>21.5</td>
<td><strong>12.5 mm ≤ $h_p$ ≤ 15 mm</strong></td>
</tr>
<tr>
<td></td>
<td>$h_p &gt; 15$ mm</td>
<td><strong>$h_p &gt; 15$ mm</strong></td>
</tr>
<tr>
<td><strong>Type A, two layers</strong></td>
<td>$2.1 h_{p,tot} - 14$ (6.15)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>49</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$h_{p,tot} ≥ 30$ mm</td>
<td></td>
</tr>
<tr>
<td><strong>Type A, three layers</strong></td>
<td>55</td>
<td>$h_{p,tot} ≥ 37.5$ mm</td>
</tr>
<tr>
<td><strong>GF, one layer</strong></td>
<td>$2.4 h_p - 4$ (6.16)</td>
<td>_ $^d$</td>
</tr>
</tbody>
</table>

$^a$ Outer layer Type F, inner layer type A  
$^b$ Thickness of first layer (Type F)  
$^c$ Same as EN 1995-1-2 Clause 3.4.3.3(3)  
$^d$ No data available.

6.4.4.5.1 Worked example: Calculation of start of charring of timber stud

![Timber frame wall assembly](image)

Key:  
1. Solid timber stud  
2. 12 mm OSB  
3. 15 mm gypsum plasterboard Type F

Figure 6.9. Timber frame wall assembly.

**a. Calculation in accordance with EN 1995-1-2 [6.1]**

Material properties for gypsum plasterboard Type F:

Since no failure times of gypsum plasterboard Type F are given in EN 1995-1-2 [6.1], they must be determined by testing, see [6.1] 3.4.3.1(2). The value of failure time of gypsum plasterboard Type F attached to floors given by the producer is $t_f = 65$ minutes.

Material properties for OSB: $\rho_k = 550$ N/mm$^2$ in accordance with EN 12369-1 [6.14].

Below, following Expressions of EN 1995-1-2 [6.1] are applied: (3.4), (3.5), (3.6), (3.7) and (3.11).

Start of charring of OSB:

$\tau_{ch} = 2.8 h_p - 14 = 2.8 \times 15 - 14 = 28 \text{ min}$

Charring rate of unprotected OSB with $\beta_0 = 0.9$ mm/min from Table 3.1 in [6.1]:
\[ \beta_{0,p,t} = \beta_0 k_p k_t = \beta_0 \sqrt[20]{\frac{550}{12} \frac{20}{20}} = 0.9 \times \sqrt[20]{\frac{550}{12}} = 1.05 \text{ mm/min} \]

With
\[ k_2 = 1 - 0.018 h_p = 1 - 0.018 \times 15 = 0.73 \quad \text{for } 28 \text{ min} \leq t \leq 65 \text{ min} \]

the charring rate of OSB is
\[ \beta_{OSB} = k_2 \beta_{0,p,t} = 0.73 \times 1.05 = 0.77 \text{ mm/min} \]

Start of charring of timber stud:
\[ t_{ch} = t_{ch,OSB} + \frac{h_{OSB}}{\beta_{OSB}} = 28 + \frac{12}{0.77} = 43.7 \text{ min} \]

b. Calculation in accordance with Chapter 5

Protection time of gypsum plasterboard Type F (layer 1):
\[ t_{prot,0,1} = 30 \left( \frac{h_{2}}{15} \right)^{1.2} = 30 \times \left( \frac{15}{15} \right)^{1.2} = 30 \text{ min} \]

\[ k_{pos,exp,1} = 1.0 \quad \text{(no preceding layer)} \]

\[ k_{pos,unexp,1} = 1.0 \quad \text{(backed by OSB)} \]

\[ k_{j,1} = 1.0 \quad \text{(backed by OSB)} \]

\[ \Delta t_1 = 0 \quad \text{(no preceding layer)} \]

\[ t_{prot,1} = t_{prot,0,1} k_{pos,exp,1} k_{pos,unexp,1} + \Delta t_1 \]

\[ k_{j,1} = 30 \times 1.0 \times 1.0 + 0 \times 1.0 = 30 \text{ min} \]

Protection time of OSB (layer 2):
\[ t_{prot,0,2} = \min \left\{ \frac{23}{\left( \frac{20}{20} \right)^{1.1}} = 23 \times \left( \frac{20}{20} \right)^{1.1} = 13.1 \text{ min} \right. \]

\[ \frac{h_2}{k_{0,p,t}} = \frac{12}{1.05} = 11.4 \text{ min} \]

\[ k_{pos,exp,2} = 0.5 \times \sqrt{\frac{11.4}{30}} = 0.31 \quad \text{(since } t_{prot,1} > \frac{t_{prot,0,2}}{2} = \frac{11.4}{2} = 5.7 \text{ )} \]

\[ k_{pos,unexp,2} = 1.0 \quad \text{(backed by timber stud)} \]

\[ \Delta t_2 = 0.22 t_{prot,1} - 0.1 t_{prot,0,2} + 4.7 = 0.22 \times 30 - 0.1 \times 11.4 = 5.5 \text{ min} \]

since \[ t_{prot,1} = 30 \text{ min} > 12 \text{ min} \]

\[ t_{prot,2} = t_{prot,0,2} k_{pos,exp,2} k_{pos,unexp,2} + \Delta t_2 \]

\[ k_{j,2} = 11.4 \times 0.31 \times 1.0 + 5.5 \times 1.0 = 9.0 \text{ min} \]

Start of charring of timber stud, see Expression (6.4):
\[ t_{ch} = t_{prot,1} + t_{prot,2} = 30 + 9 = 39 \text{ min} \]
6.4.5  Effect of bonded joints

An experimental investigation on small-scale cross-laminated timber panels tested as horizontal elements (i.e. slab elements) showed the effect of local char ablation after the char front has reached the bondline [6.15]. Since the char layer provides effective protection of the residual cross-section against heat, this ablative behaviour of the char layer has the same effect as failure of a fire protective panel, see Figure 6.4 and 6.5. Therefore it is recommended to use an increased charring-rate in case the aforementioned effect of local char ablation is expected to occur. Additional investigations are ongoing in order to further study the effect of local char ablation. For unprotected cross-laminated timber (CLT) the effect observed in [6.15] is illustrated in 6.6.2.4.4. For CLT protected by a fire protective cladding such as gypsum plasterboard Type F, the char layer remains in place during the protection phase until failure of the fire protective cladding. However, local char ablation may take place after the fire protective cladding has fallen off. Since fire protective claddings delay the start of charring and reduce the charring rate, the effect of char ablation is very small. This is illustrated in 6.6.2.4.5. In the case of vertical structural members (i.e. walls) a less pronounced char ablation is expected and therefore this effect can be disregarded.

No observations are known about similar effects in glued-laminated timber. Since lamination thicknesses are considerably greater than in CLT, burn-through of the outer lamellae does normally not occur, or takes place at a very late stage. It is therefore recommended to disregard this effect.

6.5  Mechanical resistance

6.5.1  Simplified methods for strength and stiffness parameters

Since a limited zone immediately below the char-line of the residual cross-section, although unburned, is heated above normal temperature, strength and modulus of elasticity in this zone are reduced. Therefore, for structural fire design, the strength and stiffness parameters of the timber must be reduced as shown below. For unprotected cross sections the heat affected zone is about 35 to 40 mm.

In general, EN 1995-1-2 [6.1] gives the design strength of timber members as

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

(6.17)

With

\[
f_{20} = f_{k} k_{fi}
\]  

(6.18)

we get

\[
f_{d,fi} = k_{mod,fi} k_{fi} \frac{f_{k}}{\gamma_{M,fi}}
\]  

(6.19)

where

- \(f_{d,fi}\) is the strength property in the fire situation, e.g. bending strength;
- \(f_{20}\) is the 20 % fractile of the strength property;
- \(f_{k}\) is the characteristic strength property, e.g. as given for the strength classes in accordance with EN 338 [6.18];
- \(k_{mod,fi}\) is the modification factor for fire expressing the reduction of strength in the fire situation;
- \(\gamma_{M,fi}\) is the partial safety factor for timber in fire.
The recommended value of the partial factor for fire is 1; information on a specific national value may be given in the National annex. Values of $k_f$ are given in Table 2.1 of EN 1995-1-2 [6.1]; e.g. $k_f = 1.25$ for solid timber, $k_f = 1.15$ for glued laminated timber and $k_f = 1.1$ for LVL (although not mentioned in EN 1995-1-2 [6.1], for cross-laminated timber, see 6.6.2.4, $k_f$ should be taken as 1.15).

It should be noted that the modification factor $k_{mod}$, reducing the design strength taking into account the duration of load and moisture content at normal temperature, as given in EN 1995-1-1 [6.16], is not relevant in the fire situation and therefore not included in Expression (6.17).

In the same way as for Expression (6.17), the design value of a stiffness property, i.e. the modulus of elasticity and shear modulus, is given as

$$S_{d,fi} = k_{mod,fi} \frac{S_{20}}{\gamma_{M,fi}}$$  \hspace{1cm} (6.20)

With

$$S_{20} = S_{05} k_f$$  \hspace{1cm} (6.21)

we get

$$S_{d,fi} = k_{mod,fi} \frac{k_f}{\gamma_{M,fi}} \frac{S_{05}}{\gamma_{M,fi}}$$  \hspace{1cm} (6.22)

For a linear (1st order) ultimate limit state structural analysis, it is generally assumed that the internal force distribution is not influenced by the stiffness properties, unless the timber is combined with other materials in statically indeterminate structures. Thus, according to EN 1995-1-1, mean values of stiffness properties must be used for a 1st order structural analysis of a structure, where the distribution of internal forces is not affected by the stiffness distribution within the structure. For a non-linear (2nd order) ultimate limit state structural analysis, stiffness properties must be taken into account in order to assess higher order effects caused by deformation. In this case, design values of a stiffness property as defined previously must be used. It should be noted that, contrary to EN 1995-1-1 [6.16], where the design value of a stiffness property $E_d$ or $G_d$ is given by $E_d = \frac{E_{mean}}{\gamma_M}$ and $G_d = \frac{G_{mean}}{\gamma_M}$ respectively, the above Expression (6.20) relates to the 20 % value of the stiffness property1.

For consideration of strength and stiffness reduction of members exposed on three or four sides, EN 1995-1-2 [6.1] offers two methods:

- the reduced cross-section method, which is the recommended method;
- the reduced properties method.

The applicable national annex may give information about the national choice. These two methods were discussed elsewhere [6.17]. In the following only the reduced cross-section method is applied, except for specific applications, see 6.6.3.2 and 6.6.4.

According to the reduced cross-section method, the reduction of strength and stiffness parameters is taken into account by assuming normal temperature properties of timber multiplied by $k_f$. However a zero-strength layer of depth $d_0$ is subtracted from the residual cross-section (or, in other words, the charring depth is increased by $d_0$). Although, for unprotected cross sections, as much as 35 to 40 mm

---

1 This inconsistency is due to a late change of the final draft of [6.16] which was not taken into account accordingly in [6.1].
below the char layer are affected by elevated temperature, the depth of the zero-strength layer is only 7 mm for normal cross-sections exposed to fire on three or four sides [6.1]. In general the zero strength layer $d_0$ is a function of the geometry of the cross section. For specific cross-sections and partial protection, $d_0$ may be considerably larger, see 6.6.2 to 6.6.5.

For calculation of the resistance of a fire-exposed member, the original cross-section is therefore reduced by the notional charring depth and the zero-strength layer $d_0$. The effective charring depth is given as:

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

(6.23)

where $k_0$ takes into account the fact that the zero-strength layer is not fully effective during the first twenty minutes of initially unprotected members, or until the start of charring of protected members. EN 1995-1-2 [6.1] assumes linear increase from 0 to 1 during these time intervals, see [6.1] Figure 4.2. Therefore:

$$k_0 = \begin{cases} \frac{t}{20} & \text{for unprotected members} \\ \frac{t}{t_{ch}} & \text{for protected members} \end{cases}$$

(6.24)

As a consequence, at the time of start of charring of small-sized protected cross-sections, a considerable reduction of resistance may occur although the cross-section is still uncharred.

6.5.2 Advanced calculation methods

For determination of the mechanical resistance of structural timber members, an advanced calculation, e.g. using finite-element modelling of fire-exposed structural timber members, comprises several steps:

- Determination of temperatures in the timber member including the charring depth
- Determination of the resistance of cross-sections using the temperature field in the timber member and the temperature-dependent reduction of strength and stiffness at each location of the cross-section
- Determination of resistance of the structure (beam, column, frame etc.)

A general outline of the procedure is given in EN 1995-1-2 [6.1], Annex B. However, for the thermal analysis, it is the thermal properties only of wood that are given. Thermal properties for other materials often used together with timber, e.g. gypsum plasterboard, insulation materials and others are expected to be found in other sources. A problem is that the data from various sources may vary considerably. Since available commercial software for heat transfer calculations does not explicitly take into account mass transfer (water, steam, gases), its effect must be accounted for by using effective conductivity values rather than real ones [6.19], [6.20]. This also applies to the formation of cracks, e.g. in the char layer or gypsum plasterboards, causing increasing heat flux which is taken into account by increased conductivity values. For the char layer, this has not been considered in some sources, which give considerably lower conductivity values than 1995-1-2 [6.1], Annex B. Since the protection provided by boards and insulation is often important for the performance of the timber member, the software should be capable of taking into account sudden failure (fall-off) of applied protection. Examples of commercial software including this option are SAFIR [6.21] and ANSYS [6.22].
Thermal properties of gypsum plasterboard can be found in [6.23], see Table 6.3. The conductivity values are effective values.

*Table 6.3. Temperature-dependent thermal properties of gypsum plasterboard and gypsum fibreboard [6.23].*

<table>
<thead>
<tr>
<th>T °C</th>
<th>( \lambda ) W/mK</th>
<th>( c ) kJ/kgK</th>
<th>( \rho / \rho_{20} )</th>
<th>( \lambda ) W/mK</th>
<th>( c ) kJ/kgK</th>
<th>( \rho / \rho_{20} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.40</td>
<td>0.96</td>
<td>100</td>
<td>0.40</td>
<td>0.96</td>
<td>1.00</td>
</tr>
<tr>
<td>70</td>
<td>0.40</td>
<td>0.96</td>
<td>100</td>
<td>0.40</td>
<td>0.96</td>
<td>1.00</td>
</tr>
<tr>
<td>100</td>
<td>0.27</td>
<td>0.96</td>
<td>100</td>
<td>0.27</td>
<td>0.96</td>
<td>1.00</td>
</tr>
<tr>
<td>130</td>
<td>0.13</td>
<td>14.9</td>
<td>92.6</td>
<td>0.13</td>
<td>9.17</td>
<td>0.926</td>
</tr>
<tr>
<td>140</td>
<td>0.13</td>
<td>25.2</td>
<td>90.2</td>
<td>0.13</td>
<td>17.55</td>
<td>0.902</td>
</tr>
<tr>
<td>150</td>
<td>0.13</td>
<td>21.7</td>
<td>87.7</td>
<td>0.13</td>
<td>16.66</td>
<td>0.877</td>
</tr>
<tr>
<td>170</td>
<td>0.13</td>
<td>96</td>
<td>82.8</td>
<td>0.13</td>
<td>96</td>
<td>0.828</td>
</tr>
<tr>
<td>600</td>
<td>0.13</td>
<td>96</td>
<td>82.7</td>
<td>0.13</td>
<td>96</td>
<td>0.827</td>
</tr>
<tr>
<td>720</td>
<td>0.33</td>
<td>4.36</td>
<td>82.6</td>
<td>0.39</td>
<td>4.359</td>
<td>0.826</td>
</tr>
<tr>
<td>750</td>
<td>0.38</td>
<td>96</td>
<td>77.6</td>
<td>0.46</td>
<td>96</td>
<td>0.776</td>
</tr>
<tr>
<td>1000</td>
<td>0.80</td>
<td>96</td>
<td>77.6</td>
<td>1.00</td>
<td>96</td>
<td>0.776</td>
</tr>
<tr>
<td>1200</td>
<td>2.37</td>
<td>96</td>
<td>77.6</td>
<td>2.37</td>
<td>96</td>
<td>0.776</td>
</tr>
</tbody>
</table>

Thermal properties of wood-based panels may be taken as follows. For particle board and wood fibreboard, the thermal properties of wood apply. However, due to premature fall-off of charcoal, the thermal conductivity of OSB and plywood should be taken from Table 6.4, while the properties of wood apply to heat capacities and density ratios.

