Fire resistance of joints

STEP lecture C19
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Objective

To present the methods used for calculating the fire resistance of timber connections.

Prerequisite

A13 Behaviour of timber and wood-based materials in fire

Summary

The relevant chapters from EC5: Part 1-2 "General rules - Supplementary rules for structural fire design" are described.

Calculation methods are given relating to standard fire exposure (standard temperature-time curve) for unprotected joints with side members of wood and with external steel plates as well as for protected joints. Special attention is paid to connections for which an increase of the cross-section is necessary. In one example fire-test results and calculations are compared.

General

The load-bearing capacity of fasteners made of fire-unprotected steel is considerably weakend by heat. All-round protection with wood or wood based materials offers resistance to heat, thereby protecting the steel members. The area of the non-protected surfaces of the steel-members is therefore relevant to the fire-behaviour of fasteners made of steel. Figure 1 describes the relevant yield point Ψ of steel dependent on the temperature ϑ.

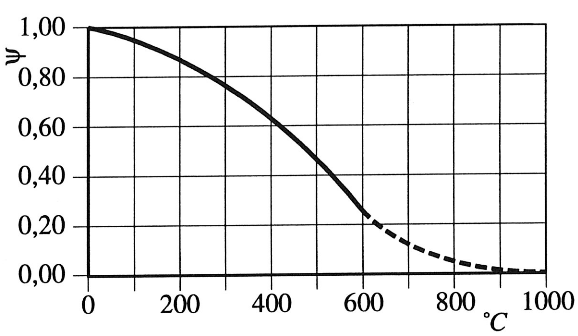

Figure 1 Relevant yield point Ψ of steel in dependence on the temperature ϑ.

EC5: Part 1-2: 4.5.1

The following section relates to joints between members in standard fire exposure situations formed using nails, bolts, dowels, screws, connectors and steel plates. The EC5-rules are valid only for joints under lateral load and deal only with forces which are transmitted symmetrically (see EC5: Part 1-1: Figure 6.2.1, g-k). This restriction in EC5 has in practice often to be replaced by a logical, mathematical derivation of the fire resistance, e.g. in order to determine the fire resistance of timber structures with single connections (see Figure 2).

Unprotected joints with side members of wood

EC5: Part 1-2: 4.5.2

Assuming that joints and their fasteners are designed according to EC5: Part 1-1, they may be expected to display a fire resistance of 15 minutes (R15). Unless otherwise stated all joints should be designed for R15.

For a fire resistance of more than 15 minutes (more than R15) the end and edge distances should be increased by a_f (see EC5: Part 1-2: Figure 4.4) which should be taken as:

EC5: Part 1-2: Equation 4.2

$$a_f = \beta_0 \, (t_{f,req} - 15) \qquad \text{in } mm \tag{1}$$

where

$t_{f,req}$ is the required standard fire resistance in minutes.

β_0 is the charring rate according EC5: Part 1-2: (see STEP lecture A13, Table 1).

The minimum end and edge distances a_3 and a_4 of fasteners should be increased by the extra distance equal to a_f. No extra distance is required if the following condition for a_3 and a_4 is satisfied:

EC5: Part 1-2: Equation 4.7, Equation 4.8

$$\left.\begin{array}{l} a_3 \\ a_4 \end{array}\right\} \geq \beta_0 \, (t_{f,req} + 15) \tag{2}$$

The total thickness t_1 of the side members should satisfy the following conditions:

EC5: Part 1-2: Equation 4.3

$$t_1 \geq \frac{t_{f,req}}{1{,}25 - \eta_n} \qquad \text{in } mm \tag{3}$$

Equation 4.4

$$t_1 \geq 1{,}6 \, t_{f,req} \qquad \text{in } mm \tag{4}$$

Equation 4.5

$$t_1 \geq t_{1,min} + a_f \qquad \text{in } mm \tag{5}$$

where

Equation 4.6

$$\eta_n = \frac{E_d}{R_{d,n}} \tag{6}$$

The fire resistance R30 is satisfied if

$$\eta_n = \frac{E_d}{R_{d,n}} \leq \eta_{30} \tag{7}$$

The requirements of Equation (7) may be satisfied by increasing the number of fasteners in a joint, by choosing fasteners with a higher load-carrying capacity or by using coverings.

