Fire resistance of timber members

STEP lecture B17
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Objective

To present calculation methods for structural fire design.

Prerequisites

A4 Wood as a building material
A13 Behaviour of timber and wood-based materials in fire

Summary

The calculation methods for structural fire design according to EC5: Part 1-2 "General rules - Supplementary rules for structural fire design" are discussed and a comparison between these methods is shown with the help of an example.

Introduction

Generally the same principles are followed to calculate fire resistance as in standard design. Thus for actions and for material properties characteristic values are applied. However most fire testing is based on deterministic methods using mean values for strength. In order to ensure the same safety level EC5: Part 1-2, gives approximate calculation methods that satisfy both requirements, see Figure 1.

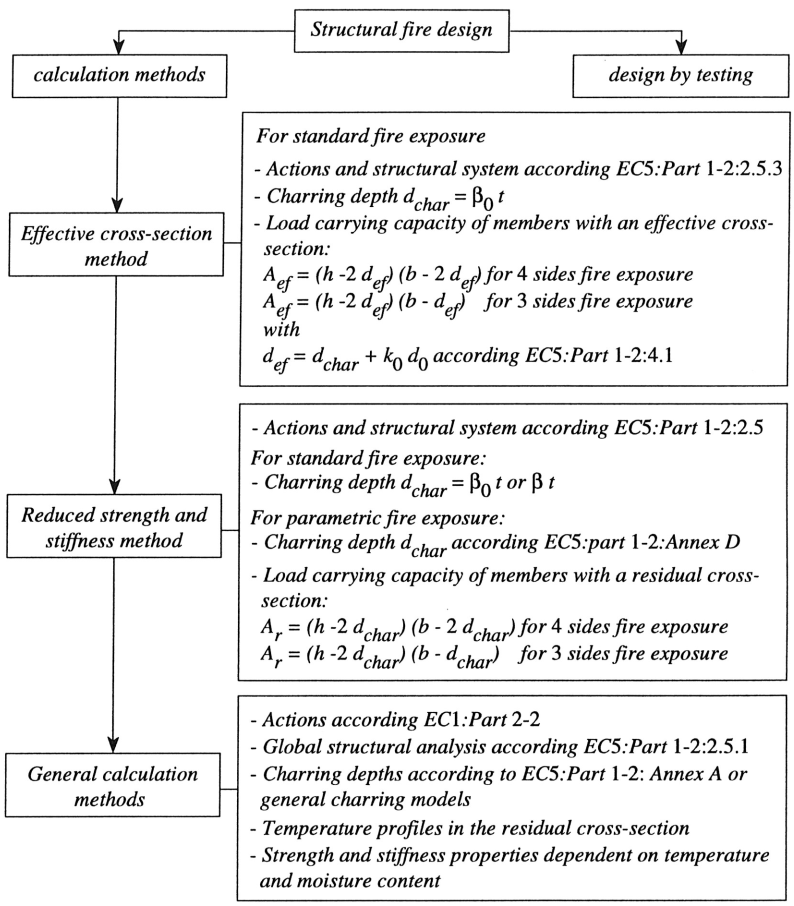


Figure 1 Structural fire design according EC5:Part 1-2

EC5: Part 1-2: 2.5

Verification

The effect of actions E(t) and the resistance of timber members R(t) during fire exposure is in principle shown in Figure 2. The fire resistance is reached at the time t_f when R(t) becomes less than E(t). Thus the verification on the design level is

$$E_{f,d} < R_{f,d} \tag{1}$$

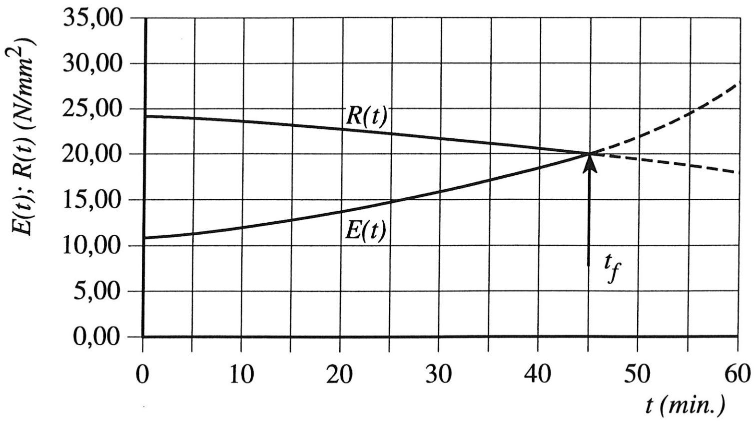


Figure 2 Effect of actions E(t) and resistance of timber members R(t) during fire exposure.

