Calculation of the Fire Resistance of Steel Constructions
National Research Council of Canada. Division of Building Research

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Title: The calculation of the fire resistance of steel construction (Berechnung des Brandwiderstandes von Stahlkonstructionen)

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PREFACE

This publication describes a unique approach for evaluating the fire hazard to steel constructions and providing adequate protection where required.

The fire load of a compartment is taken as the dominant variable and the structure must be able to resist it in case of an ignition. On this basis the design of a fire-resistant structure proceeds according to well-known principles of heat transfer and structural behaviour.

Using the EMPA standard time-temperature curve, the methods presented rest on the concept of a critical temperature at which an element is no longer capable of performing its intended structural function. Since collapse can occur only when three hinges are present in an element, the critical temperature for compact sections is higher than for noncompact sections. The availability of methods as described enables the engineer to calculate the appropriate protection (if any) for the steel structure he designs.

The translation has been prepared by W. W. Stanzak, Steel Industries Fellow, working in the Fire Research Section of the Division and has been checked by W. R. Schriever, Head of the Building Structures Section, DBR, and by D. A. Sinclair, Chief Translator, National Science Library, National Research Council of Canada.

Ottawa
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THE CALCULATION OF THE FIRE RESISTANCE OF STEEL CONSTRUCTION

1. The Most Important Concepts

Fire resistance

Fire resistance is equal to the fire duration required to heat a steel element to the extent that it can no longer perform its loadbearing function.

Critical temperature

Critical temperature is a function of the steel quality, the structural design and the static loading of the construction element, and means that average cross-sectional temperature at which the element loses its load-bearing ability.

Heating process

The heating process depends on the course of the fire, the shape and cross-section size of the steel and the type of protection present.

Development of the fire

The development of the fire is taken according to the EMPA standard time-temperature curve (Table 1).

2. Scope of the Calculation Method

The available data permit an approximate determination of the fire resistance for theoretically unrestrained elements either unprotected or protected.

During the fire, due to restricted thermal expansions, considerably greater stresses can arise than those considered in the static calculation at room temperature. In assessing the fire resistance this circumstance must be kept in mind, particularly if the structure is of mixed construction (heating-up at different rates). With such constructions the possible rise in compressive stresses should be considered when calculating the critical temperature.

Statically indeterminate structures, in which the development of plastic hinges is possible, can exhibit higher critical temperatures than the theoretically unrestrained elements. While calculating the critical temperatures for such structures the higher fire resistance can be better understood by taking into consideration the static system.

The present calculation fundamentals are based on the currently available research data. At the present time extensive research programmes are being drawn up at the request of the "Europaische Konvention der Stahlbauverbände," which may result in new discoveries.

3. The Fire Resistance of Unprotected Steel Elements

The calculation of the fire resistance of unprotected steel elements requires determination of the critical temperature and the time-temperature function for the steel element.

The fire resistance is defined as the time it takes for the steel element to reach the critical temperature.

3.1. Critical temperature

Nomenclature:

T_{kr}	Critical temperature (°C)
W	Elastic moment of resistance of the steel section (cm^3)
W_{pl}	Plastic moment of resistance of the steel section (cm^3)
F	Cross-sectional area of the steel element (cm^2)
E	Modulus of elasticity of the steel (kg/cm^2)
E_T	Modulus of elasticity of the steel at temperature T (kg/cm^2)
h_1	Depth of web of steel section (cm)
h	Depth of steel section (cm)
d	Web thickness of steel section (cm)
λ	Slenderness ratio
σ_N	Stress in cross-section due to axial force (kg/cm^2)
σ_M	Stress in cross-section due to the bending moment (kg/cm^2)

σ_f Yield stress (kg/cm^2)

$\sigma_{f,T}$ Yield stress at temperature T (kg/cm^2)

σ_R Compressive stresses at extreme fibres of an eccentrically compressed column (kg/cm^2).

In the range of 300-600°C a linear relationship may be assumed respectively between the temperature and the yield stress or the modulus of elasticity of the steel.

$$\sigma_{f,T} = [0{,}690 - 0{,}0011 \cdot (T - 300)] \cdot \sigma_f \tag{1}$$

$$E_T = [0{,}787 - 0{,}00078 \cdot (T - 300)] \cdot E \tag{2}$$

The critical temperature is:

(a) Axially loaded columns with a slenderness ratio of

$$\lambda \leq 4552 \cdot \sqrt{\frac{0{,}240}{\sigma_N} + \frac{0{,}700}{\sigma_f}}$$

$$T_{kr} = 927 - 1136 \cdot \frac{\sigma_N}{\sigma_f} \tag{3}$$

and for larger slenderness ratios:

$$T_{kr} = 1310 - 1602 \, \frac{\sigma_N \cdot \lambda^2}{E \cdot \pi^2} \tag{4}$$

(b) Eccentrically compressed columns:

In equations (3) and (4) the extreme fibre stress is substituted for σ_N.

(c) Tension members:

$$T_{kr} = 927 - 909 \cdot \frac{\sigma_N}{\sigma_f} \tag{5}$$

(d) Flexural elements capable of developing the fully plastic moment:

$$T_{kr} = 927 - 909 \cdot \frac{W \cdot \sigma_M}{W_{pl} \cdot \sigma_f} \tag{6}$$

(e) Flexural elements not capable of attaining the fully plastic moment, but secured against tipping:

$$T_{kr}' = 927 - 909 \cdot \frac{\sigma_M}{\sigma_f} \tag{7}$$

(f) Unbraced flexural elements:

$$T_{kr} = 927 - 1136 \cdot \frac{\sigma_M}{\sigma_f} \qquad (8)$$

(g) For axially and flexurally stressed elements capable of developing the fully plastic moment with a depth of web of $h_1 > \sigma_N \cdot F/\sigma_{f,T} \cdot d$ (neutral axis in web):

$$T_{kr} = 927 - 909 \cdot \frac{A + \sqrt{A^2 + B}}{\sigma_f} \qquad (9)$$

$$\text{where}: \quad A = \frac{\sigma_M \cdot W}{2 \cdot W_{pl}} \quad \text{und} \quad B = \frac{(\sigma_N \cdot F)^2}{4 \cdot d \cdot W_{pl}}$$

For elements with smaller depth of web (neutral axis in flange):

$$T_{kr} = 927 - 909 \frac{\frac{2 \cdot W \cdot \sigma_M}{F \cdot h} + \sigma_N}{\sigma_f} \qquad (10)$$

(h) For flexurally and axially stressed elements that are not capable of developing the fully plastic moment, the critical temperatures shall be calculated using equations (7) and (8), respectively, substituting $\sigma_R = \sigma_M + \sigma_N$ (for σ_M).

The critical temperature must not exceed 600°C. The following critical temperatures may be assumed without calculation:

(a) Columns	T_{kr}	=	400°C
(b) Tension members	T_{kr}	=	350°C
(c) Braced beams	T_{kr}	=	400°C
(d) Unbraced beams	T_{kr}	=	350°C

3.2. Calculation of the heating process

Nomenclature:

ΔT_i Temperature rise in the steel element during the time Δt (°C)

T_a Average fire temperature during the time Δt (°C)

T_i Temperature of the steel element at the beginning of the interval Δt (°C)

U Surface of the steel element exposed to the fire (m^2/m)

G Weight of the steel element (kg/m)

Δt Time interval in hours

The heating of the steel element can be calculated by means of equation (11):

$$\Delta T_i = 231 \cdot \frac{U}{G} \cdot (T_a - T_i) \cdot \Delta t \qquad (11)$$

and for time intervals of 5 minutes:

$$\Delta T_i = 19{,}3 \cdot \frac{U}{G} \cdot (T_a - T_i) \qquad (11\,A)$$

The development of average fire temperatures for time intervals of 5 minutes is given in Table 1.