*Table 6.4. Temperature-dependent thermal properties of OSB, plywood and wood fibreboard [6.23].*

<table>
<thead>
<tr>
<th>T °C</th>
<th>( \lambda ) W/mK</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.12</td>
</tr>
<tr>
<td>100</td>
<td>0.30</td>
</tr>
<tr>
<td>110</td>
<td>0.23</td>
</tr>
<tr>
<td>120</td>
<td>0.15</td>
</tr>
<tr>
<td>200</td>
<td>0.18</td>
</tr>
<tr>
<td>275</td>
<td>0.14</td>
</tr>
<tr>
<td>350</td>
<td>0.09</td>
</tr>
<tr>
<td>500</td>
<td>0.23</td>
</tr>
<tr>
<td>800</td>
<td>0.74</td>
</tr>
<tr>
<td>1200</td>
<td>4.2</td>
</tr>
</tbody>
</table>

Thermal properties of insulation materials can be found in [6.23], see Table 6.5 and Table 6.6. For stone wool, effective conductivity values are determined as

\[
\lambda = \begin{cases} 
\rho_0 & \text{for } T \leq 100^\circ C \\
\rho_0 \times 11e^{-0.05\rho} + 1.9 & \text{for } T > 100^\circ C
\end{cases} \tag{6.25}
\]

where \( \rho_0 \) is given in Table 6.5.
Table 6.5. Temperature-dependent thermal properties of batt-type stone wool insulation [6.23].

<table>
<thead>
<tr>
<th>T °C</th>
<th>(\lambda(p_0)) W/mK</th>
<th>T °C</th>
<th>c kJ/kgK</th>
<th>(\rho/\rho_{20}) –</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0,036</td>
<td>20</td>
<td>0,880</td>
<td>1,00</td>
</tr>
<tr>
<td>100</td>
<td>0,047</td>
<td>100</td>
<td>1,040</td>
<td>1,00</td>
</tr>
<tr>
<td>400</td>
<td>0,090</td>
<td>200</td>
<td>1,160</td>
<td>0,980</td>
</tr>
<tr>
<td>600</td>
<td>0,150</td>
<td>400</td>
<td>1,280</td>
<td>0,977</td>
</tr>
<tr>
<td>800</td>
<td>0,230</td>
<td>600</td>
<td>1,355</td>
<td>0,973</td>
</tr>
<tr>
<td>925</td>
<td>0,300</td>
<td>800</td>
<td>1,430</td>
<td>0,970</td>
</tr>
<tr>
<td>1200</td>
<td>0,450</td>
<td>925</td>
<td>1,477</td>
<td>0,960</td>
</tr>
<tr>
<td>1200</td>
<td>1,580</td>
<td>1200</td>
<td>1,400</td>
<td>0,940</td>
</tr>
</tbody>
</table>

Table 6.6. Temperature-dependent thermal properties of batt-type glass wool insulation [6.23].

<table>
<thead>
<tr>
<th>T °C</th>
<th>(\lambda(p_0)) W/mK</th>
<th>c kJ/kgK</th>
<th>(\rho/\rho_{20}) –</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0,035</td>
<td>1,200</td>
<td>1,00</td>
</tr>
<tr>
<td>100</td>
<td>0,068</td>
<td>1,340</td>
<td>0,983</td>
</tr>
<tr>
<td>200</td>
<td>0,110</td>
<td>1,380</td>
<td>0,961</td>
</tr>
<tr>
<td>300</td>
<td>0,151</td>
<td>1,382</td>
<td>0,940</td>
</tr>
<tr>
<td>400</td>
<td>0,192</td>
<td>1,384</td>
<td>0,940</td>
</tr>
<tr>
<td>510</td>
<td>0,238</td>
<td>1,386</td>
<td>0,940</td>
</tr>
<tr>
<td>660</td>
<td>0,300</td>
<td>1,389</td>
<td>0,940</td>
</tr>
<tr>
<td>1200</td>
<td>100</td>
<td>1,400</td>
<td>0,940</td>
</tr>
</tbody>
</table>

For calculation of mechanical resistance, EN 1995-1-2 [6.1], Annex B gives temperature-dependent reduction factors \(k_\Theta\) for compressive, tensile and shear strength as well as for the modulus of elasticity parallel to the grain for timber. The reduction factor \(k_\Theta\) should not be confused with the modification factor for fire, \(k_{\text{mod,fi}}\), see 6.5.1, which gives the reduction of a strength or stiffness property with the assumption of a linear relationship between stress and strain ([6.16] Clause 3.1.2).

Note: The reduction of strength properties given in EN 1995-1-2 [6.1] may appear be in conflict with EN 1995-1-1 [6.16] which is valid for temperatures not greater than 60°C. As above, the strength properties given in EN 1995-1-2 Annex B are local properties and include the effect of moisture. Timber members exposed to long-term elevated temperatures up to 60°C would dry out causing increased strength properties. A reduction of the modification factor under ambient conditions up to 60°C, \(k_{\text{mod}}\), is therefore not necessary, see also the test results given in [6.24].

For timber members, it is sufficient to assume ideal elastic-plastic behaviour for compression and purely elastic behaviour for tension, see Figure 6.10, relationship for 20°C. The figure also shows the corresponding relationships for some other temperatures obtained by multiplication by \(k_\Theta\).
Using values for $f_c$ and $f_t$ in accordance with EN 338 [6.18] will never permit plastic flow on the compression side of a member in bending at ambient temperature, since $f_c > f_t$. Plastic flow would only occur at locations with a large rise of temperature. It would, however, be more correct to use the local strength values, see [6.25]. The compressive strength should be determined from

$$f_c = 0,114 \rho_{0,12} - 9 \quad (6.26)$$

where $\rho_{0,12}$ is the dry density in kg/m$^3$.

Replacing dry density with the density of wood with a moisture content of 12%, we get

$$f_c = 0,1 \rho - 9 \quad (6.27)$$

Expression (6.26) was derived from data given in [6.26]. Although the values were determined for Swedish-grown Scotch pine wood, it is reasonable to apply them to timbers used for strength classes up to C40 and GL36 in accordance with [6.18] and [6.27].

6.6 Structural elements

6.6.1 Beams and columns exposed on three or four sides

6.6.1.1 unprotected members

6.6.1.1.1 Charring

The notional charring rate to be used is (see 6.4.3):

$\beta_n = 0.7$ mm/min for glued laminated timber

$\beta_n = 0.8$ mm/min for solid timber.

The corresponding charring depths are calculated as (see Expression (6.2))

$$d_{ch,n} = \beta_n t \quad \text{where } t \text{ is the time of fire exposure.}$$

6.6.1.1.2 Strength and stiffness properties

The reduction of strength and stiffness properties (e.g. bending strength or modulus of elasticity) is taken into account by increasing the charring depth by a zero-strength layer of depth $d_0 \leq 7$ mm. The effective charring depth is, see Expression (6.23):

$$d_{ef} = d_{ch,n} + k_0 d_0$$

with $k_0$ according to Expression (6.24).
6.6.1.1.3 Worked example 1

A timber floor consists of joists and tongued and grooved decking. The dimensions of the joists of Strength Class C18 are $b = 45\text{ mm}$, $h = 195\text{ mm}$ and their spacing is $c = 600\text{ mm}$ on centres. The thickness (depth) of the decking is $h_1 = 28\text{ mm}$. The joints of the decking are double tongued and grooved with a maximum gap width of 2 mm. The required fire resistance is REI 15. Determine the design moment resistance of the joists for a) fire under the floor, b) fire above the floor. (For fire resistance of the decking, see worked example in 6.6.2.1.)

![Figure 6.11. Dimensions of timber floor](image)

**a. Joists fire exposed from below**

Since there is no lining fixed to the joists, they are exposed and unprotected on three sides. Determination of effective charring depth:

$$d_{ef} = \beta_a t + \frac{t}{20} d_0 = 0,8 \times 15 + \frac{15}{20} \times 7 = 12 + 5,25 = 17,25\text{ mm}$$

The section modulus is:

$$W_{ef} = \frac{(b - 2d_{ef}) \times (h - d_{ef})^2}{6} = \frac{(45 - 2 \times 17,25) \times (195 - 17,25)^2}{6} = 55291\text{ mm}^3$$

The design bending resistance is:

$$M_{d,fi} = \frac{\int_{M_{k,fi}} W_{ef}}{Y_{M,fi}} = \frac{18 \times 1,25 \times 55291 \times 10^{-3}}{1} = 1244\text{ Nm}$$

**b. Joists fire exposed from above**

The joists are protected by the decking against fire exposure. Since drying gaps at each tongued and grooved joint normally exist, the influence of increased charring should be taken into account by using notional charring rates rather than the one-dimensional values. The charring depth in the tongued and grooved decking at time $t = 15\text{ min}$ is

$$d_{char,n} = \beta_a t = 0,8 \times 15 = 12\text{ mm}$$

Alternatively, a check of protection time can be made, see Table 5.1:

$$t_{pos,0,1} = 30 \times \left(\frac{h}{20}\right)^{1,1} = 30 \times \left(\frac{28}{20}\right)^{1,1} = 43,4\text{ min} \leq \frac{h_1}{0,65} = \frac{28}{0,65} = 43,1\text{ min}$$

$$t_{pos,exp} = t_{pos,unexp} = 1$$

$$t_{ch} = t_{pos,1} = 43,1\text{ min}$$

Check of insulation criterion EI 15:

$$t_{ins,0,1} = 19 \times \left(\frac{h}{20}\right)^{1,4} = 19 \times \left(\frac{28}{20}\right)^{1,4} = 30,4\text{ min}$$

$$t_{pos,exp} = t_{pos,unexp} = 1$$

$$k_{j,1} = 0,6$$

$$t_{ins,1} = t_{ins,0,1} t_{pos,exp} t_{pos,unexp} k_{j,1} = 30,4 \times 1 \times 0,6 = 18,2\text{ min}$$

Conclusion 1: There is no risk of charring of the joists. Although there might be a slight increase of temperature in the joist, this effect can be neglected.

Conclusion 2: For bending resistance of joists, fire exposure from below the floor is decisive.

Note: For verification of the load resistance of the decking, see 6.6.2.1c.
6.6.2 Solid timber decks and walls

6.6.2.1 Tongued and grooved timber decking

6.6.2.1.1 Charring
Unlike the constructions shown in the following subclauses, a tongued and grooved timber deck is normally supported by a number of timber frame joists. EN 1995-1-2 [6.1] gives no specific information on charring rates of tongue and groove timber decking. Since timber decking is load-bearing, charring rates should be taken from Table 3.1 in [6.1] Rows a to c, but not d (wood panelling). Since drying gaps normally exist at each joint, the influence of increased charring should be taken into account by using notional charring rates, given in Table 3.1 in [6.1], rather than the one-dimensional values. Since tongued and grooved joints are “weak zones” with respect to heat transfer, the protection time \( t_{prot} \) for determination of burn-through time should be determined and be multiplied by the joint coefficient \( k_j \), see Table 5.5.

6.6.2.1.2 Strength and stiffness properties
The reduction of strength and stiffness properties (e.g. bending strength or modulus of elasticity) is taken into account by increasing the charring depth by a zero-strength layer of depth \( d_0 \). It has been shown [6.28] that \( d_0 \) for timber deck plates may be greater than 7 mm. For plate thicknesses of up to 35 mm, \( d_0 = 7 \) mm should be a reasonable value. Therefore (see Expression (6.23))

\[
d_{ef} = d_{char,n} + k_0 \cdot d_0
\]

with \( k_0 \) according to Expression (6.24).

6.6.2.1.3 Worked example
See 6.6.1.1, worked example. Check mechanical resistance of decking.

a. Decking exposed to fire from below
Decking: C18. Dimension of planks 36 mm ×150 mm.
According to EN 1991-1-1 [6.29], the recommended actions on the floor for Category A are \( q_k = 2 \) kN/m² and \( Q_k = 2 \) kN. For local action effects, only the concentrated load needs to be taken into account, acting on a surface area of 50 mm × 50 mm. It is assumed that the concentrated load is taken by one plank (no load distribution to other planks).

Combination factor for quasi-permanent action: \( \gamma_{Q,1} = 0.3 \) (see EN 1991-1-1 [6.29])

Partial factor for leading variable action: \( \gamma_{Q,1} = 1.5 \) (see EN 1990 [6.31], Table A1.2(A))

Neglecting the self-weight of the decking, the design action in the fire situation is:

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

\[
\eta_{fi} = \frac{G_k + \gamma_{z,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} = 0 + 0.3 \times 2
\]

\[
0 + 1.5 \times 2 = 2
\]

(see EN 1995-1-2 [6.1], Expression (2.8))

Design value of action at normal temperature:

\[
Q_d = \gamma_G Q_k = 1.5 \times 2 = 3 \text{ kN}
\]

Maximum bending moment:

\[
M_{d,fi} = 0.175 \times Q_d \eta_{fi} \ell = 0.175 \times 3000 \times 0.2 \times 0.6 = 63 \text{ Nm}
\]

Since there are gaps of maximum 2 mm width in the decking, a notional charring rate of 0.8 mm/min should be used.

Charring depth at 15 minutes:

\[
d_{char} = \beta_0 t_{fi} = 0.8 \times 15 = 12 \text{ mm}
\]

Effective charring depth:

\[
d_{ef} = d_{char} + d_0 k_0 = 12 + \frac{15}{20} \times 7 = 17.3 \text{ mm}
\]

Effective deck thickness:

\[
h_{ef} = h - d_{ef} = 36 - 17.25 = 18.7 \text{ mm}
\]
Neglecting the dispersion angle $\beta = 15^\circ$ of the concentrated load perpendicular to the grain (see EN 1995-2 [6.30], clause 5.2), the width of the beam under the concentrated load is 50 mm. Therefore:

Effective section modulus:

$$W_{ef} = \frac{50 \times 18,75^2}{6} = 2930 \text{ mm}^3$$

Design value of maximum bending stress:

$$\sigma_{d,\text{fi}} = \frac{M_{d,\text{fi}}}{W_{ef}} = \frac{63000}{2930} = 21,5 \text{ N/mm}^2$$

Design value of bending strength in fire:

$$f_{m,d,\text{fi}} = k_{\text{fi}} f_{m,k} = 1,25 \times 18 = 22,5 \text{ N/mm}^2$$

6.6.2.2 Glued-laminated and stress-laminated timber plates

This clause deals with timber deck or wall plates made of edgewise-arranged laminations of solid timber, held together either by adhesive bonding or by pre-stressing, see Figure 6.12.