$E_{f,d}$ is the design effect of actions in the event of fire
$R_{f,d}$ is the design resistance in the event of fire

Design values for material properties

EC5: Part 1-2: 2.3

Material properties for thermal analysis

$$X_{f,d} = \frac{X_k(\Theta)}{\gamma_{M,f}} \quad or \quad X_{f,d} = X_k(\Theta)\,\gamma_{M,f} \tag{2}$$

$X_k(\Theta)$ is the characteristic value for material properties at a temperature Θ, depending on whether property increase is favourable for safety or not.

Thermo mechanical properties of strength and modulus of elasticity

For load-carrying verification the design strength and stiffness values shall be determined from

EC5: Part 1-2: 2.3

$$X_{f,d} = k_{mod,f}\, k_f \frac{X_k}{\gamma_{M,f}} \tag{3}$$

For deformation verification the stiffness values shall be taken from

$$E_{f,d} = k_{mod,f} \frac{E_{mean}}{\gamma_{M,f}} \tag{4}$$

$X_{f,d}$ is the design value of material property (strength or modulus of elasticity)
X_k is the characteristic value of material property
$k_{mod,f}$ is a reduction factor taking account of the influence of temperature and moisture content on strength and stiffness in case of fire
k_f is a coefficient which alters the characteristic property to a mean value

k_f	= 1,15 for glued laminated timber and wood based panels
k_f	= 1,25 for solid timber
$\gamma_{M,f}$	= 1,0 is a partial safety factor for material properties

Charring depths

Charring depths for members exposed to fire can be calculated by means of the charring rate (linear relation between charring and time) and the time of exposure to the fire (see STEP lecture A13).

Design values for actions and effect of actions

According to EC5: Part 1-1, the accidental combination rule is used in fire design.

EC5: Part 1-1: 2.3.2.2b

$$\sum \gamma_{GA,j}\, G_{k,j} + A_d + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i} \tag{5}$$

Where G_k are the permanent loads and Q_k the variable loads. γ is a partial safety factor and ψ is a combination value. (A_d is normally equal to zero, but has to be justified in the fire situation).

For fire design the forces and moments ($S_{f,d}$) can be derived from the normal design value (S_d) by the following equation:

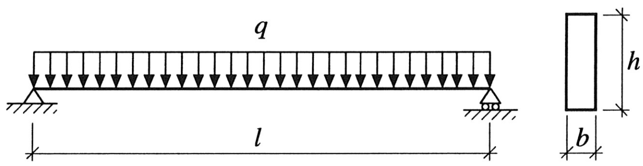
$$S_{f,d} = \eta\, S_d \tag{6}$$

$$\eta = \frac{\sum \gamma_{GA,j} G_{k,j} + A_d + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}}{\sum \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}} \tag{7}$$

η can be calculated by division of the fundamental and the accidental combination rule (case of fire) or η is simplified to a value of [0,6]. In this case conservative results are possible.

Example

System:



l is the span and e the distance between the beams; l = 5,0 *m*; e = 1,20 *m*

Loading:

G_k	= 2,1 kN/m^2	Permanent action
$Q_{k,1}$	= 1,2 kN/m^2	Variable action (snow)
$Q_{k,2}$	= 0,5 kN/m^2	Variable action

Normal design (fundamental combination):