3.3. Example

Axially loaded column of pipe ϕ 219, 1/20. Effective length 350 cm. Steel St37 with $\sigma_f = 2400$ kg/cm^2.

Section data

$U = 0.688\ m^2/m, \quad F = 125\ cm^2, \quad G = 98.2\ kg/m, \quad i = 7.07\ cm$

Slenderness $\lambda = \frac{350}{7,07} = 50$

Static loading

$P = 120\,000\ kg \quad \sigma_N = \frac{120\,000}{125} = 960\ kg/cm^2$

Critical Temperature

Equation (3) applies, thus:

$$\lambda = 50 < 4552 \cdot \sqrt{\frac{0{,}240}{960} + \frac{0{,}700}{2400}} = 106 > 50$$

$$T_{kr} = 927 - 1136 \frac{960}{2400} = 473\,^\circ C$$

Heating equation (11A)

$$\Delta T_i = 19{,}3 \cdot \frac{0{,}688}{98{,}2} \cdot (T_a - T_i) = 0{,}135 \cdot (T_a - T_i)$$

Calculation of the Heating Curve (development of fire follows Table 1):

Time t Minutes	T_a °C		$(T_a - T_i)$ °C	ΔT_i °C	T_i °C
0	20	280	260	35	20
5	540	620	565	76	55
10	700	727	596	80	131
15	754	773	562	76	211
20	791	806	519	70	287
25	820	832	475	64	357
30	843	853	432	58	421
35	864				479 > T_{kr} = 473

Explanation of the calculation

The average fire temperature during the time from 0 to 5 minutes is T_a = 280°C, the steel temperature at time t = 0 is T_i = 20°C, therefore the difference is $(T_a - T_i)$ = 260°C.

The temperature rise during the time t = 0 to t = 5 is $\Delta T_i = 260 \cdot 0.135 =$ 35°C and the steel temperature after 5 minutes becomes $T_i = 20 + 35 = 55$°C.

This calculation is repeated until the steel temperature reaches the critical temperature. At t = 35 minutes the steel temperature is 479°C and at t = 30 minutes 421°C.

$$\Delta T_i = 58°C/5 \text{ minutes}$$

Fire Resistance

The fire resistance of the column with T_{kr} = 473°C is calculated at:

$$t_w = 30 + \frac{473 - 421}{58} \cdot 5 = \text{approx. } 34 \text{ minutes.}$$

For practical purposes the determination of the fire resistance may also be done with aid of Table 2.

With a given critical temperature T_{kr} and K-value ($K = 19.3 \frac{U}{G}$), the fire resistance of the element can be read from this table.

4. Fire Resistance of Protected Steel Elements

In calculating the fire resistance of protected steel elements the statement of the critical temperature may be omitted.

The fire resistance of protected steel construction consists of two partial resistances:

$$t_w = t_i + t_v \qquad (12)$$

where t_i = partial resistance depending on the insulating value of the protection.
t_v = partial resistance depending on the moisture content of the protection.

4.1. Partial resistance t_i

The partial resistance t_i can be calculated from the following equation:

$$t_i = 5 + 29.\mathrm{ctgh}\,(K) \qquad (13)$$

The function t_i versus K is depicted in Table 3.

Nomenclature:

t_i partial resistance in minutes

K Insulation factor $K = \frac{\Sigma\,(U.k)}{\Sigma\,(G.c)}$

U Development of the inner surface of the protection (m^2/m)

G Weight of the protected elements (kg/m)

c Specific heat of the protected elements (kcal/kg · °C)

k Overall coefficient of heat transfer of the protective covering ($kcal/m^2 \cdot h \cdot °C$)

$$k = \frac{1}{\frac{1}{\alpha} + \frac{d}{\lambda}} \qquad (14)$$

α Coefficient of heat transfer ($kcal/m^2 \cdot h \cdot °C$)

d Thickness of protection (m)

λ Thermal conductivity of the protection (kcal/m · h · °C).

With multi-layer protective coverings, the sum $\Sigma\,(d_i/\lambda_i)$ is substituted for d/λ in equation (14), where d_i and λ_i are related to the corresponding protective layer. For protected steel constructions the coefficient of heat transfer shall be taken as at least $\alpha = 7\ kcal/m^2 \cdot h \cdot °C$. In doing this the values for thermal conductivity shall not be assumed to be more favourable than those shown in Table 4.

For hollow-core building elements (light protection) the heat sink $\Sigma(G.c)$ shall be calculated only for the steel section.

4.2. Partial resistance t_v

Partial resistance t_v can be determined by means of equation (15):

$$t_v = \frac{36 \cdot p \cdot d \cdot \gamma}{k_v} \tag{15}$$

Nomenclature:

- p Water content as a percentage of the density of the protection
- d Thickness of the protection (m)
- γ Density of the protection (kg/m^3)
- k_v Overall coefficient of heat transfer related to the middle of the insulation thickness ($kcal/m^2 \cdot h \cdot °C$)

$$k_v = \frac{1}{\frac{1}{\alpha} + \frac{0{,}5 \cdot d}{\lambda}} \tag{16}$$

Should the protection consist of several insulating materials with various p and λ values, the partial resistance t_v must be calculated separately for each layer.

For the first layer:

$$k_{v.1} = \frac{1}{\frac{1}{\alpha} + \frac{d_1}{2 \cdot \lambda_1}},$$

$$t_{v.1} = \frac{36 \cdot p_1 \cdot d_1 \cdot \gamma_1}{k_{v.1}}$$

For the second layer:

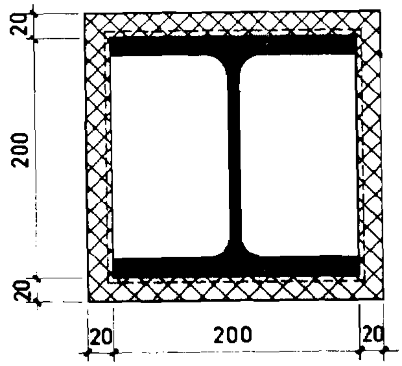
$$k_{v.2} = \frac{1}{\frac{1}{\alpha} + \frac{d_1}{\lambda_1} + \frac{d_2}{2 \cdot \lambda_2}},$$

$$t_{v.2} = \frac{36 \cdot p_2 \cdot d_2 \cdot \gamma_2}{k_{v.2}}$$

etc. The partial resistance t_v then becomes: $t_v = t_v = \Sigma(t_{v,i})$.

4.3. Examples

4.3.1. Light protection of sprayed asbestos



Column: HE 200B, G = 64.9 kg/m

c = 0.13 kcal/kg · °C

Box-shaped protection of 2 cm sprayed asbestos on metal lath.

Inner development of the protection:

$U = 4 \cdot 0.20 = 0.80\ m^2/m$

Overall coefficient of heat transfer (Table 4): k = 3.94

Insulation factor $K = \dfrac{0.80 \cdot 3.94}{64.9 \cdot 0.13} = 0.374$

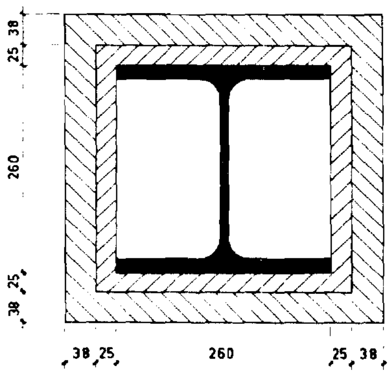
Partial resistance t_i from equation (13):

$$t_i = 5 + 29 \text{ ctgh } (0.374) = 86 \text{ minutes}$$

Partial resistance t_v (from Table 4) 2 minutes

Total Fire Resistance $t_w = t_i + t_v =$ 88 minutes

4.3.2. Light, multi-layer protection



Column: HE 260 B, G = 93 kg/m

c = 0.13 kcal/kg · °C

Box protection of 2.5 cm gypsum board and 3.8 cm vermiculite plaster.