![Figure 6.12. Examples of deck plates: a. Glued-laminated, b. Pre-stressed.](image)

The charring depth should be calculated in accordance with EN 1995-1-2 [6.1] Clause 3.4, using the one-dimensional charring rate $\beta_b$.

For verification of mechanical resistance, the reduced cross-section method given in EN 1995-1-2 [6.1] Clause 4.2.2 should be used, but with the values for the zero-strength layer $d_0$ [6.28] given in Table 6.7:

<table>
<thead>
<tr>
<th>Exposure on</th>
<th>Floors protected</th>
<th>Walls protected</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension side</td>
<td>$\frac{h}{55} + 8,5$ (6.28)</td>
<td>As unprotected.</td>
</tr>
<tr>
<td>Compression side</td>
<td>$\frac{h}{20} + 9$ (6.29)</td>
<td>As unprotected.</td>
</tr>
</tbody>
</table>

*a Values also to be used for $t > t_l$  
*b Not relevant

6.6.2.3 Nail-laminated timber plates

This clause deals with timber deck or wall plates made of edgewise-arranged laminations made of solid timber, held together by nailing, see Figure 6.13.
The charring depth should be calculated in accordance with EN 1995-1-2 [6.1], Clause 3.4, using the notional charring rate \( \beta_n \) charring rate for solid timber [6.32]. For verification of mechanical resistance, see 6.6.2.2.

### 6.6.2.4 Cross-laminated timber plates (CLT)

#### 6.6.2.4.1 General

This clause deals with cross-laminated timber panels (solid timber panels) consisting of an odd number of layers, each with a minimum thickness of 15 mm, which are bonded together. In the fire situation the joints between two adjacent CLT panels are assumed to be capable of transferring shear forces but not bending moments. Therefore they are assumed to be supported on two opposite edges, e.g. one-span floors and axially loaded walls, or supported on several supports acting as a continuous floor, see Figure 6.14.

![Figure 6.14. Example of cross-laminated deck or wall plate.](image)

The direction of the grain of outer layers and every second layer (layers No. 1, 3, 5, etc.) is orientated in the (main) load-bearing direction, while layers 2, 4, etc. are orientated in the transverse direction, see Figure 6.15a. In the design model presented here, the layers in the transverse direction are not regarded as load-bearing in the longitudinal direction; they contribute to load resistance by taking shear forces between the layers in the longitudinal direction.

In unprotected CLT a depth beyond the char line of 35 to 40 mm is affected by elevated temperature (see 6.5.1). If a fire protection is attached this depth of temperature impact is normally greater. The positive effect of an attached protection is the delay and reduction of charring, however the reduction of strength may be increased. The total effect of protection is however always positive.

Research including simulations and test on CLT in fire ([6.28] [6.33]) showed that the zero strength layer \( d_0 \) for CLT is not a constant value as given in EN 1995-1-2 [6.1] as 7 mm for large members: for some build ups the zero strength layer \( d_0 \) is larger for others lower than 7 mm. In [6.33] a simplified model for a large number of build ups of CLT is presented combining simplicity and reasonable accuracy. As said above (see 6.5.1) for the time being no simplified model exists to cover all types of structural members.

The simplified model for CLT in fire follows the same procedure for the reduced cross section method as given above in 6.5.1. Since the reduction of the residual cross section may also comprise a part of a non load bearing cross layer, in the following the depth of reduction is given as a compensating layer \( s_0 \).
In the following clauses, design expressions for fire exposure on one side are given, see Figure 6.15b.

![Diagram](image)

**Figure 6.15. Cross-section of cross-laminated timber and definitions:**

a. Cross-section at ambient temperature,
b. Residual cross-section, char layer and zero-strength layer of cross-section exposed to fire on one side.

### 6.6.2.4.2 Charring

Where a layer of a cross-laminated timber panel consists of boards bonded together along their edges, or if the edge width between two boards is not greater than 2 mm, the one-dimensional charring rate (see 6.4.2) should be applied. When the gap width is greater than 2 mm but not more than 6 mm, a notional charring rate should be applied according to Expression (6.3) where $k_n = 1.2$. When the gap width is greater than 6 mm, each board should be regarded as exposed on three sides. Charring of protected members should be calculated in accordance with 6.4.4.1. Some novel adhesives may cause premature fall-off of the char layer once the char front has reached a bondline, and so the charring rate should be increased accordingly, see 6.4.5.

### 6.6.2.4.3 Strength and stiffness properties

The simplified design model given for strength verification below follows the general outline of 6.4.2 and 6.5.1 (reduced cross-section method), i.e. the original cross-section is reduced by the effective charring depth given, in analogy to Expression (6.23), as

$$d_{ef} = d_{char,0} + k_0 s_0$$  

(6.31)  

or

$$d_{ef} = d_{char,n} + k_0 s_0$$  

(6.32)

with $k_0$ according to Expression (6.24). For the residual cross-section complete composite action may be assumed, i.e. the effect of shear deformation may be disregarded.

CLT with all laminations orientated in the (main) load-bearing direction having the same thickness, however fire tests including a CLT product with five layers, where the outer layers are considerably thicker than the middle layer, showed good agreement with the simulation results. These advanced
calculations [6.33][6.34] are in accordance with EN 1995-1-2 [6.1], clause 4.4 and annex B and are verified by tests. For advanced calculation methods, see also clause 6.5.2 above. Since simplified design methods necessarily are more conservative, it may be advantageous to carry out advanced calculations for specific CLT products.

For CLT in bending, the simplified model was fitted to the results from the simulations such that the best agreement is achieved in the range between 20 and 40 % of the bending resistance at ambient temperature. For the design of walls, the simulations were carried out for a load ratio of 30 %. The compensating layer $s_0$ was derived for determining the bending stiffness as the dominant parameter in the case of buckling.

Depending on layer thicknesses and charring depth, the compensating layer $s_0$ often includes parts of the non-loadbearing cross-layer. For other load ratios the simplified method may be more conservative. The method should not be used for fire durations of more than two hours. The fire protective effect of claddings (boards and insulation batts on the fire-exposed side) is taken into account in accordance with 6.4.4. If the residual depth of a charred layer is less than 3 mm, it should not be taken into account when deriving $h_{ef}$.

In general, $s_0$ for cross-laminated timber panels depends on:

- the number of layers;
- the depth of the cross-laminated member;
- the state of stress (tension or compression) on the fire-exposed side;
- the temperature gradient below the char layer (i.e. whether the member is protected or unprotected).

For members exposed on one side, the depth of the zero-strength layer should be taken from Table 6.8 to 6.9. For walls, zero-strength layer values, $d_0$, are given for exposed compression side only, since tensile stresses may only appear on the unexposed side of the wall (timber walls exposed to fire deflect away from the fire). For walls exposed on two sides, the design should be based on the results of fire tests.

*Table 6.8. Compensating layer $s_0$ in mm for CLT with three layers where $h$ is in mm.*

<table>
<thead>
<tr>
<th>Exposure on</th>
<th>Floors</th>
<th>Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>unprotected</td>
<td>protected$^a$</td>
</tr>
<tr>
<td>Tension side</td>
<td>$\frac{h}{30} + 3,7$ (6.33)</td>
<td>10</td>
</tr>
<tr>
<td>Compression side</td>
<td>$\frac{h}{25} + 4,5$ (6.34)</td>
<td>$\min \left{ \frac{13,5}{h_{12,5}} + 7 \right}$ (6.35)</td>
</tr>
</tbody>
</table>

$^a$ Values also to be used for $t > t_i$

$^b$ Not relevant
### Table 6.9. Compensating layer $s_0$ in mm for CLT with five layers where $h$ is in mm.

<table>
<thead>
<tr>
<th>Exposure on</th>
<th>Floors unprotected</th>
<th>Walls unprotected</th>
<th>Floors protected$^a$</th>
<th>Walls protected$^a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension side</td>
<td>$\frac{h}{100} + 10$ (6.38)</td>
<td>For $75 \text{ mm} \leq h \leq 100 \text{ mm}$: $- \frac{h}{4} + 34$ (6.39)</td>
<td>$- b$</td>
<td>$- b$</td>
</tr>
<tr>
<td>Compression side</td>
<td>$\frac{h}{20} + 11$ (6.41)</td>
<td>18</td>
<td>$\frac{h}{15} + 10.5$ (6.42)</td>
<td></td>
</tr>
</tbody>
</table>

$^a$ Values also to be used for $t > t_i$

$^b$ Not relevant

### Table 6.10. Compensating layer $s_0$ in mm for CLT with seven layers where $h$ is in mm.

<table>
<thead>
<tr>
<th>Exposure on</th>
<th>Floors unprotected</th>
<th>Walls unprotected</th>
<th>Floors protected$^a$</th>
<th>Walls protected$^a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension side</td>
<td>$\frac{h}{6} + 2.5$ (6.43)</td>
<td>For $105 \text{ mm} \leq h \leq 175 \text{ mm}$: $\frac{h}{6}$</td>
<td>$- b$</td>
<td>$- b$</td>
</tr>
<tr>
<td>Compression side</td>
<td>$\frac{h}{6} + 2.5$ (6.44)</td>
<td>For $105 \text{ mm} \leq h \leq 175 \text{ mm}$: $\frac{h}{6}$</td>
<td>For $105 \text{ mm} \leq h \leq 175 \text{ mm}$: $\frac{h}{6} + 4$ (6.45)</td>
<td>As unprotected</td>
</tr>
</tbody>
</table>

$^a$ Values also to be used for $t > t_i$

$^b$ Not relevant

### 6.6.2.4.4 Worked example 1 – Determination of charring depth and effective depth of an unprotected CLT deck plate

An unprotected cross-laminated timber plate (CLT) consisting of seven 19 mm thick layers is exposed to fire on one side. There are no gaps between the boards of each lamination. Determine the effective plate depth at 60 minutes.

**a. No char ablation**

The adhesive used for bonding between laminations is fully effective in fire. Charring depth at 60 minutes: $d_{\text{char}} = f_{\text{b, req}} = 0.65 \times 60 = 39 \text{ mm}$

Compensating layer (for exposed side in tension, see Table 6.10):

$$s_0 = \frac{h}{6} + 2.5 = \frac{133}{6} + 2.5 = 24.7 \text{ mm}$$

Effective depth:

$$h_{\text{eff}} = h - d_{\text{char}} - s_0 = 133 - 39 - 24.7 = 69.3 \text{ mm}$$

Note: Since the effective depth also comprises a part of the non-loadbearing cross-layer No. 4, the effective cross-section only consists of a three-layer CLT plate (layers 5 to 7, for layer numbers see Figure 6.15).
b. Calculation with ablation of char layer (see 6.4.5)
The failure time of Layer 1 is equal to the start of charring of Layer 2:
\[ t_{ch} = \frac{h}{\beta_0} = \frac{19}{0.65} = 29.2 \text{ min} \]
Charring depth at 60 minutes:
\[ d_{char} = h + t_{req} - t_f \beta_0 k_3 = 19 + 60 - 29.2 - 0.65 \times 2 = 59 \text{ mm} \]
Compensating layer as above:
\[ s_0 = 24.7 \text{ mm} \]
Effective depth:
\[ h_{ef} = h - d_{char} - s_0 = 133 - 59.2 - 24.7 = 49.1 \text{ mm} \]

6.6.2.4.5 Worked example 2 – Determination of charring depth and effective depth of a protected CLT deck plate

The same CLT as above in 6.6.2.4.4 is protected by a 12.5 mm thick layer of Type F gypsum plasterboard. Determine the charring depth at 60 minutes. The failure time of Type F gypsum plasterboard is given by the producer as \( t_f = 45 \) minutes.

a. No char ablation
The adhesive used for bonding between laminations is fully effective in fire.
Start of charring:
\[ t_{ch} = 2.8 h_p - 14 = 2.8 \times 12.5 - 14 = 21 \text{ min} \]
Protection factor according to EN 1995-1-2 [6.1] Expression (3.7):
\[ k_2 = 1 - 0.018 h_p = 1 - 0.018 \times 12.5 = 0.775 \]
Post-protection factor according to EN 1995-1-2 [6.1] Paragraph 3.4.3.2(4):
\[ k_3 = 2 \]
Calculation of \( t_a \) according to EN 1995-1-2 [6.1] Expression (3.9), see Figure 6.6:
\[ t_a = \frac{25 - t_f}{k_3 \beta_0} + t_f = \frac{25 - 45 - 21 \times 0.775 \times 0.65}{2 \times 0.65} + 45 = 54.9 \text{ min} \]
Charring depth at 60 minutes:
\[ d_{char} = 25 + t_{req} - t_a \beta_0 = 25 + 60 - 54.9 \times 0.65 = 28.3 \text{ mm} \]
Compensating layer as above:
\[ s_0 = \frac{h}{6} + 2.5 = \frac{133}{6} + 2.5 = 24.7 \text{ mm} \]
Effective depth:
\[ h_{ef} = h - d_{char} - s_0 = 133 - 28.3 - 24.7 = 80 \text{ mm} \]
Effective thickness of layer 3:
\[ h_{5,ef} = 80 - 4 \times 19 = 4 \times 3 \text{ mm} \]
(For layer numbering, see Figure 6.15)

b. Calculation with ablation of char layer (see 6.4.5)
Charring depth at time of failure \( t_f = 45 \) min:
\[ d_{char,f} = t_f - t_{ch} \beta_0 k_3 = 45 - 21 \times 0.65 \times 0.775 = 12.1 \text{ mm} \]
Since the char layer falls off the CLT-plate before the charring depth is equal to 25 mm, no value of \( t_a \) exists, i.e. charring continues at a rate of \( k_3 \beta_0 \).
Charring depth at 60 minutes:
\[ d_{char} = d_{char,f} + t_{req} - t_f \beta_0 k_3 = 12.1 + 60 - 45 = 31.6 \text{ mm} \]
Compensating layer as above:
\[ s_0 = 24.7 \text{ mm} \]
Effective depth:
\[ h_{ef} = h - d_{char} - s_0 = 133 - 31.6 - 24.7 = 76.7 \text{ mm} \]
Effective thickness of layer 3:
\[ h_{5,ef} = 76.7 - 4 \times 19 = 0.7 < 3 \text{ mm} \]
Since the effective depth of layer 3 is smaller than 3 mm, this layer does not contribute to bending resistance.
6.6.3 Timber frame floor and wall assemblies

6.6.3.1 General

Timber frame assemblies are normally built up of the timber frame (floor joists or wall studs) and a cladding attached to each side of the timber frame (the cladding may be a lining or, in the case of floors, the decking or a sub-floor and additional layers). The cavities may be void or partially or completely filled with insulation. Since the timber frame is sensitive to fire exposure, it must be effectively protected against fire.