For fire resistance between R30 and R60, η may be determined by:

EC5: Part 1-2: Equation 4.9

$$\eta = \eta_{30} \left(\frac{30}{t_{f,req}} \right)^2 \tag{8}$$

Determination of the η_{30}-values

If the conditions in EC5: Part 1-2: Table 4.2, 4.3 are fulfilled, η_{30} could be taken as:

EC5: Part 1-2: Table 4.2

$\eta_{30} = 0{,}80$ for unprotected wood-to-wood joints with nails, non-projecting dowels and connectors with nails.

EC5: Part 1-2: Table 4.3

$\eta_{30} = 1{,}00$ for unprotected steel-to-wood joints with nails or non-projecting dowels.

EC5: Part 1-2: Table 4.2, Table 4.3

$\eta_{30} = 0{,}45$ for unprotected wood-to-wood and steel-to-wood joints with bolts and connectors with bolts.

Otherwise η_{30} has to be calculated according to EC5: Part 1-2: Annex B.

Unprotected joints with steel plates as middle members

30 or 60 minutes fire resistance (R30 or R60) of a joint with unprotected steel plates as middle members (thickness ≥ 2 *mm*) is achieved if the widths b_{st} of the steel plates given in Table 1 are observed.

	unprotected edges in general	unprotected edges on one or two sides
R30	≥ 200 *mm*	≥ 120 *mm*
R60	≥ 440 *mm*	≥ 280 *mm*

EC5: Part 1-2: Annex B5

Table 1 Width b_{st} of steel plates with unprotected edges.

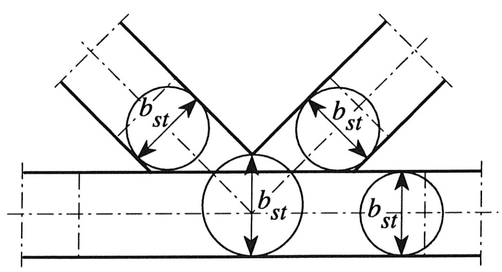

Figure 2 Steel plate joints - definition of b_{sr}

Summarising, the classification of unprotected joints with side members of wood under R30 or R60 calls for a new design of the joint with *n* times the number of actions (Table 2) than at normal temperature design.

$$n = \frac{R_{d,n}}{E_d} = \frac{1}{\eta_{30}} \text{ resp. } \frac{1}{\eta_{60}} \qquad (9)$$

	wood-to-wood joints nails, dowels	steel-to-wood joints nails, dowels	bolts
R30	$\frac{1}{0{,}80} = 1{,}25$	$\frac{1}{1{,}00} = 1{,}00$	$\frac{1}{0{,}45} = 2{,}22$
R60	$\frac{1}{0{,}20} = 5{,}00$	$\frac{1}{0{,}25} = 4{,}00$	$\frac{1}{0{,}1125} = 8{,}89$

Table 2 n times actions needed on joints for R30 and R60 - in general.

Unprotected joints with external steel plates

EC5: Part 1-2: 4.5.3

For unprotected external steel plates which are directly exposed only on one side, the fire resistance R30 is satisfied for a plate thickness of:

$t_1 \geq 6\ mm$ for joints with $\eta \leq 0{,}45$

Otherwise calculation according to EC3, structural fire design, is required.

Example for an unprotected joint with side members of wood

Test member, tensile joint with dowels:

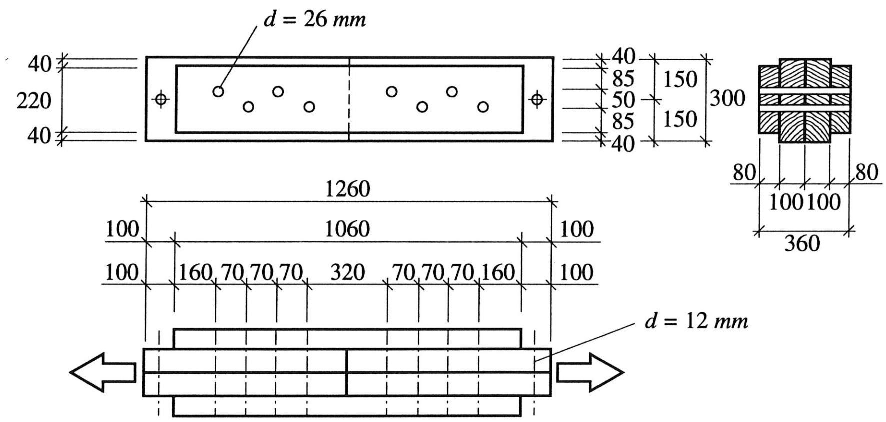

Figure 3 *Tensile joint with side members of wood and dowels in double shear.*

$t_1 = 80\ mm$ $t_2 = 200\ mm$ $d = 26\ mm$

$l_{dow} = 340\ mm$ *(10 mm thick plug on each side)* $t_{1,dow} = 70\ mm$

prEN 338: Table 1

strength class: coniferous timber C 27:

$f_{t,0,k} = 16\ N/mm^2$ $\rho_k = 370\ kg/m^3$

dowel: $f_{u,k} = 360\ N/mm^2$

distances: $a_1 = 140\ mm$ $a_{4,t} = 85\ mm$

$a_{3,t} = 160\ mm$

EC5: Part 1-1: Table 6.6a

minimum distances:

$a_{1,min} = a_{3,t,min} = 7\ d = 7 \cdot 26 = 182\ mm$

$a_{4,t,min} = 3\ d = 3 \cdot 26 = 78\ mm$

Determination of design load-carrying capacity at normal temperature

a) design load-carrying capacity of dowels

Characteristic values:

EC5: Part 1-1: 6.5.1.2b

$$f_{h,1,k} = 0{,}082\ (1 - 0{,}01\ d)\ \rho_k \sqrt{\frac{a_1}{a_{1,\min}}} =$$

$$= 0{,}082 \cdot (1 - 0{,}26) \cdot 370 \cdot \sqrt{\frac{140}{182}} = 19{,}7\ N/mm^2$$

EC5: Part 1-1: 6.5.1.2e

$$M_{y,k} = 0{,}8 f_{u,k} \frac{d^3}{6} = 0{,}8 \cdot 360 \cdot \frac{26^3}{6} = 844\ Nm$$

Design values:

EC5: Part 1-1: 6.2.1l

$$f_{h,1,d} = \frac{k_{mod,1}\, f_{h,1,k}}{\gamma_M} = \frac{0{,}8 \cdot 19{,}7}{1{,}3} = 12{,}1\ N/mm^2$$

EC5: Part 1-1: 6.2.1n

$$M_{y,d} = \frac{M_{y,k}}{\gamma_M} = \frac{844}{1{,}1} = 767\ Nmm$$

EC5: Part 1-1: 6.2.1j

Design load-carrying capacity per shear plane per dowel:

$$R_d = 1{,}1\,\frac{f_{h,1,d}\, t_{1,dow}\, d}{2+\beta}\left(\sqrt{2\beta\,(1+\beta) + \frac{4\,\beta\,(2+\beta)\,M_{y,d}}{f_{h,1,d}\, d\, t_{1,dow}^{\ 2}}} - \beta\right)$$

$$= 1{,}1 \cdot \frac{12{,}1 \cdot 70 \cdot 26}{3}\left(\sqrt{2 \cdot 2 + \frac{4 \cdot 3 \cdot 767 \cdot 10^3}{12{,}1 \cdot 26 \cdot 70^2}} - 1\right) = 17\ 400\ N$$

Design load-carrying capacity of dowels:

$R_{d,dow} = 2 \cdot 4 \cdot 17\ 400 = 139\ 000\ N$

b) design load-carrying capacity of side members:

cross-section area	A	$= 2 \cdot 220 \cdot 80$	$= 35200\ mm^2$
area of dowel holes	A_{dow}	$= 2 \cdot 26 \cdot 80$	$= 4160\ mm^2$
net area	A_{net}	$= 35200 - 4160$	$= 31000\ mm^2$

EC5: Part 1-1: 6.2.1l

$$f_{t,0,d} = \frac{k_{mod,1}\, f_{t,0,k}}{\gamma_M} = \frac{0{,}8 \cdot 16}{1{,}3} = 9{,}85\ N/mm^2$$

$R_{d,sm} = A_{net}\, f_{t,0,d} = 31\ 000 \cdot 9{,}85 = 306\ kN$ ...sm...side member