Inner development of the protection:

$U = 4 \cdot 0.26 = 1.04\ m^2/m$

From Table 4:

For gypsum boards	$\lambda = 0.5$	p = 20%	$\gamma = 800$
For vermiculite plaster	$\lambda = 0.16$	= 20%	$\gamma = 640$

Overall coefficient of heat transfer:

$$k = \frac{1}{\frac{1}{7} + \frac{0{,}038}{0{,}16} + \frac{0{,}025}{0{,}50}} = \frac{1}{0{,}143 + 0{,}238 + 0{,}050} = 2{,}320$$

$$k_{v,1} = \frac{1}{0{,}143 + 0{,}5 \cdot 0{,}238} = 3{,}816 \text{ (for vermiculite plaster)}$$

$$k_{v,2} = \frac{1}{0{,}143 + 0{,}238 + 0{,}5 \cdot 0{,}050} = 2{,}463 \text{ (for gypsum board)}$$

Partial resistances:

Insulation factor $K = \frac{1.04 \times 2.32}{93 \times 0.13} = 0.200$

Partial resistance $t_i = 5 + 29 \cdot \mathrm{ctgh}\,(0.200) = 152$ minutes

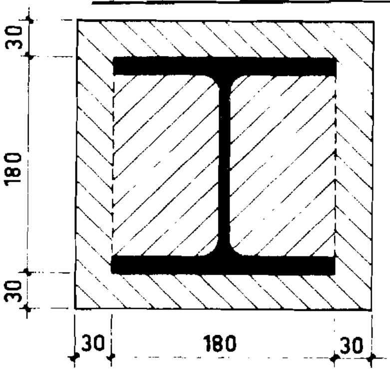
Partial resistance t_v :

$$t_{v,1} = \frac{36 \cdot 0{,}20 \cdot 0{,}038 \cdot 640}{3{,}816} = 46 \quad \text{minutes (vermiculite plaster)}$$

$$t_{v,2} = \frac{36 \cdot 0{,}20 \cdot 0{,}025 \cdot 800}{2{,}463} = 58 \quad \text{minutes (gypsum board)}$$

Fire resistance $t_w = 152 + 48 + 58 =$ <u>256 minutes</u>

4.3.3. Massive protection with core of gravel-aggregate concrete



Column: HE 180 B, G = 51.2 kg/m

$F = 65.3\ \mathrm{cm}^2 \quad c = 0.13$ kcal/kg · °C

Protection: column provided with 3 cm concrete cover

Inner development of the protection

$U = 4 \times 0.18 = 0.72\ \mathrm{m}^2/\mathrm{m}$

From Table 4, for gravel concrete: $\lambda = 1.2$, $p = 5\%$. $\gamma = 1800\ \mathrm{kg/m}^3 \quad c = 0.20$

Overall coefficients of heat transfer:

$$k = \frac{1}{0{,}143 + \frac{0{,}03}{1{,}2}} = \frac{1}{0{,}143 + 0{,}025} = 5{,}952$$

$$k_v = \frac{1}{0{,}143 + 0{,}5 \cdot 0{,}025} = 6{,}431$$

Core weight (without steel): $G_k = (0.18 \times 0.18 - 0.00653) \times 1800 = 46.6$ kg/m

Heat sink of concrete core and steel cross-section:

$$\Sigma G_i \cdot c_i = 51.2 \times 0.13 + 46.6 \times 0.20 = 15.976 \text{ kcal/°C}$$

Insulation factor $K = \frac{0.72 \cdot 5.952}{15.976} = 0.268$

Partial resistance $t_i = 5 + 29 \cdot \text{ctgh}\,(0.268) = 115$ minutes

Partial resistance $t_v = \frac{36 \cdot 0.03 \cdot 0.05 \cdot 1800}{6.431} = \underline{15}$ minutes

Total fire resistance $t_w = 115 + 15 = \underline{130}$ minutes

5. Fire Resistance of Partially Protected Steel Elements

The calculation of the fire resistance of partially protected steel construction involves the calculation of the critical temperature and the heating curve of the steel element.

Fire resistance is the time it takes for steel temperature to reach the critical temperatures.

5.1. Critical temperature

The critical temperature shall be determined in the same way as for unprotected steel constructions (3.1).

5.2. Heating curve

The heating of the steel element can be calculated by means of equation (17):

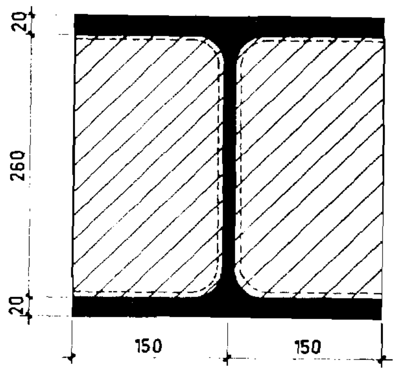
$$\Delta T_i = \frac{30 \cdot U_s + k \cdot U_v}{0{,}13 \cdot G} \cdot (T_a - T_i) \cdot \Delta t \qquad (17)$$

Nomenclature:

T_a, T_i, ΔT_i, Δt, G (as in Section 3.2.)
U_s Unprotected steel surface (m^2/m)
U_v Steel surface covered by protection (m^2/m)
k As in Section 4.1. Equation (14)

5.3. Example

Axially loaded column, concrete filled between flanges



Steel cross-section HE 300B

Load P = 140, 000 kg

Effective Length 540 cm

Steel St37 with σ_f = 2400 kg/cm^2

Section Data

F = 149 cm^2, G = 117 kg/m, i = 7.58 cm

ΣU = 1.732 m^2/m

Unprotected steel surface: $U_s = 2 \cdot 0.30 + 4 \cdot 0.02 = 0.68$ m^2/m

Protected steel surface: $U_v = 1.732 - 0.68 = 1.052$ m^2/m

Critical temperature

$$\sigma = \frac{140,000}{149} = 940 \text{ kg/cm}^2$$

Slenderness ratio $\frac{540}{7.58} = 71$; equation (3) is applicable

$$T_{kr} = 927 - 1136 \cdot \frac{940}{2400} = \underline{482\,°C}$$

Heating equation after equation (17)

The overall coefficient of heat transfer is calculated with respect to the average thickness of protection (from Table 4): λ = 1.2 kcal/m.h.°C

$$k = \frac{1}{0{,}143 + \frac{0{,}075}{1{,}2}} = 4{,}87 \text{ kcal/m}^2 \cdot \text{h} \cdot °\text{C}$$

Time interval Δt = 5 minutes = $\frac{1}{12}$ hour

$$T_i = \frac{30 \cdot 0{,}68 + 4{,}87 \cdot 1{,}052}{117 \cdot 0{,}13} \cdot (T_a - T_i) \cdot \frac{1}{12}$$

$$\underline{T_i = 0{,}140 \cdot (T_a - T_i)}$$

Calculation of the heating curve (explanation as in example 3.3)

Time t Minutes	T_a °C		$(T_a - T_i)$ °C	ΔT_i °C	T_i °C
0	20				20
		280	260	36	
5	540				56
		620	564	79	
10	700				135
		727	592	83	
15	754				218
		773	555	78	
20	791				296
		806	510	71	
25	820				367
		832	465	65	
30	843				432
		853	421	58	
35	864				490 > T_{kr} = 482

Fire resistance of the column: $t_w = 30 + \frac{482 - 432}{58} \cdot 5 = \underline{34 \text{ minutes}}$

As with unprotected steel elements, the determination of the fire resistance may be done with the aid of Table 2.