In the design and optimisation of a timber frame assembly, the following rules are important with respect to maximising fire resistance:

- There exists a hierarchy of contribution to fire resistance of various layers of the assembly;
- The greatest contribution to fire resistance is obtained from the membrane (layer) on the fire-exposed side first directly exposed to the fire, both with respect to insulation and failure (fall-off) of the membrane.
- In general, it is difficult to compensate for poor fire protection performance of the first membrane by improved fire protection performance of the following layers.
- Cavity insulation improves the fire resistance of the timber frame. The best protection against fire is achieved when the insulation effectively protects the sides of the timber member facing the cavity against the fire.

For the stage before failure of the cladding (protection phase $t \leq t_f$), or, more precisely, failure of the layer of the cladding next to the insulation, both batt-type and loose-fill mineral wool (stone or glass wool) insulation perform approximately equally. However, once the cladding has fallen off and the insulation is directly exposed to the fire (post-protection phase $t \geq t_f$), glass wool insulation will undergo decomposition, gradually losing its protecting effect for the timber member by surface recession. Stone wool insulation, provided that it remains in place, will continue to protect the sides of the timber member facing the cavity. During this post-protection stage, loose-fill insulation should not be used. Batt-type mineral wool insulation should always be secured mechanically, e.g. by resilient steel channels or battens. The steel channels must be fixed with screws of sufficient length to prevent pull-out failure due to extensive charring of the joists. In the case of wall assemblies, mineral wool batts are normally fixed by oversizing the width of the batts. When the thickness of the batts is insufficient, they tend prematurely to fall off a wall assembly; and so batts less than 120 mm thick should be mechanically secured, e.g. by wires or chicken net fixed to the studs, with the wires or chicken net in turn being secured by staples of sufficient length to prevent pull-out failure due to extensive charring of the studs.

The design model presented in 6.6.3.2 is valid for cavities filled with stone wool. Where glass wool is used, the model is valid until failure (fall-off) of the cladding. Immediate failure of the assembly is a conservative assumption. From full-scale wall tests, it is known that it will take some time until the glass wool insulation has completely recessed once it has been directly exposed to the fire. A model of cavities filled with batt-type glass wool insulation has been developed, describing the surface recession of the glass wool insulation [6.35]. A new design model for the post-protection stage of assemblies with glass wool cavity insulation can be found in 6.6.3.4.

Where the wall is placed between two floors (platform frame construction), each providing a horizontal support, it may be advantageous to take into account partial restraint at the supports for the calculation of the axial load capacity of the wall with respect to buckling of the studs perpendicular to the wall. The partial restraint is due to the movement of the reaction forces towards the unexposed edge of a stud as its ends rotate, thus reducing the eccentricity of the axial loading. This positive effect was studied in [6.36] and may be taken into account assuming a buckling length of $\ell_y = 0.7 \ell_{wall}$.
where

\[ \ell_y = \text{the buckling length with respect to buckling about the y-axis (out-of-plane buckling)} \]

\[ \ell_{\text{wall}} = \text{the height of the wall including sole and head plates.} \]

For best protection against noise, walls separating two dwellings are normally built as two separate timber frame walls with studs either being staggered or placed opposite to each other. It is normally not possible or advantageous to attach sheeting panels to the unexposed side of the studs which would act as a bracing with respect to in-plane buckling of the studs. Once the fire-protective cladding attached to the exposed side of the studs has lost its bracing function, the stud is unbraced. Therefore, in order to reduce the buckling length with respect to in-plane buckling of the studs, noggins should be inserted between the studs. In order to prevent in-plane buckling of all studs into the same direction, a horizontal support is necessary, e.g. by a wall in transverse direction, or by attaching diagonal steel straps to the unexposed side of the studs (the latter is not possible where the studs are staggered).

Studs in walls exposed to fire on two sides may also need to be braced against in-plane buckling once both claddings have lost their bracing function in the fire situation, by means of noggins inserted between the studs and a horizontal support at the ends of the wall.

Since the design models given below are valid only under the assumption that the insulation remains in place, Table 6.11 gives an overview of design models and recommendations regarding the need for mechanical fixing of the insulation.

### Table 6.11. Design models for floor and wall assemblies with insulated cavities and the need for mechanical fixing of the insulation (\(t_f = \text{failure time of cladding}\)).

<table>
<thead>
<tr>
<th>Insulation</th>
<th>Batt-type(^a)</th>
<th>Loose fill</th>
<th>Batt-type(^a)</th>
<th>Loose fill</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floors</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
</tr>
<tr>
<td>(t \leq t_f)</td>
<td>Mechanical fixing</td>
<td>--</td>
<td>Mechanical fixing</td>
<td>--</td>
</tr>
<tr>
<td>(t \geq t_f)</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
</tr>
<tr>
<td>Walls</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
</tr>
<tr>
<td>(t \leq t_f)</td>
<td>Mechanical fixing for (h_{\text{ins}} &lt; 120 \text{ mm}.) 6.6.3.2</td>
<td>--</td>
<td>Mechanical fixing for (h_{\text{ins}} &lt; 120 \text{ mm}.) 6.6.3.4</td>
<td>--</td>
</tr>
<tr>
<td>(t \geq t_f)</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
<td>6.6.3.2</td>
</tr>
</tbody>
</table>

\(^a\) For floors also mat-type.

### 6.6.3.2 Assemblies with cavities completely or partially filled with stone wool insulation

#### 6.6.3.2.1 General

A design model is given in EN 1995-1-2 [6.1] Annex C (informative). The model was originally published in [6.37] and [6.38], and is based on research results reported in [6.39] and [6.40]. According to EN 1995-1-2 [6.1], the model should not be used for durations exceeding one hour, while [6.37] and [6.38] do not express such a limit, since several wall tests performed in accordance with the model have lasted for more than 90 minutes. The model was developed for cross-sections with widths from 38 to 60 mm; for greater widths the model is more conservative. The design model was developed for assemblies with cavities that are completely filled with insulation, see 6.16a. Thermal analyses have shown that the model may be used for partially filled cavities where the insulation is placed on the fire-exposed side of the cavity and the insulation thickness is not less than 100 mm. The model does not explicitly take into account air gaps between the insulation and the lining caused by the installation of resilient channels or battens. Although the influence of these air gaps theoretically is slightly positive, it is known from full-scale testing that their influence is actually negative, since once the fire penetrates through the lining the fire and hot gases can easily spread across the entire width of the assembly; see [6.41].
6.6.3.2 Charring

Although the timber members are protected by insulation batts on their wide sides, it is not only on their fire-exposed narrow side that charring takes place. Due to the heat flux through the insulation, the timber members also char on their wide sides, giving rise to extensive arris roundings, see Figure 6.16b. No consolidation of the charring rate is therefore possible, see Figure 6.8. For simplicity, the irregular residual cross-section is replaced by an equivalent rectangular cross-section, replacing the charring depth $d_{\text{char}}$ and arris roundings with the notional (or equivalent) charring depth $d_{\text{char,n}}$, see Figure 6.16c.

![Diagram](image)

Key:
1. Solid timber member (stud or joist)
2. Cladding
3. Insulation
4. Residual cross-section (real shape)
5. Char layer (real shape)
6. Equivalent residual cross-section
7. Char layer with notional charring depth

Figure 6.16. Charring of timber frame member (stud or joist): a. Section through assembly. b. Real residual cross-section and char layer. c. Notional charring depth and equivalent residual cross-section.

The notional charring rate is given as

$$\beta_n = \beta_0 \cdot k_s \cdot k_n \cdot k_p$$

(6.46)

The coefficients $k_s$, $k_n$ and $k_p$ are explained as follows:

The cross-section factor $k_s$ takes into account the effect of the width of the original cross-section. In [6.37] and [6.38] it given as

$$k_s = \begin{cases} 0.000167 b^2 - 0.029 b + 2.27 & \text{for } 38 \text{ mm} \leq b \leq 90 \text{ mm} \\ 1 & \text{for } b > 90 \text{ mm} \end{cases}$$

(6.47)

while [6.1] gives only a table with values for specific widths $b$.

Expression (6.47) assumes a linear relationship between $d_{\text{char}}$ and time, which is slightly conservative for $d_{\text{char}} < 30$ mm and non-conservative for $d_{\text{char}} > 30$ mm (for $d_{\text{char}} > 30$ mm the load resistance is normally exhausted).
New results from [6.42] show that stone wool from various producers provides somewhat less fire protection of the wide sides of the timber member than reported in [6.38][6.39][6.40]. Expression (6.47) should therefore be replaced with

$$k_s = \begin{cases} 
0.00023 b^2 - 0.045 b + 3.19 & \text{for } 38 \text{ mm} \leq b \leq 90 \text{ mm} \\
1 & \text{for } b > 90 \text{ mm}
\end{cases} \quad (6.48)$$

This expression can be used for stone wool insulation of minimum density of 26 kg/m$^2$.

The coefficient $k_n$ converts the irregular charring depth into a notional charring depth, see Figure 6.16b and c. Strictly speaking, it depends on time, cross-section dimensions and the cross-section property in question (area, section modulus or second moment of area). The value $k_n = 1.5$ given by [6.1] is a reasonable approximation for the notional charring depth that would be relevant for a relative resistance between 0.2 and 0.4. For the more conservative conditions of lower values of relative resistance and member widths of $b > 60$ mm, $k_n = 1.25$ would be more appropriate.

The coefficient $k_p$ expresses the effect of the protection by claddings, where

$$k_p = k_1 = 1.0 \quad \text{for initially unprotected members (in practice such applications exist for attic floors without a decking where the fire exposure is from above)}$$
$$k_p = k_2 \quad \text{for charring phase before the cladding has fallen off } (t_{ch} < t_f)$$
$$k_p = k_3 \quad \text{for charring phase after the cladding has fallen off } (t \geq t_f)$$

Expression (6.47) should only be used for timber members over joints of Type F gypsum plasterboards with tapered edges (this is not explicitly said in EN 1995-1-2).

The expressions for the notional charring depth, $d_{\text{char},n}$, are given as:

$$d_{\text{char},n} = \beta_0 k_s k_k k_2 t - t_{ch} \quad \text{for } t_{ch} \leq t \leq t_f \quad (6.49)$$
$$d_{\text{char},n} = \beta_0 k_s k_k [k_2 t_f - t_{ch} + k_3 t - t_f] \quad \text{for } t_f \leq t \quad (6.50)$$

### 6.6.3.2.3 Strength and stiffness properties using the reduced properties method

According to EN 1995-1-2 [6.1] and [6.37], the strength and stiffness properties of the timber members are determined using modification factors for fire $k_{\text{mod,fi}}$, see Expressions (6.19) to (6.22). The expressions for $k_{\text{mod,fi}}$ are given as:

$$k_{\text{mod,fi,step}} = a_0 + a_1 \frac{d_{\text{char},n}}{h} \quad (6.51)$$
$$k_{\text{mod,E,fi}} = b_0 + b_1 \frac{d_{\text{char},n}}{h} \quad (6.52)$$

for strength and stiffness properties respectively, with parameters $a_0$, $a_1$, $b_0$, and $b_1$ given for specific cross-sections in a number of tables. For other cross-sections, these parameters can be determined by linear interpolation.

The number of cross-sections in EN 1995-1-2 [6.1] was reduced in comparison with [6.37].
6.6.3.2.4 Strength and stiffness properties using the reduced cross-section method

In order to simplify the calculations, zero-strength layers $d_0$ were derived for application of the reduced cross-section method, see [6.43]. The effective charring depth is calculated as (see Figure 6.17)

$$d_{ef} = d_{char,n} + d_0$$  \hspace{1cm} (6.53)

The following values of $d_0$ should be used for members in bending (floor joists) with $b \geq 38 \text{ mm}$ and $h \geq 95 \text{ mm}$:

- For members with the fire-exposed side in tension

$$d_0 = 13.5 + 0.1h$$  \hspace{1cm} (6.54)

- For members with the fire-exposed side in compression

$$d_0 = 21.5 + 0.1h$$  \hspace{1cm} (6.55)

![Diagram showing cross-sections and charring depths](image)

**Key:**
1. Residual cross-section
2. Char layer
3. Notional (equivalent) cross-section
4. Notional char layer
5. Effective cross-section
6. Zero-strength layer below char layer

**Figure 6.17. Definition of charring depth, notional charring depth, effective charring depth and zero-strength layer.**

The values of $d_0$ given in Table 6.12 should be used for axially loaded members (wall studs).
Table 6.12. Values for $d_0$ for wall studs with $h$ in mm.

<table>
<thead>
<tr>
<th>Construction</th>
<th>Exposure</th>
<th>Buckling about</th>
<th>Limitations</th>
<th>$d_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall studs</td>
<td>One-sided</td>
<td>Stiff axis y-y</td>
<td>$b \geq 38$ mm, $h \geq 95$ mm</td>
<td>$13.5 + 0.1h$ (6.56)</td>
</tr>
<tr>
<td>Wall studs</td>
<td>One-sided</td>
<td>Weak axis x-x</td>
<td>$b \geq 38$ mm, $h \geq 95$ mm</td>
<td>$17 + 0.25h$ (6.57)</td>
</tr>
<tr>
<td>Wall studs</td>
<td>Two-sided</td>
<td>Stiff axis y-y</td>
<td>$b \geq 38$ mm, $h = 145$ mm</td>
<td>$25$ mm</td>
</tr>
<tr>
<td>Wall studs</td>
<td>Two-sided</td>
<td>Weak axis x-x</td>
<td>$b \geq 38$ mm, $h = 145$ mm</td>
<td>$44$ mm</td>
</tr>
</tbody>
</table>

### 6.6.3.3 Assemblies with void cavities

Timber frame floors consist of solid timber joists, a decking made of wood-based panels or timber and cladding directly fixed to the joints, resilient channels or other, see Figure 6.18. Timber frame walls consist of solid timber studs and claddings (linings) fixed directly to the studs.

![Figure 6.18. Timber frame floor with void cavities and the cladding directly fixed to the joists.](image)

**Key:**
1. Solid timber joist
2. Decking
3. Lining
4. Void cavity

At that time, the charring depth is about 10 mm.
For charring before the lining has fallen off ($t_{ch} < t_f$, Charring Phase 2 in Figure 6.6), charring on the narrow side of the cross-section must be taken into account with $k_{p,2a} = k_2$ according to EN 1995-1-2.