γ is the partial safety factor for actions; γ_G = 1,35 for permanent actions and γ_Q = 1,5 for variable actions

$$1{,}35 \cdot 2{,}1 + 1{,}5 \cdot 1{,}2 + 1{,}5 \cdot 0{,}7 \cdot 0{,}5 = 5{,}16\ kN/m^2$$

$$q_d = 5{,}16 \cdot 1{,}20 = 6{,}19\ kN/m$$

Bending moment:

$$M_d = \frac{q_d\, l^2}{8} = \frac{6{,}19 \cdot 5{,}0^2}{8} = 19{,}3 \ kNm$$

EC5:Part1-1:2.3.2.2b

Fire design (accidental combination):
$\gamma_{G,A}$ is the partial safety factor in the event of fire; $\gamma_{G,A} = 1{,}0$
ψ_i is a combination value; $\psi_{1,1} = 0{,}2$ combination value for the first variable action in the event of fire, $\psi_{2,i} = 0{,}3$ combination value for other variable actions in the event of fire.

$$1{,}0 \cdot 2{,}1 + 0{,}2 \cdot 1{,}2 + 0{,}3 \cdot 0{,}5 = 2{,}49 \ kN/m^2$$

$$q_{f,d} = 2{,}49 \cdot 1{,}20 = 2{,}99 \ kN/m$$

Bending moment:

$$M_{f,d} = \frac{q_{d,f}\, l^2}{8} = \frac{2{,}99 \cdot 5{,}0^2}{8} = 9{,}34 \ kNm$$

$$\eta = \frac{M_{f,d}}{M_d} = \frac{9{,}34}{19{,}3} = 0{,}48 < 0{,}6$$

Calculation methods

Temperature profiles
The temperature for the actual charline is of a magnitude of about 300 °C. The charline derived from β_0 (β) can be put at 200 °C. For a fire exposure of more than 20 minutes ambient temperatures are reached at a distance below the charline which remain constant for the remaining exposure time. This distance is about 30 *mm* from the charline and for the charline related to β_0 (β) about 25 *mm*. The shape of the temperature profile is given in Figure 3.

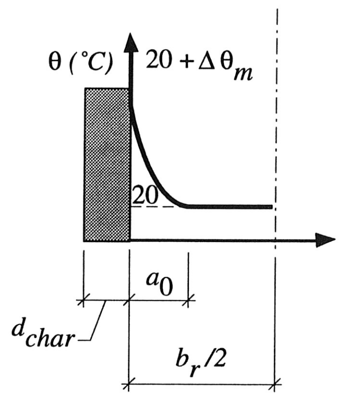


Figure 3 Temperature profile for $b_r > a_0$, see Hartl (1990).

If the width of the residual cross-section b_r is smaller than a_0 for exposure from one side or smaller than $2a_0$ for exposure from two sides, the gradient has to be modified to account for a temperature increase beyond ambient in the middle of the section (see Figure 4).

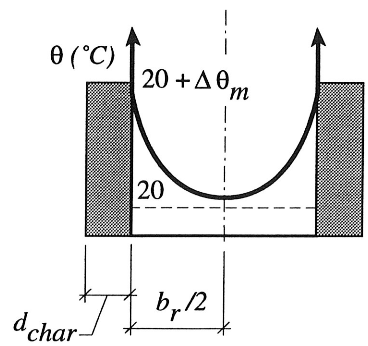


Figure 4 Temperature profile for $b_r < 2\ a_0$, see Hartl (1990).

Temperature dependent strength and stiffness properties

The dependency of the strength and stiffness properties is shown in Figure 5, where E is the modulus of elasticity, f_t, f_m and f_c are the strength values for tension, bending and compression strength of solid timber. From these fundamental findings the following calculation methods are derived.

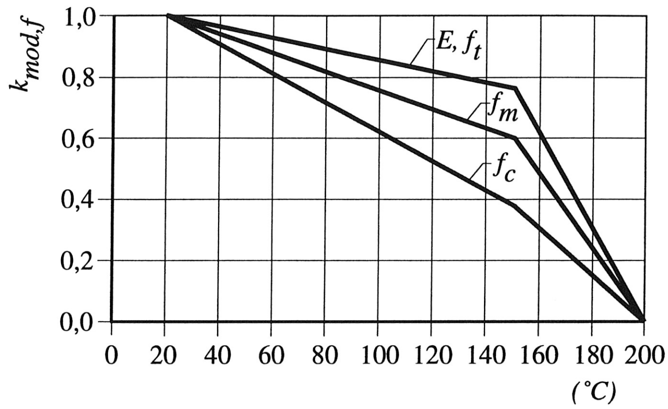


Figure 5 Temperature dependent strength and stiffness properties, see Glos (1990).