$E_d = R_{d,dow} = 139\ 000\ N$

EC5: Part 1-2: 4.5.2

Verification of 60 minutes fire resistance according to EC5: Part 1-2

$t_{f,req} = 60\ min$

$\eta_{n,1} = 1{,}0$

(1): $a_f = 0{,}8 \cdot (60 - 15) = 0{,}8 \cdot 45 = 36\ mm$

(2): $a_4 = 85 \geq \beta_0\,(t_{f,req} + 15) = 0{,}8 \cdot (60 + 15) = 60\ mm$

(2): $a_{3,t} = 160 \geq \beta_0\,(t_{f,req} + 15) = 60\ mm$

(5): $t_{1,2} = t_{1,min} + a_f = 80 + 36 = 116\ mm$

$t_{1,2,dow} = 106\ mm$

(4): $t_{1,2} = 116 \geq 1{,}6\ t_{f,req} = 1{,}6 \cdot 60 = 96\ mm$

(3): $$t_{1,2} = 116 \geq \frac{t_{f,req}}{1{,}25 - \eta_{n,2}} = \frac{60}{1{,}25 - 0{,}908} = \frac{60}{0{,}342} = 175\ mm$$

EC5: Part 1-1: 6.2.1j

$$R_d = 1{,}1 \frac{f_{h,1,d}\, t_{1,2,dow}\, d}{2+\beta} \left(\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta) M_{y,d}}{f_{h,1,d}\, d\, t_{1,2,dow}^2}} - \beta \right)$$

$$= 1{,}1 \cdot \frac{12{,}1 \cdot 106 \cdot 26}{3} \left(\sqrt{2 \cdot 2 + \frac{4 \cdot 3 \cdot 767 \cdot 10^3}{12{,}1 \cdot 26 \cdot 106^2}} - 1 \right) = 19\ 200\, N$$

$$R_{d,dow,2} = 2 \cdot 4 \cdot 19\ 200 = 154\ kN$$

(6):

$$\eta_{n,2} = \frac{E_d}{R_{d,dow,2}} = \frac{139}{154} = 0{,}903$$

Determination of η_{30}:

EC5: Part 1-2: Table 4.2

$$l_{dow} = 2\, t_{1,2,dow} + t_2 = 2 \cdot 106 + 200 = 412\, mm \geq 150\, mm$$

$$\frac{t_{max}}{d} = \frac{t_2}{d} = \frac{200}{26} = 7{,}69 \geq 6$$ condition not met

$\Rightarrow$ Determination of η_{30} in accordance with Annex B:

EC5: Part 1-2: Annex B3

a)

$$\eta_{30} \leq \frac{c\ d}{\mu \left(1{,}0 + \left[\frac{110}{l'}\right]^4\right)} = \frac{6{,}0 \cdot 26}{152 \left(1{,}0 + \left[\frac{110}{412}\right]^4\right)} = 1{,}02$$

where

$$\mu = \sqrt{t_2\ t_{1,2}} = \sqrt{200 \cdot 116} = 152$$

$$c = 6{,}0$$

$$l' = l_{dow} = 412\, mm$$

b) $\eta_{30} \leq 1{,}0$

$\eta_{30} = 1{,}0$

Determination of η_{60}:

(8): $$\eta_{60} = \eta_{30} \left[\frac{30}{t_{f,req}}\right]^2 = 1{,}0 \cdot \left[\frac{30}{60}\right]^2 = 0{,}25$$

(7): verification: $\eta_{n,2} \leq \eta_{60}$

$0{,}903 \leq 0{,}25$

This condition is not fulfilled, therefore a new joint has to be designed:

n_2 = 15 dowels in 3 rows

$t_{1,3} = t_{1,3,dow} = 100\ mm$ (without plugs)

EC5: Part 1-1: 6.2.1j

$$R_{d,3} = 1{,}1\,\frac{f_{h,1,d}\,t_{1,3,dow}\,d}{2+\beta}\left(\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,d}}{f_{h,1,d}\,d\,t_{1,3,dow}^{\,2}}}-\beta\right)$$

$$= 1{,}1\cdot\frac{12{,}1\cdot 100\cdot 26}{3}\left(\sqrt{2\cdot 2+\frac{4\cdot 3\cdot 767\cdot 10^3}{12{,}1\cdot 26\cdot 100^2}}-1\right) = 18\ 800\,N$$