The charring rates during the first phase, 3a, and second phase, 3b, after failure of the cladding (see Figure 6.18) depend on the failure time of the cladding (i.e. implicitly on the gas temperature in the fire compartment). On the narrow side of the cross-section the charring rate during the second phase, 3b, after consolidation is also dependent on the width of the joists or studs, i.e. on the degree of two-dimensional heat flux giving rise to extensive influence of arris roundings. The set of expressions for the calculation of charring depths is given in the following, with definitions where $d_{char,1}$ is the charring depth on the wide sides and $d_{char,2}$ is the charring depth on the narrow side, see Figure 6.2:

![Diagram of charring depth vs. time](image)

Key:
1. Relationship for initially unprotected members
3a. Increased charring rate after failure of ceiling lining
3b. Charring after consolidation at time $t^*$

**Figure 6.19. Charring depth vs. time according to [6.45].**

For charring phase 3a with $t_a \leq t \leq t^*$

$$d_{char,1} = d_{char,2} = k_{3a} \beta_0 t$$  \hspace{1cm} (6.59)

For charring phase 3b with $t \geq t^*$

$$d_{char,1} = d_{char,2} = k_{3a} \beta_0 t^* - t_f + k_{3b} \beta_0 \left( t - t^* \right)$$  \hspace{1cm} (6.60)

where

$\beta_0 = 0.65 \text{ mm/min}$ for softwood (solid and glued laminated timber and LVL)

$$k_{3a} = \begin{cases} 1 + \frac{8}{75} t_f & \text{for } 0 \leq t_f \leq 15 \text{ min} \\ 1.9 + \frac{7}{150} t_f & \text{for } 15 \text{ min} \leq t_f \leq 60 \text{ min} \end{cases}$$  \hspace{1cm} (6.61)

$$k_{3b} = 1 + \frac{2}{225} t_f$$  \hspace{1cm} (6.62)
The corresponding notional charring depths are calculated as

\[
\begin{align*}
  d_{\text{char},1,n} &= d_{\text{char},1} \\
  d_{\text{char},2,n} &= d_{\text{char},2} k_n
\end{align*}
\]  

(6.63, 6.64)

where

\[ k_n = 1.25 \]

Note 1: [6.45] gives limits of validity for these expressions. These limits are normally not of practical importance, since the joist would fail much earlier.

Note 2: The model from [6.45] is valid for \( b \geq 60 \text{ mm} \). For \( b < 60 \text{ mm} \), only Stage 3a, but unlimited, should be used.

b. Strength and stiffness properties

Strength and stiffness properties can be determined according to the reduced cross-section method given in EN 1995-1-2 [6.1], Clause 4.2.

6.6.3.4 Cavities completely filled with glass wool insulation

a. Charring

The different charring phases are illustrated in Figure 6.20. For the time before failure of the cladding, charring takes place as described in 6.6.3.2, see Figure 6.20a. Once the cladding has fallen off at time \( t = t_f \), surface recession of the glass wool insulation takes place due to thermal decomposition, so that the wide sides of the timber member are increasingly exposed to the fire and start to char, see Figure 6.20b. When surface recession of the glass wool insulation has reached the unexposed side of the insulation at \( t = t_{f,\text{ins}} \) (Figure 6.20c), charring on the wide sides of the timber member will take place over the whole depth of the cross-section (Figure 6.20d).

Note: EN 1995-1-2 [6.1] conservatively assumes that the mechanical resistance of the timber member is exhausted at the time of failure of the cladding, \( t_f \).

![Figure 6.20. Illustration of charring phases:](image)

- a. Charring on narrow fire-exposed side before cladding has fallen off \( (t_{ch} \leq t \leq t_f) \)
- b. Charring on narrow side and wide sides during surface recession of glass wool insulation \( (t_f \leq t \leq t_{f,\text{ins}}) \)
- c. Recession of glass wool completed \( (t = t_{f,\text{ins}}) \)
- d. Charring on three sides after failure of glass wool insulation \( (t \geq t_{f,\text{ins}}) \)
In the following, it is assumed either that the cladding remains in place after the start of charring of the timber member, i.e., \( t_{ch} \leq t_f \), or that the cladding falls off at the time of start of charring, i.e. \( t_{ch} = t_f \).

For \( t_{ch} \leq t \leq t_f \) (Figure 6.20a) the design model described in 6.6.3.2 applies:

\[
d_{\text{char,n}} = \beta_0 k_n k_n k_2 \left( t_f - t_{ch} \right) \quad (6.65)
\]

For \( t_f \leq t \leq t_{f,ins} \) (Figure 6.20b) where \( t_{f,ins} \) is the time after complete recession of the insulation given by

\[
t_{f,ins} = t_f + \frac{h}{v_{\text{rec,ins}}} \quad (6.66)
\]

the notional charring depths should be calculated as:

\[
d_{\text{char,1,n}} = k_3 \beta_0 \left( t - t_f \right) \quad (6.67)
\]

\[
k_{\text{char,3}} = v_{\text{rec,ins}} (t - t_f) \quad (6.68)
\]

\[
d_{\text{char,2,n}} = \beta_0 k_2 k_n k_n (t_f - t_{ch}) + k_3 \beta_0 (t - t_f) \quad (6.69)
\]

where

\[ v_{\text{rec,ins}} \] is the surface recession rate for glass wool insulation [6.35]:

\[ v_{\text{rec,ins}} = 30 \text{ mm/min} \]

\( k_2 \) is the insulation factor of the cladding from EN 1995-1-2 [6.1] expressions (C.3) or (C.4), given as:

\[
\begin{align*}
    k_2 &= 1.05 - 0.0073 h_p & \text{for unjointed claddings} \\
    k_2 &= 0.86 - 0.0073 h_p & \text{for jointed claddings}
\end{align*} \quad (6.70)
\]

\( k_3 \) is the post-protection factor, given as, see Expression (6.61),

\[
k_{3a} = \begin{cases} 
1 + \frac{8}{75} t_f & \text{for } 0 \leq t_f \leq 15 \text{ min} \\
1.9 + \frac{7}{150} t_f & \text{for } 15 \text{ min} \leq t_f \leq 60 \text{ min}
\end{cases} \quad (6.72)
\]

\[
k_{3b} = 1 + \frac{2}{225} t_f & \text{for } 0 \leq t_f \leq 60 \text{ min} \quad (6.73)
\]

Note: Since this stage is relatively short, intermediate values of cross-section properties can also be determined by linear interpolation of cross-section properties calculated at time \( t = t_f \) and \( t = t_{f,ins} \).

For \( t \geq t_{f,ins} \) (Figure 6.20d) the notional charring depth \( d_{\text{char,1,n}} \) should be calculated according to Expression (6.67) and the charring depths at the unexposed edge of the wide sides of the timber member should be calculated as:

\[
d_{\text{char,1,unexp,n}} = k_3 \beta_0 (t - t_{f,ins}) \quad (6.74)
\]

b. Strength and stiffness properties

For \( t_{ch} \leq t \leq t_f \), the design model described in 6.6.3.2 applies.

For \( t > t_f \), the reduced cross-section method given by EN 1995-1-2 [6.1], Clause 4.2.2 should be used, i.e. the charring depth should be increased by \( d_0 = 7 \text{ mm} \).

Note: For assemblies with \( t_{ch} \leq t_f \) at time \( t = t_f \) and shortly after, this is not consistent with 6.6.3.2.
6.6.3.5 Cavities filled with insulation other than glass or stone wool

Insulation other than glass or stone wool may exhibit better or poorer protection of the timber member. In 6.6.3.2, insulation performance may affect the following parameters:

- $k_s$ see Expression (6.47) or (6.48)
- $k_n$ normally taken as $k_n = 1,5$
- $k_{mod.fm,fi}$ see Expression (6.51)
- $k_{mod,E,fi}$ see Expression (6.52)
- $d_0$ see Expressions (6.54), (6.55) and Table 6.12.

Where insulation materials behave in a similar manner to glass wool, by thermal decomposition when a specific critical temperature is exceeded, the surface recession rate $v_{rec,ins}$ may be different from that for glass wool, see 6.6.3.4. These parameters may be declared by the producer of the insulation, see Section 9.1.2 for guidance to determine these parameters.

6.6.3.6 Worked examples

Worked example 1

A timber floor as shown in Figure 6.21 is exposed to fire from below. The dimensions of the joists are 45 mm × 220 mm and the strength class is C 24. The cavities are filled with 100 mm thick stone wool batts. The cladding consisting of an outer layer of 12.5 mm thick Type A gypsum plasterboard (Layer 1), and an inner layer of 12 mm thick OSB (Layer 2) is fixed to resilient channels of depth 25 mm. The resilient channels are fixed to the joists using screws of length $t_f = 29$ mm. Determine the moment resistance of the joists for a fire resistance of $R_{30}$ and check the screw length with respect to risk of withdrawal failure.

**Figure 6.21. Cross-section of floor assembly.**

**Determination of charring depth:**

Charring of the joists starts at the failure time of the cladding $t_{ch} = t_f + t_{prot,1} + t_{prot,2} = 24,1 + 3,5 = 27,6$ min

where the protection times of Layers 1 and 2 are determined as in 5.5.1.

The notional charring rate to be used is (EN 1995-1-2 [6.1], Expression (C.2))

$$\beta_n = \beta_0 k_s k_n k_3 = 0,00167 \times 1,3 \times 1,5 \times 2,0 = 2,54 \text{ mm/min}$$

with

- $k_s = 0,00167 b^2 - 0,029 b + 2,27 = 1,3$
- $k_n = 1,5$
- $k_3 = 0,036 t_f + 1 = 0,036 \times 27,6 + 1 = 2,0$
- $\beta_0 = 0,65 \text{ mm/min}$

The notional charring depth at $t_f = 30$ min is

$$d_{char,n} = \beta_n \times t_f = 2,54 \times 30 \approx 27,6 \approx 6,1 \text{ mm}$$
Determination of design moment resistance in fire in accordance with EN 1995-1-2 [6.1], Annex C:
The section modulus of the notional residual cross-section is

\[ W_n = \frac{b \ h - d_{\text{char},n}^2}{6} = \frac{45 \times 220 - 6.1^2}{6} = 343149 \text{ mm}^3 \]

The design bending strength in fire is

\[ f_{\text{m,d,fi}} = k_{\text{mod,n}} \frac{f_{\text{m,k}}}{\gamma_{\text{M,fi}}} \], see Expression (6.19)

With \( a_0 = 0.76 \) and \( a_1 = 0.51 \) (see EN 1995-1-2 [6.1], table C2)

\[ k_{\text{mod,fi}} = a_0 - a_1 \frac{d_{\text{char},n}}{h} = 0.76 - 0.51 \times \frac{6.1}{220} = 0.746 \] (see Expression (6.51))

\[ f_{\text{m,k}} = 24 \text{ N/mm}^2 \]

\[ k_{\text{fi}} = 1.25 \] (see EN 1995-1-2 [6.1], Clause 2.3)

\[ \gamma_{\text{M,fi}} = 1.0 \]

\[ f_{\text{m,d,fi}} = 0.746 \times 1.25 \times \frac{24}{1.0} = 22.4 \text{ N/mm}^2 \]

The design moment resistance in fire is

\[ M_{\text{d,fi}} = W_n \frac{k_{\text{fi}}}{\gamma_{\text{M,fi}}} = 343.149 \times 22.4 = 7.679.675 \text{ Nmm} = 7.68 \text{ kNm} \]

Determination of design moment resistance in fire by the reduced cross section method, see 6.6.3.2.4:
Calculate the effective charring depth using Expressions (6.53) and (6.54)

\[ d_{\text{ch}} = 13.5 + 0.1d = 13.5 + 0.1 \times 220 = 35.5 \text{ mm} \]

\[ d_{\text{ef}} = d_{\text{char},n} + d_0 = 6.1 + 35.5 = 41.6 \text{ mm} \]

The section modulus of the effective residual cross-section is

\[ W_{\text{ef}} = \frac{b \ h - d_{\text{ef}}^2}{6} = \frac{45 \times 220 - 41.6^2}{6} = 238.699 \text{ mm}^3 \]

The design moment resistance in fire is

\[ M_{\text{d,fi}} = W_{\text{ef}} \frac{k_{\text{fi}}}{\gamma_{\text{M,fi}}} = 238.699 \times 24 \times \frac{1.25}{1.0} = 7.160.976 \text{ Nmm} = 7.16 \text{ kNm} \]

Check of screw length:
Calculate the failure time of the screws, \( t_{\text{sf}} \), in accordance with EN 1995-1-2 [6.1], Expression (C.12) as

\[ t_{\text{sf}} = t_{\ell} + \frac{\ell_{\ell} - \ell_{a,\min} - k_2 k_n \beta_0 (t_{\ell} - t_{a,\ell}) - t_{\ell}}{k_2 k_n k_{\beta,0}} = 27.6 + \frac{29 - 10 - 0 - 0.6}{1.3 \times 2.0 \times 1.5 \times 0.65} = 34.9 \text{ min} > t_{\text{req}} = 30 \text{ min} \]

Conclusion: There is no risk of premature failure of the resilient channels.

Worked example 2
A wall assembly with a total height of 2800 mm, including top and sole plates, is shown in Figure 6.22. The dimensions of the studs are 45 mm × 145 mm. The strength class of the studs is C 24. The cavities are completely filled with 145 mm batt-type stone wool. The cladding on the fire-exposed side consists of one layer of 15 mm thick Type F gypsum plasterboard with a declared failure time from the producer. The cladding on the unexposed side is made of one layer of Type H gypsum plasterboard. The length of the screws on the fire-exposed side is \( \ell = 52 \text{ mm} \). Determine the design value of the axial load resistance of a stud without a cladding joint above it for a fire resistance of R 60.

![Figure 6.22. Cross-section of wall assembly.](image-url)
Determination of charring depth:
The time of start of charring is given by EN 1995-1-2 [6.1] Expression (3.11), as
\[ t_a = 2.8h_p - 14 = 2.8 \times 15 - 14 = 28 \text{ min} \]
The insulation factor, \( k_2 \), is given by EN 1995-1-2 [6.1], Expression (C.3), as
\[ k_2 = 1.05 - 0.0073h_p = 1.05 - 0.0073 \times 15 = 0.94 \]
The failure time of the cladding with respect to thermal degradation is declared by the producer as \( t_f = 65 \text{ min} \).
The failure time of the cladding with respect to pull-out failure of screws is (see EN 1995-1-2 [6.1], Expression (C.9)), calculated as
\[ t_a = t_{ch} + \frac{t_f - t_{f,\min}}{h_p} = 28 + \frac{52 - 10 - 15}{1.3 \times 0.94 \times 1.5 \times 1.0 \times 0.65} = 50.7 \text{ min} < 65 \text{ min} \]
The post-protection factor, \( k_3 \), is given by EN 1995-1-2 [6.1], Expression (C.5), as
\[ k_3 = 0.036t_f + 1 = 0.036 \times 50.7 + 1 = 2.825 \]
The notional charring depth at 60 minutes is:
\[ d_{char,n} = k_s k_2 k_3 h_p (t_f - t_{ch} + k_s k_2 h_p) \\
= 1.3 \times 0.94 \times 0.65 \times 50.7 - 28 + 1.3 \times 2.825 \times 0.65 \times 60 - 50.7 = 40.2 \text{ mm} \]
The dimensions of the residual cross-section of the studs are
\[ b_t = 45 \text{ mm} \]
\[ h_t = 145 - 40.2 = 104.8 \text{ mm} \]

Determination of design value of axial resistance in fire in accordance with EN 1995-1-2 [6.1], Annex C:

With
\[ a_0 = 0.55 \]
\[ a_1 = 0.40 \]
\[ b_0 = 0.60 \]
\[ b_1 = 0.84 \]

Calculate the modification factors for fire (for compressive strength and modulus of elasticity respectively): see EN 1995-1-2 [6.1], Expressions (C.13) and (C.14):
\[ k_{mod,fc,fi} = a_0 - a_1 \frac{d_{char,n}}{h} = 0.55 - 0.40 \times \frac{40.2}{145} = 0.44 \]
\[ k_{mod,E,fi} = b_0 - b_1 \frac{d_{char,n}}{h} = 0.60 - 0.84 \times \frac{40.2}{145} = 0.36 \]