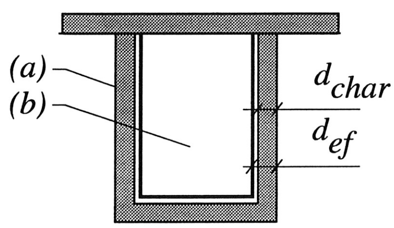
Effective cross section method

For the effective cross section method the time of fire resistance depends on the load bearing capacity of the uncharred remaining cross section. This effective cross section is shown in Figure 6, with

EC5: Part 1-2: 4.1

$$d_{ef} = d_{char} + k_0\ d_0 \tag{8}$$

where $d_0 = 7\ mm$.



EC5: Part1-2: Figure 4.1

Figure 6 Effective cross section.

The factor d_0 is calculated in the following way:
Integration of the temperature profile according to Figure 3 gives an average temperature of about 80 °C. Adopting an average temperature-dependent strength at 80 °C of about 70 % (between bending and compression in Figure 5) implies that 70 % of a_0 may be regarded as unaffected and 30 % as ineffective. 30 % of a_0 = 25 *mm* gives about 7 *mm* (d_0).

Factor k_0 according to the required time of fire resistance is given in Table 1.

Unprotected surfaces	$t_{f,req} < 20$ min.	$k_0 = \frac{t_{f,req}}{20}$
	$t_{f,req} \geq 20$ min.	$k_0 = 1,0$
Surfaces protected by wood based panels	$t_{f,req} - t_{pr} < 20$ min.	$k_0 = \frac{t_{f,req} - t_{pr}}{20}$
	$t_{f,req} - t_{pr} \geq 20$ min.	$k_0 = 1,0$
Surfaces protected by gypsum plasterboards (inner layer)	$t_{f,req} - t_{pr} < 10$ min.	$k_0 = \frac{t_{f,req} - t_{pr}}{10}$
	$t_{f,req} - t_{pr} \geq 10$ min.	$k_0 = 1,0$

$t_{f,req}$ is the required time of fire resistance and t_{pr} the failure time of protective claddings.

EC5: Part 1-2: 4.1

Table 1 Determination of k_0.

Reduced strength and stiffness method
This method is also derived from the afore mentioned temperature profiles (by integration average temperature). The load carrying capacity is calculated for the residual cross-section.

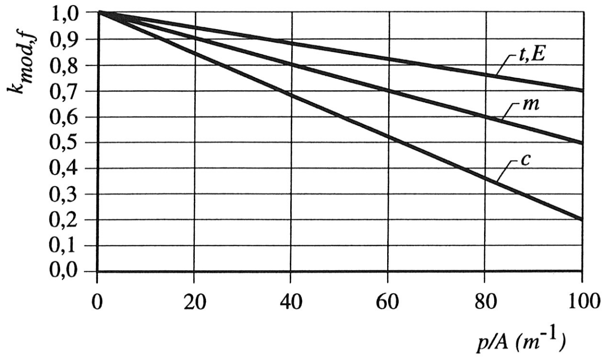
Due to an almost linear relationship between temperature and strength and stiffness properties an equation was found where the reduction factor can be calculated in dependence on the perimeter of the fire exposed cross section (p) and the area of the residual cross section (A_r), see Figure 7.

E.g. $k_{mod,f}$ for standard fire exposure, coniferous timber and for bending strength

EC5: Part 1-2: Annex A(4)

$$k_{mod,f} = 1,0 - \frac{1}{200}\frac{p}{A_r} \tag{9}$$

where
p is the perimeter of the fire exposed residual cross-section in metres
A_r is the area of the residual cross-section in m^2



EC5: Part 1-2: Annex A

Figure 7 $k_{mod,f}$ *for tension (t), bending (m), compression (c) and modules of elasticity (E).*

General calculation methods

For general calculation methods the temperature and moisture content in any point of the cross section is considered. Also the relationship between strength and stiffness properties and temperature and moisture content has to be taken into account. Therefore an increase in the amount of design work is inevitable, but these more complex methods would lead to more economical constructions.