6. Consideration of the Required Fire Resistance in Structural Calculations

6.1. Steel temperatures and critical stresses

In practice the design engineer must first answer the question: Do the cross-sections selected to satisfy structural requirements provide the necessary fire resistance? Steel temperatures attained after a fire duration of 30, 60 and 90 minutes are assembled in Tables 5 and 6 for selected unprotected and protected steel sections. The critical loads for axially loaded columns of St37 can be read off simultaneously. For other steel elements the critical stresses are to be calculated with the following equations:

(a) Eccentrically loaded columns or elements subjected to flexural and compressive loads, which are not capable of developing the fully plastic moment:

$$\sigma_{R,kr} = \frac{(927 - T) \cdot \sigma_f}{1136} \tag{18}$$

(b) Flexural elements capable of developing the fully plastic moment:

$$\sigma_{M,kr} = \frac{(927 - T) \cdot \sigma_f}{909} \cdot \frac{W_{pl}}{W} \tag{19}$$

(c) Flexural elements, which are not capable of developing the fully plastic moment, but are secured against tipping:

$$\sigma_{M,kr} = \frac{(927 - T) \cdot \sigma_f}{909} \tag{20}$$

(d) Elements subject to flexural and compressive force which are capable of developing the fully plastic moment and have a depth of web of $h_1 \geq \sigma_N \cdot F/\sigma_{f,T} \cdot d$ (neutral axis in web):

$$\sigma_{M,kr} = -C + \sqrt{C^2 + D} \tag{21}$$

$$C = \frac{2 \cdot W \cdot d \cdot \sigma_{f,T}}{(\alpha \cdot F)^2}, \qquad D = \frac{4 \cdot W_{pl} \cdot d\,(\sigma_{f,T})^2}{(\alpha \cdot F)^2}$$

For elements with smaller web depths (neutral axis in web):

$$\sigma_{M,kr} = \frac{(927 - T) \cdot \sigma_f}{909} \cdot \frac{1}{\frac{2 \cdot W}{F \cdot h} + \alpha} \tag{22}$$

In these equations:

T Steel temperatures read from Tables 5 and 6 (°C)

α Relationship between compressive and flexural stresses

$$\alpha = \sigma_N/\sigma_M$$

All other symbols according to Section 3.1.

The critical stresses thus calculated for theoretically unrestrained elements provide the maximum permissible loads for the required fire resistance, which must be greater than the design loads.

If restraints to thermal expansion are expected in the structure, this effect must be taken into consideration. For columns in steel frame structures the effect may be assumed without calculation, as additional compressive stress in the order of

$$\sigma_{N,T} = 0.89\ T\ \mathrm{kg/cm^2}$$

Normally the effect of thermal expansions in beams in steel framed structures can be neglected.

With mixed building systems (for example, steel columns and reinforced concrete walls) the effect of the restricted thermal expansion shall be calculated on the basis of the stiffness of the structural system.

6.2. Examples

6.2.1. Axially loaded interior column in a steel framed structure

Cross-section: HEB 260, with concrete between the flanges steel St37 with $\sigma_f = 2400\ kg/cm^2$, $F = 118.4\ cm^2$

Required fire resistance: FR = 30 minutes

From Table 5:

Steel temperature attained after 30 minutes T = 460°C

Critical load for the column $P_{kr} = 118.9\ t$

Effect of the restricted thermal expansion:

$$\sigma_{N,T} = 0.89 \cdot 460 = 409\ kg/cm^2,\ \text{i.e.}\ P_T = F\sigma_{N,T} = 48.4\ t$$

Allowable load: $P_{zul} = 118.9 - 48.4 = 70.5\ t$

6.2.2. Floor beam in a steel framed structure

Cross-section: I PE 300 with concrete between the flanges Steel ST37 with $\sigma_f = 2400\ kg/cm^2$. The beam is able to develop the full plastic moment: $W_{pl}/W = 1.127$

From Table 6: T = 452°C (FR = 30 minutes)

Critical flexural stresses (equation 19):

$$\sigma_{M,kr} = \frac{(927 - 452) \cdot 2400}{909} \cdot 1{,}127 = 1543\ kg/cm^2$$

Allowable moment: $M_{zul} = 1543\ W = 8.59\ mt$

6.2.3. Element subject to flexural and compressive force in a steel framed structure

Cross-section: HEB 400 with concrete between the flanges, shielded from one side. Steel St37. Cross-section is able to develop the fully plastic moment. Required fire resistance FR = 60 minutes.

$\sigma_N/\sigma_M = \alpha = 0{,}2$ ($\sigma_N = 200\ kg/cm^2$)
$F = 197{,}8\ cm^2$ $W = 2880\ cm^3$ $W_{pl} = 3240\ cm^3$ $d = 1{,}35\ cm$
$h_1 = 35{,}2\ cm$

From Table 6: T = 473°C. Yield stress at this temperature (equation 1):

$$\sigma_{f,T} = [0{,}69 - 0{,}0011 \cdot (T - 300)] \cdot \sigma_f = 1200 \text{ kg/cm}^2$$

$$\frac{\sigma_N \cdot F}{\sigma_{f,T} \cdot d} = \frac{200 \cdot 197{,}8}{1200 \cdot 1{,}35} = 24{,}4 \text{ cm} < 35{,}2 \text{ cm}$$

Equation (21) applies:

$$C = \frac{2 \cdot 2880 \cdot 1{,}35 \cdot 1200}{(0{,}2 \cdot 197{,}8)^2} = 5962 \quad D = \frac{4 \cdot 3240 \cdot 1{,}35 \cdot 1200^2}{(0{,}2 \cdot 197{,}8)^2} = 16{,}1 \cdot 10^6$$

$$\sigma_{M,kr} = -5962 + \sqrt{5962^2 + 16{,}1 \cdot 10^6} = 1224 \text{ kg/cm}^2$$

Allowable moment for normal force P = 1224 · 0.2 · 197.8 = 48.4 t:

$$M_{zul} = 1224 \cdot W = 1224 \cdot 2880 = 34.2 \text{ mt}$$

If a resistance to thermal expansion must be considered for this element (for example an exterior column), a rise in the normal force of $\sigma_{f,T} = 473 \cdot 0.89 = 420 \text{ kg/cm}^2$ must be included in the calculation.

$$\Sigma(\sigma_N) = 200 + 420 = 620 \text{ kg/cm}^2, \text{ i.e. } \Sigma P = 122.6 \text{ t}$$

1. Trial: $\sigma_{M,kr} = 900 \text{ kg/cm}^2$, $\alpha = \frac{620}{900} = \text{ca. } 0{,}70$

$$\sigma_N \cdot F/\sigma_{f,T} \cdot d = 620 \cdot 197{,}8/1200 \cdot 1{,}35 = 100 \text{ cm} > 35{,}2 \text{ cm}$$

Equation (22) applies:

$$\sigma_{M,kr} = \frac{(927 - 473) \cdot 2400}{909} \cdot \frac{1}{\frac{2 \cdot 2880}{197{,}8 \cdot 40} + 0{,}70} = 840 \text{ kg/cm}^2 < 900 \text{ kg/cm}^2$$

2. Trial: $\sigma_{M,kr} = 800 \text{ kg/cm}^2$, $\alpha = \frac{620}{800} = 0{,}775$

$$\sigma_{M,kr} = \frac{(927 - 473) \cdot 2400}{909} \cdot \frac{1}{\frac{2 \cdot 2880}{197{,}8 \cdot 40} + 0{,}775} = 798 = \text{ca. } 800 \text{ kg/cm}^2$$