The design values of compressive strength and modulus of elasticity are, see Expressions (6.19) and (6.22):
\[ f_{c,\text{rel}} = k_{mod,fc,fi} f_{c,k} = 0.44 \times 21 = 11.55 \text{ N/mm}^2 \]
\[ E_{\text{rel}} = k_{mod,E,fi} E_{M,k} = 0.36 \times 7400 = 3300 \text{ N/mm}^2 \]

With EN 1995-1-1 [6.16], Expressions (6.21) to (6.29), we get
\[ \lambda_y = \frac{f_y \sqrt{\frac{12}{i}}}{h_t} = \frac{2800 \sqrt{12}}{104.8} = 92.55 \]
\[ \lambda_{\text{rel,y}} = \frac{\lambda_y}{\pi \sqrt{k_{mod,fc,fi} f_{c,0,k} E_{0.05}}} = \frac{92.55}{\pi \sqrt{0.44 \times 21 \times 0.36}} = 1.73 \]
\[ \beta_c = 0.2 \]
\[ k_y = 0.5 + \beta_c \lambda_{\text{rel,y}} - 0.3 + \lambda_{\text{rel,y}}^2 = 0.5 \times 1 + 0.2 \times 1.73 - 0.3 + 1.73^2 = 2.139 \]
\[ k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{\text{rel,y}}^2}} = \frac{1}{2.139 + \sqrt{2.139^2 - 1.73^2}} = 0.294 \]
The design value of the axial load resistance of one stud is
\[ N_{d,fi} = A_y k_{c,y} f_{c,\text{rel}} = 45 \times 104.8 \times 0.294 \times 11.55 = 16.014 \text{ N} = 16.0 \text{ kN} \]
Determination of design value of axial resistance in fire by the reduced cross-section method, see 6.6.3.2.4:

Calculate the effective charring depth as (Expressions (6.53) and (6.56)):

\[ d_0 = 13.5 + 0.1h = 13.5 + 0.1 \times 145 = 28.0 \text{ mm} \]
\[ d_{\text{ef}} = d_{\text{char}} + d_0 = 40.2 + 28.0 = 68.2 \text{ mm} \]

The effective depth of the cross-section is

\[ h_{\text{ef}} = 145 - 68.2 = 76.8 \text{ mm} \]

With \( k_{\text{mod,fc,fi}} = 1.0 \), the design value of compressive strength in fire is calculated as

\[ f_{c,d,\text{ef}} = k_{\text{mod,fc,fi}} k_0 \frac{f_{c,k}}{\gamma_{\text{M6}}} = 1.0 \times 1.25 \times \frac{21}{1,0} = 26,25 \text{ N/mm}^2 \]

With EN 1995-1-1 [6.16], Expressions (6.21) to (6.29), we get

\[ \lambda_y = \frac{\ell_y}{h_{\text{ef}}} = \frac{2800 \sqrt{12}}{76,8} = 126.3 \]
\[ \lambda_{\text{rel,Y}} = \frac{\lambda_y}{\pi} \frac{k_{\text{mod,fc,fi}} f_{c,0,k}}{E_{\text{0,05}}} = \frac{126.3}{\pi} \frac{1,0 \times 21}{1,0 \times 7400} = 2.14 \]
\[ \beta_k = 0.2 \]
\[ k_y = 0.5 + \beta_k \lambda_{\text{rel,Y}} - 0.3 + \lambda_{\text{rel,Y}}^2 = 0.5 \times 1 + 0.2 \cdot 2.14 - 0.3 + 2.14^2 = 2.977 \]
\[ k_{c,Y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{\text{rel,Y}}^2}} = \frac{1}{2.977 + \sqrt{2.977^2 - 2.14^2}} = 0.198 \]

The design value of the axial load resistance of one stud is

\[ N_{d,\text{fI}} = A_k f_{c,d,\text{ef}} = 45 \times 76.8 \times 0.198 \times 26.25 = 17.963 \text{ N} = 18.0 \text{ kN} \]

6.6.4 Light weight timber frame floors with I-joists

6.6.4.1 General

The following design model is taken from [6.46]. For general aspects, see 6.6.3.1. For partially filled cavities, see 6.6.3.2.1.

6.6.4.2 Cross-section

The dimensions of an I-joist are shown in Figure 6.23. At ambient temperature, the characteristic moment resistance is calculated as:

\[ M_k = f_{m,k} W_{\text{ef}} k_h \] (6.75)

With

\[ W_{\text{ef}} = \frac{2I_{\text{ef}}}{h} \] (6.76)
\[ I_{\text{ef}} = I_f + \frac{E_w}{E_f} I_w \] (6.77)

where:

- \( f_{m,k} \) is the characteristic bending strength of the I-joist, see below;
- \( E_f \) is the mean value of the modulus of elasticity of the flange;
- \( E_w \) is the mean value of the modulus of elasticity of the web;
- \( I_f \) is the contribution of the flanges to the second moment of area;
- \( I_w \) is the contribution of the web to the second moment of area;
$k_h$ is a depth effect, where applicable; see below.

![Diagram of I-joist cross-sections](image)

**Figure 6.23. Cross-section of I-joist, a: at ambient conditions, b: in the fire situation.**

When the bending resistance of the I-joist is calculated according to EN 1995-1-1[6.16], $k_h = 1$ and the characteristic bending strength of the I-joist, $f_{m,k}$, is taken as the bending strength of the flanges. Since the mean flange design stresses $\sigma_{f,c,d}$ and $\sigma_{f,t,d}$ should not be greater than the design compressive and tensile strength of the flanges, $f_{m,k}$ can be replaced by

$$f_{m,k,ef} = \min \left\{ \frac{f_{m,k}}{h-h_f}, \frac{f_{c,k} h}{h-h_f}, \frac{f_{t,k} h}{h-h_f} \right\}$$

(6.78)

where the bending resistance of the I-joist was derived from testing, and the characteristic bending strength of the I-joist, $f_{m,k}$, and $k_h$ are declared by the producer of the I-joist.

In the fire situation, for I-beams in floor assemblies with cavities that are completely insulated, the cross-section shown in Figure 6.23b should be used to calculate the mechanical resistance for the required period of fire exposure $t$. This also applies for partially insulated cavities with a minimum insulation thickness of 100 mm.

For failure during Charring Phase 3, – that is, that the cladding has fallen off at time $t_f$ – the notional charring depth, $d_{\text{char,n}}$, should be taken as:

$$d_{\text{char,n}} = \beta_h (t - t_{f,ef})$$

(6.79)

where:

$$\beta_h = \beta_0 \, k_{b,ch} \, k_3 \, k_n$$

(6.80)

$$k_{b,ch} = \frac{27.4}{b} + 1$$

(6.81)

$$k_3 = 0.0157 \, t_f + 1$$

(6.82)
\[ t_{f,ef} = 0.9 t_f \]  
(6.83)

\[ k_n = 1.4 \]

- \( \beta_0 \) is the one-dimensional charring rate given in EN 1995-1-2 [6.1], i.e. \( \beta_0 = 0.65 \) mm/min for solid softwood and LVL;
- \( t_f \) is the failure time of the cladding, in mm. It may be given by EN 1995-1-2 [6.1] or by the producer, or be determined with respect to withdrawal failure of cladding fasteners.
- \( t_{f,ef} \) is the effective failure time of the cladding;
- \( b \) is the flange width in mm.

For failure during Charring Phase 2, i.e. that failure takes place at or before the time of failure of the cladding, the notional charring depth, \( d_{char,n} \), should be taken as:

\[ d_{char,n} = \beta_h (t - t_{ch}) \]  
(6.84)

where:

\[ \beta_h = \beta_0 k_{b,ch} k_2 k_n \]  
(6.85)

and \( k_2 = 1 \).

### 6.6.4.3 Failure of cladding fasteners

The penetration length of fasteners for fixing claddings or resilient channels should be at least 10 mm into unburnt wood. The charring depth may be taken as the notional charring depth \( d_{char,n} \).

### 6.6.4.4 Strength parameters

For I-joists in bending where the fire-exposed flange is in tension, the modification factor for bending strength, \( k_{mod,fm,fi} \), should be calculated as:

\[ k_{mod,fm,fi} = 1 - 0.016 d_{char,n} k_{b,fm} k_{hf,fm} k_{h,fm} \]  
(6.86)

With

\[ k_{b,fm} = 0.76 + \frac{11.5}{b} \]  
(6.87)

\[ k_{hf,fm} = \frac{68}{h_f} - 0.41 \]  
(6.88)

\[ k_{h,fm} = 1.4 - \frac{80}{h} \]  
(6.89)

where the notional charring depth \( d_{char,n} \), the flange width \( b \), the flange depth \( h_f \) and cross-section depth \( h \) are in mm.

For the influence of finger joints, see Sub-clause 6.5.4.1.5 below.

For shear strength verification of the web, the maximum temperature of the web should be calculated as:

\[ \Theta_{w,\text{max}} = \frac{160 k_b d_{char,n}}{h_f} - 47 \]  
(6.90)
For wood-based webs, the modification factor for shear strength, $k_{\text{mod,fv,fi}}$, may be calculated, using the reduction factor for shear strength given in Figure B4 of EN 1995-1-2 [6.1], as:

$$
k_{\text{mod,fv,fi}} = \begin{cases} 
1 & \text{for } \frac{48k_{b,\text{ch}}d_{\text{char,n}}}{h_f} \leq 20 \\
1,47 - 1,13 \frac{k_{b,\text{ch}}d_{\text{char,n}}}{h_f} & \text{for } \frac{48k_{b,\text{ch}}d_{\text{char,n}}}{h_f} > 20
\end{cases}
$$

(6.91)

For shear strength verification of the glue-line between the web and the flange, the temperature in degrees Celsius should be taken as:

$$
\Theta_{\text{joint}} = \max \left\{ \frac{666d_{\text{char,n}}k_{b,\text{ch}}}{\sqrt{b}h_f} - 12, 20 \right\}
$$

(6.92)

where $d_{\text{char,n}}$, $b$, and $h_f$ are in mm.

6.6.4.5 Finger-joint strength in flanges

Finger joint strength in fire may be dependent on the adhesive being used [6.10]. Since I-joists are sensitive to finger joint failure, the bending resistance should be determined for the design bending strength given as, replacing Expression (6.19):

$$
f_{m,\text{f,fi}} = k_{\text{mod,fi}} k_{\text{mod,fj,fi}} k_{n} \frac{f_k}{\gamma_{M,fi}}
$$

(6.93)

where $k_{\text{mod,fj,fi}}$ is the modification factor for fire, expressing the reduction of finger joint strength given in Table 6.13.

### Table 6.13. Modification factors for fire for finger joints.

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>$k_{\text{mod,fj,fi}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PRF (phenolic resorcinol formaldehyde)</td>
<td>1.0</td>
</tr>
<tr>
<td>MUF (melamine urea formaldehyde)</td>
<td>0.75</td>
</tr>
<tr>
<td>PUR (polyurethane)</td>
<td>0.75</td>
</tr>
</tbody>
</table>

6.6.4.6 Reduced cross-section method for bending strength

For application of the reduced cross-section method, see 6.5.1 or 6.6.3.2, the depth of the zero-strength layer should be calculated as, see Figure 6.24:

$$
d_0 = 5,3 + 0,165h_f - 0,018b - 0,0006h_f b
$$

(6.94)
6.6.5 Other structural elements

6.6.5.1 Timber decks made of hollow core elements

6.6.5.1.1 General

The following design model is taken from [6.47]. It was developed based on fire tests of timber decks made of hollow core elements with either void or insulation filled cavities. Stone wool typically used in Switzerland was used for the fire tests performed with filled cavities. In the design model, strength properties are determined using the reduced cross-section method.

6.6.5.1.2 Charring

The charring model takes into account two different charring phases as shown in Figure 6.25. The first charring phase is given by the time needed for complete burn-through of the fire-exposed bottom timber layer, i.e. $d_{\text{char},n} \leq h_u$). The second charring phase is characterised by charring of the vertical timber members (the webs) after the charring depth has reached the thickness of the fire-exposed bottom timber layer ($d_{\text{char},n} \geq h_u$). For simplicity, linear relationships between charring depth and time are assumed for each phase. Further, it is assumed that the vertical timber members are not exposed to fire on three sides during the required fire resistance. Thus the fire-exposed bottom timber layer is designed such that a fire penetration into the cavities is prevented, or the cavities are filled with mineral wool batts with a degradation point greater than 1000°C, remaining in place after the charred fire-exposed bottom timber layer has fallen off.
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Figure 6.25. Charring model for the calculation of the residual cross-section of the hollow core elements.

The time $t_1$, when the charring depth has reached the thickness of the fire-exposed timber layer ($d_{\text{char},n} = h_u$) should be calculated using the notional charring rate $\beta_{1,n}$ for the first charring phase as following:

$$t_1 = \frac{h_u}{\beta_{1,n}} \quad (6.95)$$

For a required time of fire resistance, $t_{\text{req}}$, the notional charring depth for the vertical members of the hollow core elements should be calculated as:

$$d_{\text{char},n} = \beta_{1,n} t_{\text{req}} \quad \text{for } 0 \leq t_{\text{req}} \leq t_1 \quad (6.96)$$

$$d_{\text{char},n} = h_u + \beta_{2,n} (t_{\text{req}} - t_1) \quad \text{for } t_{\text{req}} \geq t_1 \quad (6.97)$$

Since timber decks made of hollow core elements are load-bearing, charring rates should be taken from EN 1995-1-2 [6.1] Table 3.1 in rows a to c, but not d (wood-panelling). For the first charring phase, due to joints between the hollow core elements, the influence of increased charring should be taken into account by using notional charring rates given in table 3.1 in [6.1] rather than the one-dimensional values, i.e. $\beta_{1,n} = 0.8 \text{ mm/min}$ for hollow core elements made of solid timber. The notional charring rate $\beta_{2,n}$ for the second charring phase is mainly influenced by the thickness of the vertical members and should be calculated as

$$\beta_{2,n} = \beta_{1,n} k_s k_n \quad (6.98)$$

where the coefficient $k_s$ should be taken from Expression (6.32) and $k_n = 1.5$. Expression (6.32) takes into account the protective function of the wide sides of the vertical member using stone wool from various producers.

6.6.5.1.3 Strength and stiffness properties

The reduced cross-section method should be used for the design of timber decks made of hollow core elements, see 6.4. Calculate the effective charring depth in accordance with Expression (6.99), using the values of $d_0$ as given in Table 6.14.

<table>
<thead>
<tr>
<th>Charring phase</th>
<th>$d_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>First charring phase</td>
<td>$d_{\text{char},n} \leq h_u$</td>
</tr>
<tr>
<td>Second charring phase</td>
<td>$d_{\text{char},n} &gt; h_u$</td>
</tr>
</tbody>
</table>
6.6.5.2 Timber-concrete composite slabs

6.6.5.2.1 General

The following design model is taken from [6.48][6.49]. The design model is based on the reduced cross-section method and takes into account the temperature-dependent reduction of stiffness and strength of the connection.

6.6.5.2.2 Charring

The notional charring rate should be used (see 6.4.3) and the corresponding charring depths calculated according to Expression (6.2).

6.6.5.2.3 Strength and stiffness properties

The reduced cross-section method should be used for the design of timber-concrete composite slabs, see 6.4. Calculate the effective charring depth in accordance with Expression (6.100), (see Figure 6.26).