Example:

The following calculation is based on the example given on page 3 for fire exposure from 3 sides and for a fire resistance of R30 and R60 (30 and 60 minutes):

cross-section $b \times h$: 180×220 *mm*

strength class C27 $f_{m,k}$ = 27 *N/mm*^{2}

EC5: Part 1-2: 3.1.1a — β_0 = 0,8 *mm/min* solid timber (see STEP lecture A13)

EC5: Part 1-2: 2.3: P(2) — k_f = 1,25 solid timber

$M_{f,d}$ = 4,77 *kNm* see example page 4

Effective cross section method

EC5: Part 1-2: 4.1(2) — $k_{mod,f}$ = 1,0

R30:

EC5: Part 1-2: 4.2a — $d_{ef} = \beta_o \, t_{f,req} + k_o \, d$ effective charring depth

d_0 = 7 *mm*

k_0 = 1,0 according to Table 1

$d_{ef,30} = 0{,}8 \cdot 30 + 1{,}0 \cdot 7 = 31{,}0 \; mm$

$$W_f = \frac{b_{f,30} \, h_{f,30}^2}{6} = 118 \cdot 189^2/6 = 703 \cdot 10^3 \; mm^3$$ section modulus

EC5: Part 1-2: 2.3a,b

design bending strength

$$f_{m,f,d} = k_{mod,f} \, k_f \, \frac{f_{m,k}}{\gamma_{M,f}} = 1{,}0 \cdot 1{,}25 \cdot \frac{27}{1{,}0} = 33{,}8 \; N/mm^2$$

design bending stress

$$\sigma_{m,f,d} = \frac{M_{f,d}}{W_f} = \frac{9{,}34 \cdot 10^6}{703 \cdot 10^3} = 13{,}3 \; N/mm^2$$

Utilization factor 13,3 / 33,7 = 0,394

R60:
Utilization factor 29,4 / 33,7 = 0,871

Reduced strength and stiffness method

EC5: Part 1-2: 4.1a

$$d_{char} = \beta_0 \, t_{f,req}$$

R30: charring depth
$d_{char,30} = 0{,}8 \cdot 30 = 24{,}0$ mm
$A_r = (180 - 48)\,(220 - 24) = 25{,}9 \cdot 10^3 \; mm^2$

$$W_f = \frac{b_{f,30}\, h_{f,30}^2}{6} = 132 \cdot 196^2/6 = 845 \cdot 10^3 \; mm^3$$ section modulus

EC5: Part 1-2: Annex A(4)

$$k_{\mathrm{mod},f} = 1{,}0 - \frac{1}{200}\frac{p}{A_r}$$

$k_{mod,f} = 1{,}0 - 0{,}005 \cdot 0{,}524 / 0{,}025872 = 0{,}8987$

design bending strength

EC5: Part 1-2: 2.3a,b

$$f_{m,f,d} = k_{\mathrm{mod},f}\, k_f \frac{f_{m,k}}{\gamma_{M,f}} = 0{,}899 \cdot 1{,}25 \cdot 27 = 30{,}3 \;\; N/mm^2$$

design bending stress

$$\sigma_{m,f,d} = \frac{9{,}34 \cdot 10^6}{845 \cdot 10^3} = 11{,}1 \;\; N/mm^2$$

Utilization factor 11,1 / 30,3 = 0,364

R60:
Utilization factor 22,5 / 28,8 = 0,784

Conclusion

It depends on the amount of design work how economical the results of calculating the fire resistance will be. It should also be pointed out that not all the problems related to fire resistance are calculable. The results from fire testing especially for floor and wall design are useful.

The effective cross section method and the reduced strength and stiffnes method are very useful for approximate results for the fire resistance, which might be enough in most cases. They are not adequate if the fire resistance time needs to be very precise or if 2.-order effects are not negligible. If timber members are covered by panels and if they are be included in the calculation, other design procedures should be applied. In this case testing or more detailed calculation is unavoidable (see e.g. STEP lecture E12).

References

Hartl, H. (1990). Brandverhalten von Holzkonstruktionen. Informationsheft.

Glos, P (1990). Festigkeit von Bauholz bei hohen Temperaturen. Forschungsbericht.