Allowable moment for normal force P = 0.775 · 800 · 197.8 = 122.6 t is

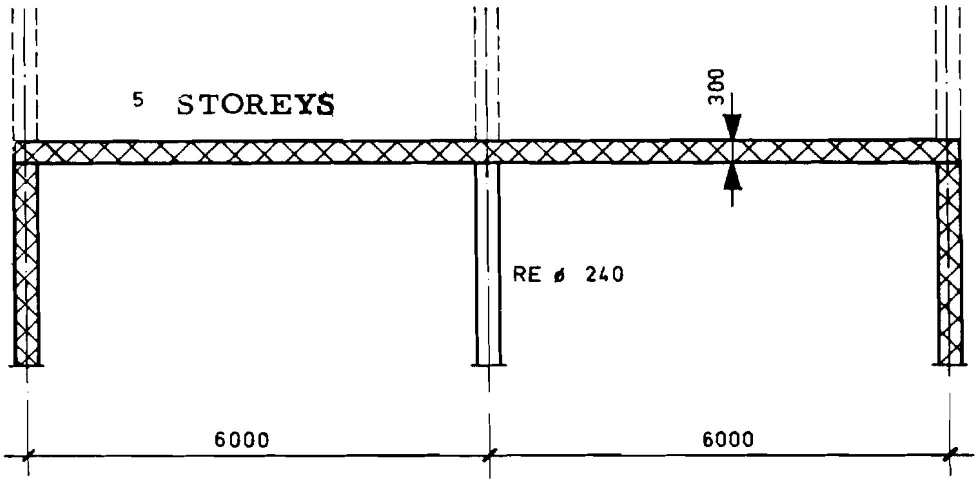
$$M_{zul} = 800 \times 2800 = 23.0 \text{ mt}$$

6.2.4. <u>Axially loaded interior column in mixed construction</u>

Cross-section of column: RE ϕ 240, F = 452 cm^2, $\sigma_f = 2200 \text{ kg/cm}^2$

Required fire resistance: FR = 60 minutes

From Table 5: $T = 350°C$ $P_{kr} = 511.0$ t



Thermal expansion of the column under the unfavourable assumption that the reinforced concrete walls will not exhibit any temperature rise:

$$f = \Delta T \cdot \alpha_t \cdot L = (350 - 20) \times 0.00001 \times 300 = 0.99 \text{ cm.}$$

This expansion may be restrained by the rigidity of the five 30 cm thick reinforced concrete floors above.

For one floor (column separation in long direction 6.0 m):

$$I_b = \frac{6 \cdot 0{,}3^3}{12} = 0{,}0135 \text{ m}^4 \qquad E_b = 3000000 \text{ t/m}^2$$

$$E_b \cdot I_b = 40500 \text{ tm}^2$$

Force required (span $2 \cdot 6.0 = 12$ m) to permit thermal expansion

$$P_T = \frac{48\ EIf}{L^3} = \frac{48 \times 40,500 \times 0.0099}{12^3} = 11.2 \text{ t}$$

For 5 floors $\Sigma P_T = 56.0$ t

Allowable load on the column:

$$P_{zul} = 511 - 56 = 455.0 \text{ t}$$

Table 1. Calculation of the heating curve; course of fire follows EMPA standard time-temperature curve

$$\Delta T_i = 19{,}3 \, \frac{U}{G} \, (T_a - T_i)$$

Time t. Min.	T_a		$(T_a - T_i)$ °C	ΔT_i	T_i	Time t. Min.
0	20		260		20	0
		280				
5	540					5
		620				
10	700					10
		727				
15	754					15
		773				
20	791					20
		806				
25	820					25
		832				
30	843					30
		853				
35	864					35
		873				
40	881					40
		888				
45	896					45
		902				
50	910					50
		916				
55	922					55
		927				
60	932					60
		938				
65	943					65
		948				
70	953					70
		958				
75	962					75
		966				
80	970					80
		974				
85	978					85
		982				
90	985					90
		989				
95	992					95
		995				
100	999					100
		1002				
105	1005					105
		1008				
110	1011					110
		1014				
115	1017					115
		1020				
120	1022					120
		1025				
125	1028					125
		1031				
130	1033					130
		1036				
135	1038					135
		1040				
140	1042					140
		1045				
145	1047					145
		1049				
150	1051					150
		1053				
155	1055					155
		1058				
160	1060					160
		1062				
165	1064					165
		1066				
170	1067					170
		1069				
175	1071					175
		1073				
180	1075					180

Fire resistance $t_i = t_n + \frac{(T_{kr} - T_{i,n}) \cdot 5}{\Delta T_{i,n}}$ Min.

Table 2. Fire resistance of unprotected steel elements of St37

K_1 = 0,135, T_{kr} = 473 °C, t_w > 30 Min.