![Initial cross section](image1)

![Effective cross section](image2)

*Figure 6.26. Determination of the effective cross-section in fire.*

The timber board protects the concrete slab from the influence of high temperatures. As the reduction of stiffness and strength properties of concrete are negligible at temperatures up to 200°C, the properties of concrete at normal temperature may be assumed for a fire exposure of \( t \leq 60 \) minutes and a timber board thickness of \( h_s \geq 20 \) mm.

The temperature-dependent reduction of stiffness and strength of the connection is taken into account using the modification factor \( k_{mod,fi} \). For connections with screws arranged at an inclination of ± 45°, the modification factors \( k_{mod,fi} \) depend on the side cover \( x \) of the connectors (see Figure 6.26) and are given in Table 6.3. The influence of heat flux from the bottom and the opposite side of the connector may be neglected if \( x_u \geq x + 20 \) mm and \( x_s \geq 20 \) mm.
Table 6.15. Modification factor $k_{\text{mod,fi}}$ for fire, taking into account the effects of temperature on the mechanical properties of the screwed connection, where $x$ is the side cover in mm as shown in Figure 6.26 and $t$ is the fire duration time in minutes.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$k_{\text{mod,fi}}$</th>
<th>valid for</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slipp modulus</td>
<td>$0,2x-0,12t$</td>
<td>$x \leq 0,6t$</td>
</tr>
<tr>
<td>of screwed connection</td>
<td>$0,2t+3$</td>
<td>$0,6t \leq x \leq 0,8t+3$</td>
</tr>
<tr>
<td></td>
<td>$0,8x-0,6t+1,8$</td>
<td>$0,8t+3 \leq x \leq t+24$</td>
</tr>
<tr>
<td></td>
<td>$0,2t+21$</td>
<td>$x \geq t+24$</td>
</tr>
<tr>
<td></td>
<td>$1,0$</td>
<td></td>
</tr>
<tr>
<td>Strength of screws</td>
<td>$0,44x-0,264t$</td>
<td>$x \leq 0,6t$</td>
</tr>
<tr>
<td>connection</td>
<td>$0,2t+5$</td>
<td>$0,6t \leq x \leq 0,8t+5$</td>
</tr>
<tr>
<td></td>
<td>$0,56x-0,36t+7,32$</td>
<td>$0,8t+5 \leq x \leq t+28$</td>
</tr>
<tr>
<td></td>
<td>$0,2t+23$</td>
<td>$x \geq t+28$</td>
</tr>
<tr>
<td></td>
<td>$1,0$</td>
<td></td>
</tr>
</tbody>
</table>

The expressions given in Table 6.15 for the temperature-dependent strength of the screwed connection are the same as in [1], for axially loaded screws.

The normal stresses in timber and concrete, the shear stress in the timber and the connection forces can be calculated according to the simplified method ($\gamma$-method) for mechanically jointed beams with flexible elastic connections given in EN 1995-1-1 Annex B, using the effective cross-section as shown in Figure 6.26 and the modification factors $k_{\text{mod,fi}}$ for fire according to Table 6.15, taking into account the effects of elevated temperature on the mechanical properties of the screwed connection.

6.7 Calculation vs. full-scale testing

Structural fire design by calculation uses partial factors for actions and materials in order to achieve a required reliability level. For timber structures, for example, design values for resistance are based on the 20 % fractile of strength, see EN 1995-1-2 [6.1] and 6.5.1 above.

The traditional way of structural fire design of building elements was carried out with the aid of full-scale fire tests. The test procedure is given in a series of European standards, such as EN 1365-1 [6.50]. The results are presented according to a classification standard EN 13501-2.

In design by testing, the procedures applied at ambient temperature are different from the procedure for the application of the results from fire tests. Evaluating the results from tests at ambient temperature, the result (resistance) is transformed into a characteristic resistance with respect to the strength and/or stiffness properties of the specimens in relation to the corresponding characteristic values (for more information see EN 1990 [6.31]). The results from fire tests are normally directly applied. For example, when a timber frame wall assembly is to be tested, the timber is normally randomly selected from the specific strength class to be used by the client. Since it is most likely that the strength of the studs scatter around mean values rather than 20-percentile values within the strength class in question, a direct application of the test results would lead to non-conservative mechanical resistance in comparison with the reliability level assumed by the Eurocodes. In the case of fire testing of walls, bending stiffness is the dominating parameter. It would be sufficient, prior to assembling the wall, to perform non-destructive bending stiffness tests and to calculate the characteristic resistance taking into account the real stiffness properties of the tested wall. In the case of fire testing of floors, it would be best to select more joists than needed for the fire tests. A part of
the material should be tested in bending (destructive bending tests) in order to determine (ambient) bending strength.

In order to eliminate inconsistency between the reliability system of Structural Eurocodes and application of results from full-scale fire testing, the procedure of selecting materials should be improved and include methods for determining strength and stiffness properties of the test specimens to be tested.

6.8 References


[6.12] Frangi A, König J. Effect of increased charring on the narrow side of rectangular timber cross-sections exposed to fire on three or four sides. Accepted for publication in Fire and Materials, 2010.


Chapter 6 – Load-bearing structures – Excerpt for JRC and CEN TC250/SC5


Chapter 7 – Timber connections – Excerpt for JRC and CEN TC250/SC5

7 Timber connections

This chapter presents basic requirements for timber connections. The calculation methods in Eurocode 5 are complemented with other design methods from recent research. Both timber-to-timber and steel-to-timber connections are included. The models are applied and worked examples are presented.

7.1 General

One of the several important factors in improving the fire safety of timber buildings is a thorough knowledge of the fire behaviour of connections between members. Among the various structural components, connections are key elements because of their variety of configurations. In a fire situation and in normal conditions, they determine the load-carrying capacity of the structure and its safety. In structural analyses, timber connections are usually considered either fully rigid or fully hinged, but in reality their behaviour is semi-rigid. Knowledge of the behaviour of timber connections in fire is necessary in order to perform adequate analysis and modelling of the structures at the Ultimate Limit State.

This chapter presents two approaches to calculation methods. The first is that of Eurocode 5 (EN 1995-1-1, EN 1995-1-2) [7.1, 7.2], while the second applies connection design methods developed, based on the results from experimental studies and numerical simulations.

References [7.3-7.5] describe the fire behaviour of multiple-shear steel-to-timber connections using dowels and slotted-in steel plates, based on extensive experimental and numerical analysis. The work proposes an analytical model for calculation of the fire resistance of this type of connection for fire resistances up to 60 minutes.

References [7.6-7.11] present experimental and numerical studies. They concern the timber connections using mainly bolts and dowels loaded in tension parallel to the grain. The available results for dowelled connections in tension parallel to the grain are compared with the Eurocode formulae that allow an extension of their domain of validity.

References [7.12, 7.13] present a component model for dowelled timber-to-timber or timber-to-steel connections. It is based on analysis of a steel beam on a continuous foundation (wood in embedment) using a finite-element model for thermal and mechanical analyses.

Other works concerning connections are presented in references [7.14-7.16, 7.19, 7.20] for bolted and dowelled types and in reference [7.17] for nailed types.

The main objective of a timber connection is to guarantee the mechanical resistance (R) of load-bearing structures for at least a required time in order to allow safe evacuation of the building and to ensure the safety of fire-fighters. The required time is normally expressed in terms of fire resistance using the ISO standard fire exposure, and is specified by the building regulations of each country. As far as timber connections are concerned, the majority of the available fire test results have been produced during the last ten years. Over the same period, numerical models of timber connections have been developed in some European countries. The available data concern mainly dowelled and bolted timber-to-timber and timber-to-steel connections.
Chapter 7 – Timber connections – Excerpt for JRC and CEN TC250/SC5

7.2 The EN 1995-1-2 design method

EN 1995-1-1 [7.1] provides a general methodology for the design and calculation of timber connections in normal conditions, while EN 1995-1-2 [7.2] provides a corresponding methodology for the design of timber connections exposed to fire. Formulae (8.6) and (8.7) in EN1995-1-1, used for the design of timber connections, are based on Johansen yield theory [7.18] with the rope effect contribution. They can be applied to the nails, staples, bolts, dowels and screws per shear plane per fastener. The rope effect is negligible in fire conditions. These formulae are used in one of the proposed Eurocode methods for fire situations.

Investigations for an analytical model are based on the Johansen yield theory [7.18] using an embedment strength reduction with temperature. The reduction of steel fastener characteristics with temperature is not significant for the parts of the fastener in contact with the “active” wood in embedment (not charred parts).

Under normal conditions, the load-bearing capacity of connections can be calculated using the EN 1995-1-1 rules based on a plastic limit state design. This approach uses the plastic bending resistance of the fastener and the embedment strength of timber. The embedment strength is given by formulae using two parameters: the fastener diameter, the wood density and, possibly also, the angle of the load to the grain. The wood density and the fastener diameter are also the two parameters used to calculate the initial stiffness of the connection (slip modulus, $K_{sr}$). At the ultimate limit state, the slip modulus of a connection $K_u$ is related to $K_{sr}$, and used for the structural analysis (internal loads distribution). The plastic approach considers that the fastener in bending and the wood in embedment exhibit a rigid plastic behaviour. This hypothesis is normally checked for connections loaded in the direction parallel to the grain, and respecting the minimum spacing and edge and end distances as given in Eurocode. When a force in a connection acts at an angle to the grain, the possibility of splitting caused by the tension force component perpendicular to the grain must be taken into account. A formula is available to take account of the possibility of splitting caused by the tension force component perpendicular to the grain (according to EN 1995-1-1, subclause 8.1.4). In addition, for steel-to-timber connections comprising multiple dowel-type fasteners subjected to a force component parallel to the grain near the end of the timber member, the characteristic load-carrying capacity of fracture along the perimeter of the fastener area (block shear failure and plug shear failure), should be considered. This means that the load-carrying capacity of timber connections, calculated according to the Johansen theory, can be reduced by other types of failures, depending on the configuration of the connections.

EN 1995-1-2 gives a method for designing timber connections under standard fire conditions for fire resistance not exceeding R60. In this European standard, design rules are given for connections made with nails, bolts, dowels, screws, split-ring connectors, shear-plate connectors and toothed-plate connectors.

For the connections with wood-side members (for more details: see EN 1995-1-2 clause 6.2), the design approaches concern:

- Simplified rules for unprotected and protected connections, and some additional rules for connections with internal steel plates.
- A reduced load method for unprotected and protected connections.

For connections with external steel plates (according to EN 1995-1-2 Clause 6.3), the approach concerns the design of protected or unprotected connections.

Clause 6.4 of EN 1995-1-2 presents simplified rules for axially loaded screws.

7.3 Other methods proposed for the design of timber connections

These “other” methods combine test results, finite-element approaches and analytical formulae related directly or indirectly to the Eurocode principles. They can be considered as a complement to Eurocode because they cover types of connections or validity domains which can complement the Eurocode approaches. In reality, these studies are still at the research stage, and the results can be used as a step to future improvement or as a complement to the Eurocode approaches. Nevertheless, the methods proposed can be used as a basis for designing the connections for the dimension ranges defined in this guideline. As explained above, as they are validated by experimental results, their application results in safe design.

In these methods, the calculation approaches apply to the design of timber-to-timber and steel-to-timber connections with dowel and bolt fasteners. As far as steel-to-timber connections are concerned, the calculation methods apply to connections with one, two or three slotted-in steel plates.

7.3.1 Timber-to-timber connection

Figure 7.1 is a schematic representation of an experimental realisation of a timber-to-timber connection. In order to avoid separation of the main and the side members, including connections using dowel fasteners, bolts are used near the ends of the external members ($t_i$) (see black circles in Figure 7.1). The connections are designed according to EN 1995-1-1.

The formulae given for these connections are valid for glued laminated members of Strength Class GL24h and GL28. They could be used for connections composed of other wood species if the charring rate is equal to or less than the charring rate of the tested glulam members, and if the connections are designed according to EN 1995-1-1. The same type of formulae as in EN 1995-1-2 is used to calculate the characteristic load-carrying capacity of the connection in shear (Equation 7.1). However the values of the parameter $k$ are modified as well as the domain of validity. The conversion factor $\eta$ is calculated according to equation 7.2.

\[
F_{d,t,fi} = \eta \times F_{v,Rk} \quad (7.1)
\]

with:

\[
\eta = e^{-k \cdot f_i} \quad (7.2)
\]

where:

- $F_{d,t,fi}$ is the design value of load-bearing capacity of the connection in shear under standard fire
- $F_{v,Rk}$ is the characteristic lateral load-carrying capacity of the connection with fasteners in shear at normal temperature (see EN 1995-1-1 section 8).
Figure 7.1. Type of tested timber-to-timber connections.

Table 7.1. Parameter k for the timber-to-timber connections.

<table>
<thead>
<tr>
<th>Fasteners</th>
<th>Timber element</th>
<th>k</th>
<th>Maximum period of validity for parameter –k- (unprotected connections)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolts</td>
<td>( d \geq 12 \text{ mm} )</td>
<td>( t_1 \geq 45 \text{ mm} )</td>
<td>0.060</td>
</tr>
<tr>
<td>Dowels</td>
<td>( d \geq 12 \text{ mm} )</td>
<td>( t_1 \geq 60 \text{ mm} )</td>
<td>0.035</td>
</tr>
</tbody>
</table>

Parameter \( k \) is defined in Table 7.1, where:

- \( t_1 \) is the thickness of the timber side members in the connection
- \( d \) is the fastener diameter
- \( t_\beta \) is the required fire resistance period.

For bolted connections, if \( t_\beta \) required is more than 30 minutes, the thickness of the timber side members of the connections must be increased by the following value (Equation 7.3).

\[
\Delta t = (t_\beta - 30) \cdot \beta_n
\]  

(7.3)

For dowelled connections, if \( t_\beta \) required is more than 60 minutes, the thickness of the timber side members of the connections must be increased by the following value (Equation 7.4).

\[
\Delta t = (t_\beta - 60) \cdot \beta_n
\]  

(7.4)

where:

- \( \Delta t \) is the thickness added to each timber side member of the connection,
- \( \beta_n \) is the design notional charring rate under standard fire exposure (see Table 3.1 of EN 1995-1-2).

The design model is based on the following assumptions:

- \( F_{v,Rk} \) is the characteristic lateral load-carrying capacity of the connection with fasteners in shear at normal temperature (see EN 1995-1-1 Section 8)
- \( a_1=7d, a_2=4d, a_3=7d, a_4=3d \) (see Figure 7.3), \( d \) is the fastener diameter
- the distances \( a_1 \) and \( a_2 \) are increased by \( a_1 = \beta_n \cdot f_{\text{flux}} \cdot (t_\beta - t_{d,\beta}) \)
where:

- $\beta_s$ is the charring rate for glulam as given in Table 3.1 of EN1995-1-2 (equal to 0.7)
- $k_{\text{flux}}$ is a coefficient taking into account increased heat flux through the fastener (equal to 1.5)
- $t_{\text{f,0}}$ is the required standard fire resistance duration
- $t_{d,\text{f}}$ is the fire resistance duration of the unprotected connection given in Table 7.1

- the characteristic density of timber element $\rho_i$ is taken into account in the calculations.