$R_{d,dow,3} = 2\cdot 15\cdot 18\ 800 = 564\ kN$

New dimensions of the side members and middle member:

$b_{sm,2} = b_{mm,2} = 2\ \ (95 + 90) = 370\ mm$
$l_{sm,2} = 2\cdot(160 + 4\cdot 140 + 160) = 1760\ mm$

cross-section area	A_2	$= 2\cdot 370\cdot 100$	$= 74\ 000\ mm^2$
area of dowel holes	$A_{dow,2}$	$= 6\cdot 26\cdot 100$	$= 15\ 600\ mm^2$
net area	$A_{net,2}$	$= 74\ 000 - 15\ 600$	$= 58\ 400\ mm^2$

$R_{d,sm,2} = R_{d,mm,2} = A_{net,2}\ f_{t,0,d} = 58\ 400\cdot 9{,}85 = 575\ kN$

verifications: $\eta_{n,3} = \dfrac{E_d}{R_{d,dow,3}} = \dfrac{139}{565} = 0{,}246$

$$t_{1,3} = 100 \geq 96\,mm \geq \frac{t_{f,req}}{1{,}25-\eta_{n,3}} = \frac{60}{1{,}25-0{,}246} = 59{,}8\,mm$$

Fire-test result of the test member: time of fire resistance: 62 minutes
at $F = 94600$ N (test member 6)

Increased timber consumption of the side members only:

$V_{sm,1} = 2\ l_{sm,1}\ b_{sm,1}\ t_1 = 2\cdot 1060\cdot 220\cdot 80\cdot 10^{-9} = 0{,}0373\ m^3$
$V_{sm,2} = 2\ l_{sm,2}\ b_{sm,2}\ t_{1,3} = 2\cdot 1760\cdot 370\cdot 100\cdot 10^{-9} = 0{,}130\ m^3$
$\Delta V_{sm} = 0{,}130 - 0{,}0373 = 0{,}0927\ m^3 \triangleq 249\ \%$

Based on the reduced strength and stiffness method for members (given in EC5: Part 1-2: 4.2 and Annex A) a logical, mathematical derivation of fire resistance could be applied to joints; the reduced cross-section must be checked. Care must be taken to make sure that the steel temperature within the joint does not increase more than 600 °C. This is confirmed not only by tests on dowelled frame corners but also by calculation models. Hence it can be verified that the cross-section does not need to be increased more than the necessary dimensions for the load-carrying capacity for design at normal temperature.

Verification for R60 based on the reduced strength and stiffness method

EC5: Part 1-2: Equation 2.7 $E_{f,d} = \eta_f\ E_d = 0{,}6\cdot 139 = 83{,}4\ kN$
where $\eta_f = 0{,}6$ ...in most cases of civil engineering less than 0,6.

EC5: Part 1-2: Table 3.1 charring rate $\beta_0 = 0{,}8\ mm/min$

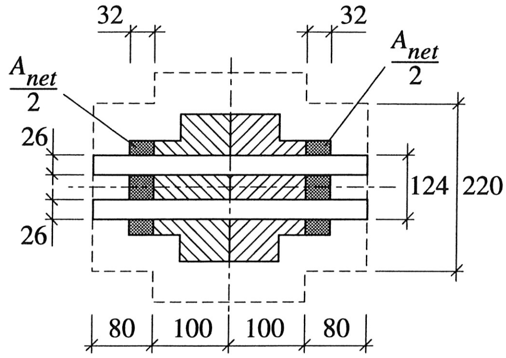

Figure 4

remaining cross-section:
$b_r = 220 - 2 \cdot 0{,}8 \cdot 60 = 124\ mm$
$t_r = 80 - 0{,}8 \cdot 60 = 32\ mm$

cross-section area	A	$= 2 \cdot 124 \cdot 32$	$= 7940\ mm^2$
area of dowel holes	A_{dow}	$= 2 \cdot 26 \cdot 32$	$= 1660\ mm^2$
net area	A_{net}	$= 7940 - 1660$	$= 6280\ mm^2$
perimeter	p_r	$= 2 \cdot (2 \cdot 32 + 124)$	$= 376\ mm$

a) design load-carrying capacity of dowels:

Characteristic value of the embedding strength:
$X_k = f_{h,1,k} = 19{,}7\ N/mm^2$

Reduction factor:

EC5: Part 1-2: Equation A.5

$$k_{mod,f,1} = 1{,}0 - 10^3 \frac{1}{330} \frac{p_r}{A} = 1 - 10^3 \cdot \frac{1}{330} \cdot \frac{376}{7940} = 0{,}856$$

Coefficient for solid timber: $k_f = 1{,}25$
Partial safety factor: $\gamma_{M,f} = 1{,}0$

Design values:

EC5: Part 1-2: Equation 2.1

$$f_{f,1,d} = k_{mod,f,1}\ k_f \frac{X_k}{\gamma_{M,f}} = 0{,}856 \cdot 1{,}25 \cdot \frac{19{,}7}{1{,}0} = 21{,}1\ N/mm^2$$

$M_{f,y,d} = k\ \ M_{y,d} = 0{,}3 \cdot 767000 = 230\ Nm$
assumption: $k = 0{,}3$ (Figure 1: steel temperature: 600 °C)

Design load-carrying capacity per shear plane per dowel:

EC5: Part 1-1: 6.2.1g

$$R_{f,d,1} = f_{f,1,d}\ t_1\ d = 21{,}1 \cdot 32 \cdot 26 = 17500\ N$$

$$R_{f,d,2} = 1{,}1 \frac{f_{f,1,d}\, t_r\, d}{2+\beta} \left(\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{f,y,d}}{f_{f,1,d}\, d\, t_{1,dow}^{2}}} - \beta \right)$$

EC5: Part 1-1: 6.2.1j

$$= 1{,}1 \cdot \frac{21{,}1 \cdot 32 \cdot 26}{3} \left(\sqrt{2 \cdot 2 + \frac{4 \cdot 3 \cdot 230 \cdot 10^3}{21{,}1 \cdot 26 \cdot 32^2}} - 1 \right) = 12{,}8\ kN$$

Design load-carrying capacity of dowels:
$R_{f,d,dow} = 2 \cdot 4 \cdot 12{,}8 = 102\ kN$

verification: $E_{f,d} < R_{f,d,dow,2}$

$83{,}4 < 102$

b) design load-carrying capacity of side members:

verification: $\sigma_{f,d} < f_{f,t,0,d}$

$$\frac{E_{f,d}}{A_{net}} = \frac{83\ 400}{6280} < k_{mod,f,1}\ k_f\ \frac{f_{t,0,k}}{\gamma_{M,f}} = 0{,}856 \cdot 1{,}25 \cdot \frac{16}{1{,}0}$$

$$13{,}3 < 17{,}1$$

Protected joints

EC5: Part 1-2: 4.5.4

Joints are considered protected if the fasteners are covered with protective plugs or wood or wood-based panels with minimum a_f according to EC5: Part 1-2: Figure 4.5 (a) and b) glued-in plugs c) protective panels). For fastening of protective boards the edge distance of fasteners should be at least equal to a_f according to Equation (1).

Concluding summary

- The method given in EC5: Part 1-2, at its present stage is distinctly on the "safe side" and consequentely is an uneconomical design for fire resistances of R30 and even more so for R60.

- One test member, a tensile joint with dowels and side members of timber, was checked according to EC5: Part 1-1. Subsequently this member was calculated using two different methods for a fire resistance of 60 minutes (R60). The first method relates to calculation for joints (Part 1-2), the second to the structural fire design of members. The first method results in extremely oversized members because of the η-value applied. η for R60 is only a quarter of that for R30, which is equal to 1 in the example. Therefore the load-carrying capacity in normal temperature design has to be four times that necessary - four times the number of dowels and tripling of the timber dimensions.

- A comparison of the results show that the second method is in very good agreement with the fire-test results.

- Classification of timber members into fire resistance classes requires that not only the single components they are built from, but also the connections should satisfy the requirements for fire resistance. It is often necessary to direct attention to joints and their fasteners, because the dimensions of the members depend in most of cases, under normal temperature-conditions, on the design of their connections.

Notation

$t_{f,req}$ required standard fire resistance in minutes as contrast to thickness t.

Subscripts

$_{dow}$ dowel.

$_{sm,mm}$ side member, middle member.