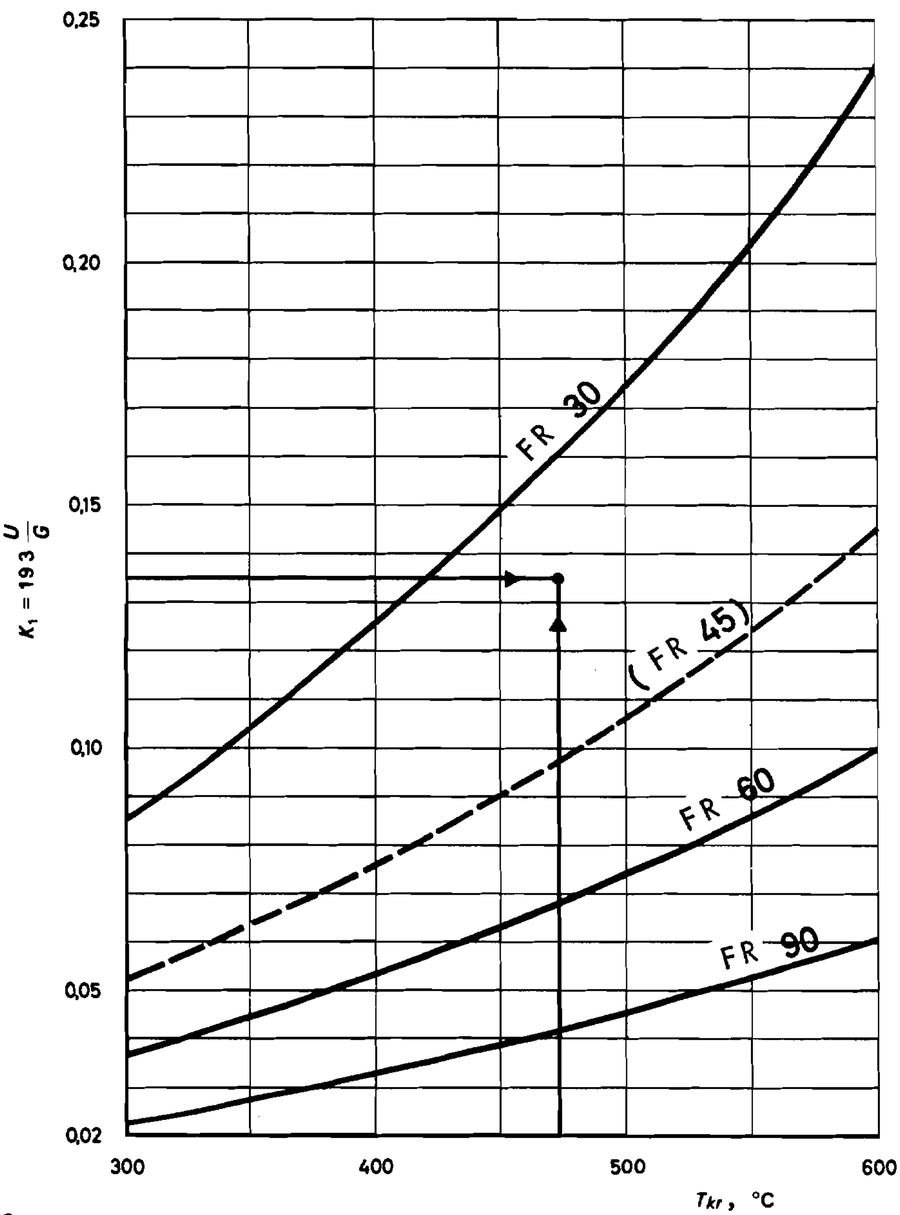


Table 3. Fire resistance of protected steel elements

Determination of partial resistance t_i

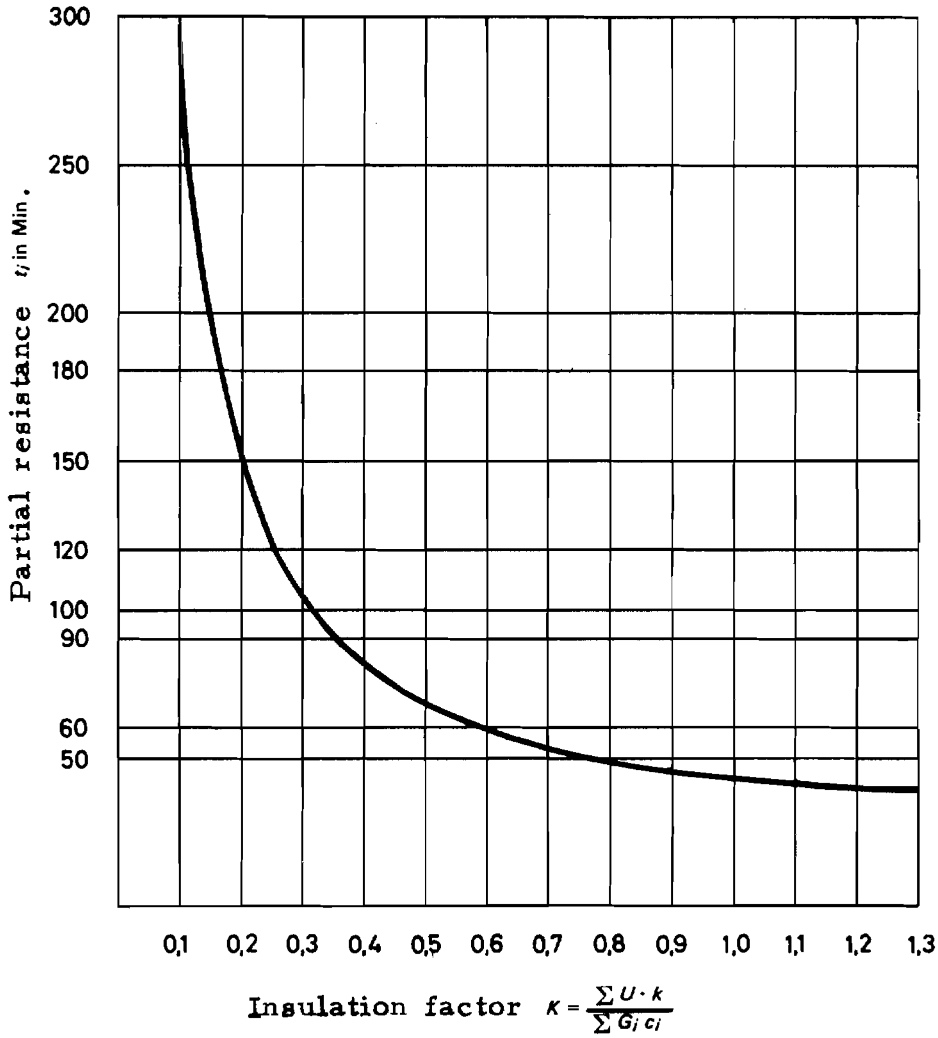


Table 4. Overall coefficients of heat transfer and vaporization times

Nomenclature:

- d thickness of protection (m)
- λ thermal conductivity (kcal/m · h · °C)
- k overall coefficient of heat transfer (kcal/m^2 · h · °C)
- k $\frac{1}{1/\alpha + d/\lambda}$
- α coefficient of heat transfer (kcal/m^2 · h · °C), for protected steel elements $\alpha = 7$
- p moisture content as a percentage of density
- γ density of the protection (kg/m^3)
- t_v vaporization time in minutes
- c_s specific heat of steel (0.13 kcal/kg · °C)
- c_i specific heat of other materials (0.20 kcal/kg · °C).

Protection	d	λ	k	p	γ	t_v
Sprayed asbestos	0,015	0,18	4,42	5	300	1
	0,020		3,94			2
	0,025		3,55			3
	0,030		3,23			4
	0,035		2,96			5
Gravel concrete	0,020	1,20	6,25	5	1800	10
	0,040		5,68			21
	0,060		5,18			33
	0,080		4,76			46
Asbestos-cement boards	0,010	0,40	5,95	—	1000	—
	0,015		5,56			
	0,020		5,18			
	0,025		4,85			
Gypsum boards	0,020	0,50	5,47	20	800	19
	0,030		4,93			30
	0,040		4,49			42
	0,050		4,12			56
Gypsum-sand plaster	0,020	0,56	5,60	8	1200	11
	0,030		5,09			18
	0,040		4,49			42
	0,050		4,12			56
Gypsum-lime plaster	0,020	0,60	5,68	8	1700	16
	0,030		5,19			25
	0,040		4,77			35
Cement-lime plaster	0,020	0,75	5,90	5	1900	11
	0,030		5,47			17
	0,040		5,10			23

Table 4 (Continued)

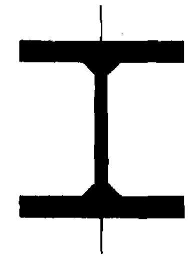
Protection	d	λ	k	p	γ	t_v
Light-aggregate concrete	0,030 0,040 0,050	0,70	5,38 5,00 4,67	5	1500	13 19 24
Hollow tiles	0,040 0,060 0,080 0,100	0,40	4,12 3,41 2,92 2,54	—	1000	—
Solid tiles	0,060 0,120	0,70	4,37 3,18	—	1800	—
Vermiculite-or-perlite-cement plaster	0,020 0,030 0,040 0,050	0,18	3,98 3,27 2,78 2,38	20	550	15 26 40 55
Vermiculite-or-perlite-gypsum plaster	0,020 0,030 0,040 0,050	0,16	3,73 3,03 2,55 2,19	20	640	19 33 49 69
Vermitecta plates	0,020 0,025 0,030 0,040	0,18	3,98 3,59 3,27 2,78	5	550	4 5 6 10
Cement stone	0,020 0,040 0,060	0,35	5,00 3,89 3,18	8	800	8 18 32

Table 5. Fire resistance of steel elements exposed on all sides (interior columns)

FR Fire resistance in minutes

T Steel temperature

P_{kr} Critical load for axially compressed columns (t) of St37



Open wide-flange profiles

Profile		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	*T*	P_{kr}	*T*	P_{kr}
HEM 180		596	80,0				
HEM 200		580	97,4				
HEM 220		569	114,5				
HEM 240		507	180,4				
HEM 260		503	200,6				
HEM 280		496	223,1				
HEM 300		451	310,6				
HEM 320		447	322,3				

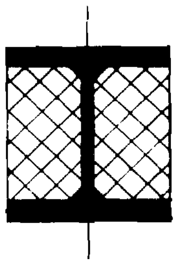
Box-shaped closed wide-flange profiles

Profile	*t*	FR 30 Min.		FR 60 Min.		FR 90 Min.	
	(mm)	*T*	P_{kr}	*T*	P_{kr}	*T*	P_{kr}
HEA 200	10	570	67,2				
HEA 220	12	531	93,3				
HEA 240	12	514	112,3				
HEA 260	12	505	127,9				
HEA 280	15	469	168,2				
HEA 300	15	455	194,0				
HEA 320	15	440	218,1				
HEA 340	15	429	238,2				
HEA 360	20	384	312,8				
HEA 400	20	372	413,9				

Table 5. (continued)

Box-shaped closed wide-flange profiles (continuation)