Note: For nailed connections the k parameter (see table 6.3 of Eurocode 5-1.2) may be modified. It has been observed that the fire resistance of nailed connections is slightly higher than for timber-to-timber connections with bolt fasteners with $d \geq 12$ mm. For that reason $k=0.06$ can be used for nailed connections. The maximum period of validity for this $k$ parameter in an unprotected connection is 30 minutes. If $t_{\text{f,0}}$ required is more than 30 minutes, the thickness of the timber side members of the connections must be increased by the value in Equation 7.3, and the nails have to be protected by the same thickness.

### 7.3.2 Steel-to-timber connection

#### 7.3.2.1 Steel-to-timber connection with one slotted-in steel plate

##### 7.3.2.1.1 Introduction

Figure 7.2 illustrates a steel-to-timber connection tested experimentally. In order to avoid separation of the main and side members, including connections with dowel fasteners, bolts are used near the ends of the external members ($t_1$) (see black circles in Figure 7.2). The connections are designed according to EN 1995-1-1.

The formulae given for these connections are valid for glued laminated members of Strength Class GL24h. They could be used for connections composed of LVL and of other wood products if the charring rate is equal to or less than the charring rate of the tested glulam members, and if the connections are designed according to EN 1995-1-1.

![Figure 7.2. Type of steel-to-timber connections.](Image)

The same formulae as for timber-to-timber connection can be used but with the factor $k$ taken from Table 7.2.
- $t_{d,fi}$ is the fire resistance period of the unprotected connection given in Table 7.2.

### Table 7.2. Parameter $k$ for the steel-to-timber connections.

<table>
<thead>
<tr>
<th>Fasteners</th>
<th>Timber element</th>
<th>$k$</th>
<th>Maximum period of validity for parameter $k$ (unprotected connections)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolts $d \geq 12$ mm</td>
<td>$t_1 \geq 45$ mm</td>
<td>0.070</td>
<td>30 min</td>
</tr>
<tr>
<td>Dowels $d \geq 12$ mm</td>
<td>$t_1 \geq 60$ mm</td>
<td>0.045</td>
<td>60 min</td>
</tr>
</tbody>
</table>

If the required time to failure ($t_{fi}$) is more than 30 minutes for the bolted connections, or 60 minutes for the dowelled connections, the thickness of the timber side members of the connections must be increased by the values given in Equations 7.3 and 7.4 respectively. The thickness of the steel member (plate) in the connection is defined in accordance with EN 1995-1-1. The design model is based on the same assumptions as the timber-to-timber model (see 7.3.1).

#### 7.3.2.1.2 Worked example

Calculate the characteristic load-carrying capacity of steel-to-timber connection with dowel fasteners (Figure 7.3) in accordance with EN 1995-1-1, using the following equations.

$$ F_{v,Rk} = \min \left\{ \begin{array}{ll} \left( \frac{f_{h,a,k} t_d}{2 + \frac{4M_{v,Rk}}{f_{h,a,k} d t^2}} - 1 \right) \sqrt{M_{y,Rk} f_{h,a,k} d} & \text{mode 1} \\ 2.3 \sqrt{M_{y,Rk} f_{h,a,k} d} & \text{mode 3} \end{array} \right. $$

![Figure 7.3. Configuration of the steel-to-timber connection (d=16 mm).](image)

For a required fire resistance of 60 minutes:

The timber elements are of strength class GL28 ($\rho_s = 410$ kg/m³). The fastener (dowel) diameter is $d=16$ mm with: $f_u = 400$ N/mm².

The dimensions of the elements and the end and edge distances are:
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$t_i = 76 \text{ mm}, h = 252 \text{ mm}, \delta_p = 8 \text{ mm} \text{ and } h_p = 114 \text{ mm}.

\[ a_1 = 7d \text{, } a_2 = 4d \text{, } a_3 = 7d \text{, } a_4 = 3d \text{ and } e_1 = 25 \text{ mm} \]

The distances \( a_3 \) and \( a_4 \) are increased by: \( a_{3,4} = \beta_{h} \cdot k_{\text{flux}} \cdot (t_{f} - t_{d,fi}) \)

\( t_{d,fi} \): for the connection using a combination of dowel and bolt fasteners, the value taken into account is equal to 15.

The connection is loaded in tension parallel to grain (\( \alpha = 0^\circ \)) and the embedment strength of timber (\( f_{h,0,k} \)) is calculated according to the following formula:

\[ f_{h,0,k} = 0.082(1 - 0.01d)\rho_k \]

The characteristic values for yield moment is calculated according to the following formula:

\[ M_{y,Rk} = 0.3 f_{u,k}d^{2.6} \]

The characteristic load-carrying capacity of the steel-to-timber connection, in cold conditions, is calculated according to Equation 7.5. Its value is \( F_{v,Rk} \approx 274 \text{ kN} \).

For this connection, as \( \eta = e^{-k_{ts}} = e^{-0.04560} = 0.0672 \), the design value of the load-bearing capacity \( F_{d,T,fi} \) is given by: \( F_{d,T,fi} = 0.0672 \cdot 274 = 18.4 \text{ kN} \).

For this connection, in order to obtain a fire resistance of R60, it is possible to apply a load \( E_{d,fi} \) lower than or equal to 18.4 kN (\( E_{d,fi} \leq F_{d,T,fi} \)).

\( E_{d,fi} \) is the design effect of actions in fire situation.

### 7.3.2.2 Steel-to-timber connection with two or three slotted-in steel plates and dowels

#### 7.3.2.2.1 Introduction

An analytical design model for calculation of the load-carrying capacity in fire of unprotected multiple-shear steel-to-timber dowelled connections was developed in the same way as for the reduced cross-section method given in EN 1995-1-2, and based on a combination of experimental and numerical analysis [7.3-7.5]. The model is based on Failure Mode I according to the Johansen yield model (i.e. embedment failure) and takes into account the influence of the steel elements (i.e. steel plates and steel dowels) on the charring of the connection. The model was developed for a fire resistance up to 60 minutes.

The effective cross-section is calculated by reducing the initial cross-section by the effective charring depth (\( d_{s} \)), as shown in Figure 7.4. The temperature-dependent reduction of strength and stiffness of timber in the heat-affected zones as well as the effects of corner roundings are considered by adding a further layer (\( d_{red} \)) to the charring depth (\( d_{char} \)). For simplicity, the same value of (\( d_{red} \)) is used for charring on side (index - s) and on top/bottom (index - o).

\[
d_{ef,s} = d_{char,s} + d_{red} \quad (7.5)\]

\[
d_{ef,o} = d_{char,o} + d_{red} \quad (7.6)\]
The design value of the load-carrying capacity $R_{d,k}$ of the connection loaded in tension parallel to the grain can be calculated as follows:

$$R_{d,k} = A_{ef} \cdot f_{t,0,k} \cdot k_f$$  \hspace{1cm} (7.7)

where:

- $R_{d,k}$ is the design value of the load-carrying capacity of the connection
- $A_{ef}$ is the effective cross-section ($A_{ef} = b_{ef} \cdot h_{ef}$)
- $f_{t,0,k}$ is the characteristic tensile strength of timber parallel to the grain direction

![Diagram of residual and effective cross-sections](image)

Figure 7.4. Residual ($A_r$) and effective cross-section ($A_{ef}$) for the determination of the load-carrying capacity of multiple shear steel-to-timber dowelled connections in fire, shown for the example of one quarter of cross-section of a connection with three slotted-in steel plates (SD = steel dowel).

The design model is based on the following assumptions:

- Unprotected multiple-shear dowelled connections with two or three slotted-in steel plates. Spacings, edge and end distances of the dowels according to EN 1995-1-1 for ambient-temperature design (except for the spacing between dowels parallel to grain direction: $a_1=7d$ instead of $a_1=5d$, where $d$ is the diameter of the steel dowels). A comparative numerical analysis showed that the assumption of a spacing $a_1=5d$ leads to a reduction of the load-carrying capacity $R_{d,k}$ of the connection in the range of 8 to 10%.
- ISO fire exposure on four sides.
- Glued laminated timber members with a minimum width of $b \geq 160 \text{ mm}$ and a minimum thickness of the timber side member $t_1 \geq 35 \text{ mm}$ (see Figure 7.4). The thickness of the timber middle member $t_2$ (see Figure 7.4) is $8d$, as normally required for design at ambient temperature for failure mode III.
- For one ($n=1$) or two ($n=2$) dowels in one row parallel to the load direction, at least strength class GL24h is required. In order to avoid net cross-section timber failure, strength class GL36h should be used for connections with three dowels ($n=3$) in one row parallel to the load direction (see Figure 7.5).
- Embedment failure, i.e. failure mode I according to the Johansen yield model [7.18] (because of charring of the timber side members, embedment failure was observed during all fire tests).

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In order to provide a fire resistance of 60 minutes, the size of the timber members must be increased by a thickness \(c\) (see Fig. 7.6). If the width \(b\) of the timber members is less than 200 mm, then a thickness of \(c=45\) mm is required, while for \(b\geq 200\) mm a thickness of \(c=40\) mm is required. For a fire resistance between 30 and 60 minutes, the thickness \(c\) can be linearly interpolated as follows:

\[
\begin{align*}
    c &= 1.5 \cdot t - 45 & \text{for } b < 200\ \text{mm and } 30 \leq t \leq 60\ \text{minutes} \\
    c &= 4/3 \cdot t - 40 & \text{for } b \geq 200\ \text{mm and } 30 \leq t \leq 60\ \text{minutes}
\end{align*}
\]

While the timber size is increased by a thickness \(c\), the width of the steel plates should not be changed. Thus, an air gap is created leading to better protection of the steel plates against heat due to the insulation effect of the air gap, particularly during the first phase of fire exposure (see Figure 7.6). Increasing the overall thickness of the timber members by a thickness \(c\) would not apply when adequately protecting the connection with timber boards or gypsum plasterboards.

**7.3.2.2.2 Residual cross-section**

An extensive FE-thermal analysis on a large number of geometries of multiple-shear dowelled connections commonly used showed that the side charring \(d_{\text{char, s}}\) is mainly influenced by the thickness of the timber side member \(t_1\), (see Figure 7.3). The required minimum thickness of the timber side member \(t_1 \geq 35\) mm and the required increased size of the timber members by a thickness \(c\) for a fire resistance between 30 and 60 minutes allowed the development of the simplified charring model for the side charring as follows:
The charring model is based on the one-dimensional charring rate $\beta_0$ and is characterised by two charring phases. For simplicity linear relationships between charring depth and time are assumed for each phase. The influence of the steel plate on charring leads to an increased charring rate of $1.5 \cdot \beta_0$ during the second charring phase ($30 \leq t \leq 60$ minutes).

The results of the extensive FE-thermal analysis showed that the top/bottom charring can be calculated assuming an increased charring rate of $(1,1 \beta_0)$ from beginning up to a fire duration of 60 minutes as follows:

$$d_{\text{char},o} = 1.1 \cdot \beta_0 \cdot t \quad \text{for } 0 \leq t \leq 60 \text{ minutes}$$ (7.12)

### 7.3.2.2.3 Effective cross section

The depth ($d_{\text{red}}$) required for the calculation of the effective cross-section (see Figure 7.4) depends mainly on the following three parameters:

- the ratio between initial width ($b$) of the cross-section and width ($b_r$) of the residual cross-section
- the time of fire exposure ($t$)
- the number of dowels ($n$) in one row parallel to the load direction (see Figure 7.5)

As the minimum thickness of the timber middle member ($t_2$) required for the design at ambient temperature is ($8d$), and timber cross-sections with a width greater than 300 mm are unusual, connections with three slotted-in steel plates were analysed only for dowel diameters up to 10 mm (for 30 minutes' fire resistance) as well as 8 mm (for 60 minutes' fire resistance).

The depth ($d_{\text{red}}$) can be calculated according to Equations 7.13 to 7.16 for a fire resistance of 30 minutes, and Equations 7.17 to 7.20 for a fire resistance of 60 minutes. Linear interpolation can be employed for a fire resistance between 30 and 60 minutes.

#### Fire resistance of 30 minutes

- Multiple shear steel-to-timber connection with two slotted-in steel plates:

  $$d_{\text{red}} = -60(b/b_r) - 0.4d + 126.5 \quad [\text{mm}] \quad \text{with } 8 \leq d \leq 16 \text{ mm}$$ (7.13)

  $$d_{\text{red}} = -40(b/b_r) - n(0.5d - 2) + 94 \quad [\text{mm}] \quad \text{with } 8 \leq d \leq 16 \text{ mm}$$ (7.14)

- Multiple shear steel-to-timber connection with three slotted-in steel plates:

  $$d_{\text{red}} = -60(b/b_r) - 0.4d + 133 \quad [\text{mm}] \quad \text{with } 6 \leq d \leq 10 \text{ mm}$$ (7.15)

  $$d_{\text{red}} = -40(b/b_r) - 0.4d(n+2) + 101 \quad [\text{mm}] \quad \text{with } 6 \leq d \leq 10 \text{ mm}$$ (7.16)
**Fire resistance of 60 minutes**

- Multiple shear steel-to-timber connection with two slotted-in steel plates:

  \[
  n = 1 \quad d_{\text{red}} = -30(b/b_y) - 0.6d + 117 \quad [\text{mm}] \quad \text{with } 8 \leq d \leq 12 \text{ mm} \quad (7.17)
  \]

  \[
  n = 2, 3 \quad d_{\text{red}} = -20(b/b_y) - d(0.2n + 1.4) + 101.5 \quad [\text{mm}] \quad \text{with } 8 \leq d \leq 12 \text{ mm} \quad (7.18)
  \]

- Multiple shear steel-to-timber connection with three slotted-in steel plates:

  \[
  n = 1 \quad d_{\text{red}} = -30(b/b_y) + 115.5 \quad [\text{mm}] \quad \text{with } 6 \leq d \leq 8 \text{ mm} \quad (7.19)
  \]

  \[
  n = 2, 3 \quad d_{\text{red}} = -20(b/b_y) - 4n + 94 \quad [\text{mm}] \quad \text{with } 6 \leq d \leq 8 \text{ mm} \quad (7.20)
  \]

The calculation of \( d_{\text{red}} \) is based on strength class GL24h for connections with one or two dowels in one row parallel to the load direction, while strength class GL36h is assumed for connections with three dowels in one row (see Figure 7.5). For other strength classes, the load-carrying capacity in fire \( R_{d,f} \) of connections with one or two dowels in one row parallel to the load direction can be increased with the conversion factors given in Table 7.3.

<table>
<thead>
<tr>
<th>Number of dowels within one row [-]</th>
<th>GL 24h</th>
<th>GL 28h</th>
<th>GL 32h</th>
<th>GL 36h</th>
</tr>
</thead>
<tbody>
<tr>
<td>( n = 1 )</td>
<td>1,00</td>
<td>1,08</td>
<td>1,13</td>
<td>1,18</td>
</tr>
<tr>
<td>( n = 2 )</td>
<td>1,00</td>
<td>1,08</td>
<td>1,13</td>
<td>1,18</td>
</tr>
<tr>
<td>( n = 3 )</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>1,00</td>
</tr>
</tbody>
</table>

**Table 7.3. Conversion factors for the calculation of the load-carrying capacity in fire \( R_{d,f} \) of multiple shear steel-to-timber connections taking into account different strength classes.**

### 7.4 References