Profile	t	FR 30 Min.		FR 60 Min.		FR 90 Min.	
	(mm)	T	P_{kr}	T	P_{kr}	T	P_{kr}
HEB 200	15	457	130,7				
HEB 220	15	445	152,8				
HEB 240	18	412	199,4				
HEB 260	18	406	223,3				
HEB 280	18	399	248,2				
HEB 300	20	378	298,8				
HEB 320	20	367	327,2				
HEB 340	20	345	360,5				
HEB 360	20	353	376,6				
HEB 400	20	342	423,1				
HEM 160	20	370	179,8				
HEM 180	20	361	211,1				
HEM 200	25	330	275,7	589	156.3		
HEM 220	25	323	313,7	579	181,1		
HEM 240	30	282	517,1	520	283,0		
HEM 260	30	278	567,3	514	315,0		
HEM 280	30	274	618,5	509	348,2		
HEM 300	40	234	820,3	446	531,2		
HEM 320	40	231	856,3	441	559,6		
HEM 340	40	230	885,4	439	581,5		
HEM 360	40	229	913,2	437	602,2		
HEM 400	40	227	971,8	433	645,3	596	429,0



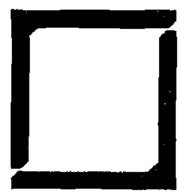
Wide-flange profiles with concrete between the flanges

Profile		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	T	P_{kr}	T	P_{kr}
HEA 220		596	45,4				
HEA 240		566	59,5				
HEA 260		550	70,3				
HEA 280		536	81,9				
HEA 300		510	101,0				
HEA 320		483	118,9				
HEA 340		464	133,0				
HEA 360		448	147,2				
HEA 400		423	172,1				

Table 5. (continued)

Wide-flange profiles with concrete between the flanges (continuation)

Profile		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	T	P_{kr}	T	P_{kr}
HEB 200		516	69,1				
HEB 220		495	84,6				
HEB 240		472	103,7				
HEB 260		460	118,9				
HEB 280		450	134,9				
HEB 300		431	159,1				
HEB 320		410	179,0				
HEB 340		396	194,6				
HEB 360		385	209,8				
HEB 400		366	237,5				
HEM 160		417	106,4				
HEM 180		402	127,7				
HEM 200		385	152,6				
HEM 220		372	177,5				
HEM 240		324	257,0	580	148,2		
HEM 260		317	285,7	571	167,5		
HEM 280		309	316,3	560	189,2		
HEM 300		276	484,9	512	270,7		
HEM 320		272	499,2	505	283,6		
HEM 340		270	505,2	503	288,5		
HEM 360		269	510,0	501	292,2		
HEM 400		266	521,2	497	301,7		



Box-shaped welded angle profiles

Profile		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	T	P_{kr}	T	P_{kr}
2 L 100.16		484	56,5				
2 L 100.20		422	78,5				
2 L 130.16		476	76,3				
2 L 150.16		474	89,1				
2 L 150.18		440	106,8				
2 L 150.20		412	124,6				
2 L 180.16		471	108,8				
2 L 180.18		437	130,4				
2 L 180.20		408	152,5				
2 L 200.20		407	170,7				
2 L 200.24		360	220,1				
2 L 200.28		323	270,6	579	156,3		
2 L 120.80.14		523	45,6				
2 L 150.100.14		517	58,6				
2 L 160.80.14		517	56,0				
2 L 200.100.14		514	71,6				
2 L 200.100.16		474	89,1				

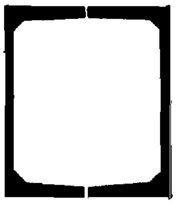
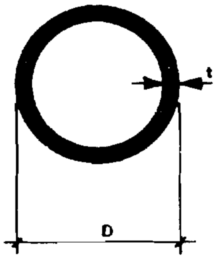


Table 5. (continued)

Box-shaped welded UNP-profiles

Profile		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	T	P_{kr}	T	P_{kr}
2 UNP 24		597	59,5				
2 UNP 26		578	72,2				
2 UNP 28		568	82,1				
2 UNP 30		556	93,8				
2 UNP 32		488	143,3				
2 UNP 35		500	146,6				
2 UNP 40		484	174,5				



Thick-walled pipes

Profile	t	FR 30 Min.		FR 60 Min.		FR 90 Min.	
D (mm)	(mm)	T	P_{kr}	T	P_{kr}	T	P_{kr}
114	12,5	572	30,4				
114	25	399	79,4				
133	12,5	567	36,5				
133	25	390	97,6				
159	16	494	67,1				
159	25	377	123,8				
194	16	487	84,7				
194	25	372	156,8				
219	20	421	136,0				
219	25	363	183,5				
244	16	479	110,6				
244	20	416	154,7				
244	25	363	207,7				
267	20	416	170,0				
267	25	358	231,2				
324	20	416	209,5				
324	25	354	287,0				

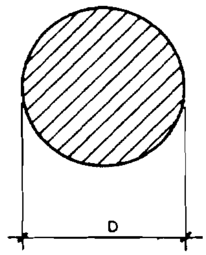


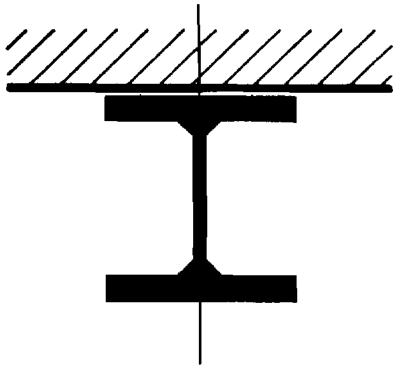
Table 5. (continued)

Rounds (σ_f = 2200 kg/cm²)

Profile D (mm)		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	T	P_{kr}	T	P_{kr}
100		334	91,1	594	51,1		
120		291	165,0	533	87,7		
140		257	224,8	483	134,9		
150		244	258,4	461	162,7		
180		211	372,3	406	261,6	565	181,8
200		194	458,4	376	339,8	529	246,6
240		179	659,9	350	511,1	496	384,2
260		157	775,2	309	641,4	443	506,7
280		148	899,3	291	899,3	420	614,8
300		140	1032,2	276	1032,2	400	732,9

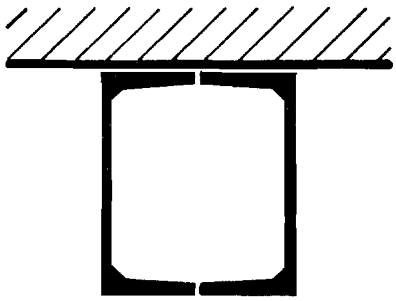
Table 6. Fire resistance of steel elements shielded on one side (exterior columns or beams)

FR Fire resistance in minutes
T Steel temperature
P_{kr} Critical load for axially compressed columns (t) of St37



Open wide-flange profiles

Profile		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	T	P_{kr}	T	P_{kr}
HEM 180		537	95,0				
HEM 200		521	114,8				
HEM 220		510	134,2				
HEM 240		450	205,0				
HEM 260		445	227,7				
HEM 280		438	252,4				
HEM 300		397	344,8				
HEM 320		394	356,7				
HEM 340		397	359,4				
HEM 360		401	360,1				
HEM 400		406	364,4				
HEM 500		419	375,7				
HEM 600		426	391,8				



Box-shaped welded UNP- profiles

Profile		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	T	P_{kr}	T	P_{kr}
2 UNP 24		524	73,4				
2 UNP 26		505	87,7				
2 UNP 28		496	98,9				
2 UNP 30		485	111,9				
2 UNP 32		423	164,3				
2 UNP 35		438	167,6				
2 UNP 40		425	197,5				

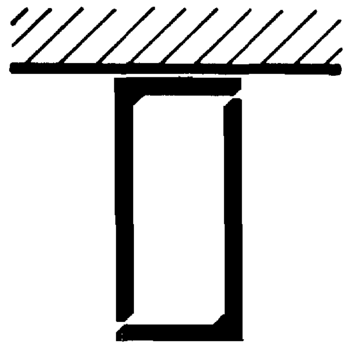


Table 6. (continued)

Box-shaped welded angle profiles

Profile		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	T	P_{kr}	T	P_{kr}
2 L 100.16		398	67,7				
2 L 100.20		343	90,3				
2 L 130.16		391	90,4				
2 L 150.16		389	105,5				
2 L 150.18		359	124,0				
2 L 150.20		334	142,7	594	80,1		
2 L 180.16		386	128,5				
2 L 180.18		356	151,2				
2 L 180.20		331	174,2	590	98,6		
2 L 200.20		330	194,9	588	110,6		
2 L 200.24		289	289,9	531	154,4		
2 L 200.28		259	336,0	485	200,0		
2 L 120.80.14		453	53,4				
2 L 150.100.14		448	68,5				
2 L 160.80.14		461	63,8				
2 L 200.100.14		457	81,5				
2 L 200.100.16		419	99,7				

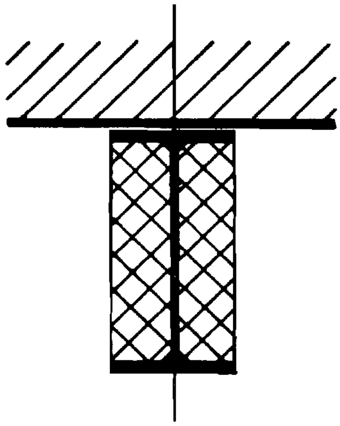


Table 6. (continued)

IPE profiles with concrete between the flanges

Profile		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	T	P_{kr}	T	P_{kr}
PE 200		533	24,1				
PE 220		512	29,8				
PE 240		487	37,0				
PE 270		465	45,6				
PE 300		452	55,0				
PE 330		426	67,3				
PE 360		402	82,0				
PE 400		380	99,0				
PE 450		357	120,5				
PE 500		334	146,8	595	82,3		
PE 550		311	176,0	563	104,7		
PE 600		294	249,6	537	130,7		
INP profiles with concrete between the flanges							
NP 20		463	33,3				
NP 22		437	41,6				
NP 24		416	50,6				
NP 26		392	61,1				
NP 28		371	72,6				
NP 30		354	84,6				
NP 32		337	97,9	599	54,4		
NP 34		322	112,1	578	64,9		
NP 36		306	128,3	555	77,4		
NP 40		281	188,8	519	103,6		
NP 45		255	235,2	480	141,5		
NP 50		232	286,4	442	186,9		
NP 55		218	339,2	419	231,6	579	157,7
NP 60		198	406,4	385	295,4	539	211,9

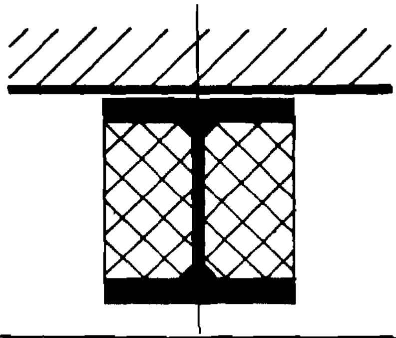


Table 6. (continued)

Wide-flange profiles with concrete between the flanges

Profile		FR 30 Min.		FR 60 Min.		FR 90 Min.	
		T	P_{kr}	T	P_{kr}	T	P_{kr}
HEA 200		467	53,3				
HEA 220		441	67,2				
HEA 240		412	85,0				
HEA 260		397	98,7				
HEA 280		384	113,3				
HEA 300		361	136,3				
HEA 320		340	156,1				
HEA 340		325	171,5	582	98,4		
HEA 360		313	186,9	566	110,7		
HEA 400		295	254,4	540	132,3		
HEA 450		277	284,8	513	158,8		
HEA 500		261	316,0	489	186,1		
HEA 550		254	338,8	477	205,2		
HEA 600		246	362,4	465	225,3		
HEB 200		367	93,6				
HEB 220		350	112,3				
HEB 240		330	135,0	589	76,5		
HEB 260		320	153,2	576	89,1		
HEB 280		312	172,3	564	102,4		
HEB 300		296	238,5	541	123,6		
HEB 320		279	258,0	516	142,5		
HEB 340		271	273,4	504	155,5		
HEB 360		264	288,9	493	168,9		
HEB 400		251	316,4	473	193,2		
HEB 450		239	348,8	453	222,4		
HEB 500		229	381,7	437	251,7		
HEB 550		223	406,5	426	273,5	589	183,9
HEB 600		217	432,0	416	296,2	577	202,4
HEM 160		284	155,3	524	84,3		
HEM 180		273	181,2	507	102,4		
HEM 200		260	210 0	488	124,2		
HEM 220		251	239,0	473	146,1		
HEM 240		215	319,3	413	220,3	573	151,3
HEM 260		210	351,3	405	246,0	564	171,1
HEM 280		205	384,3	396	273,9	552	193,3
HEM 300		181	484,9	354	371,4	502	277,6
HEM 320		179	499,2	350	385,3	496	289,8
HEM 340		178	505,2	348	390,7	494	294,3
HEM 360		178	510,0	349	394,0	495	296,6
HEM 400		178	521,2	348	403,5	493	304,2
HEM 450		177	536,6	347	415,8	493	313,7
HEM 500		177	550,8	347	427,2	492	322,5
HEM 550		175	567,0	343	442,3	487	335,4
HEM 600		174	581,9	342	454,8	486	345,4

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APPENDIX

GUIDELINES FOR THE FIRE PROTECTION OF STEEL IN HIGH BUILDINGS

(as of 28 March 1969)

This provisional guide is applicable to the
fire protection of steel

1. Principles

The fire protection of steel constructions is determined by the fire load and the criteria cited in these regulations. The fire load is a measure of the hazard as well as the fire resistance required for the load-bearing parts of the steel construction.

2. Fire Resistance

The fire resistance has to be determined so that the load-bearing ability of the construction will be retained and no permanent deformations occur with the burning of the fire load during the times stated in Section 7.

3. Fire Compartment

A fire compartment is that portion of a building enclosed by walls and floors which satisfy the requirements of the fire load. If no separations are present the fire compartment is bounded by the exterior building elements.

4. Calculation of the Fire Load

The fire load is the quantity of all combustible materials referred to the area of the fire compartment. It is expressed in Mcal per square metre. The fire load is obtained from the following formula:

$$q = \frac{\Sigma (G_i \cdot H_i)}{A}$$

where

- q Fire load in Mcal/m^2 of fire compartment area,
- G_i Weight of each combustible material in kg.
- H_i Heat of combustion of each combustible material in Mcal/kg
- A Area of the fire compartment in m^2.

Where the fire compartment contains installations such as mezzanines, false ceilings with openings, platforms of grids, sheets, etc., the area of the fire compartment A is taken as only the plan area.

5. Proof of Fire Load

The fire load must be authenticated. It includes all combustible materials (building, working, and storage) including the packaging. Materials worked or stored in a form which makes ignition impossible are not considered.

In addition, where combustibles are not distributed uniformly, the concentration of fire load near individual load-bearing construction parts must be considered.

6. Typical Fire Loads

For buildings whose fire load cannot be authenticated satisfactorily, the following fire loads apply:

(a)	Archives	480 $Mcal/m^2$
(b)	Office buildings	240 $Mcal/m^2$
(c)	Parking areas for passenger cars	60 $Mcal/m^2$
(d)	Storage rooms	480 $Mcal/m^2$
(e)	School buildings	160 $Mcal/m^2$
(f)	Sales rooms	400 $Mcal/m^2$
(g)	Residential buildings	240 $Mcal/m^2$

7. Required Fire Resistance

The course of a fire is to be taken according to the EMPA time-temperature curve. The fire duration and also the required fire resistance is dependent on the fire load and is given in the following table.

Fire load $Mcal/m^2$	Fire duration = required fire resistance in minutes	
	Normal rooms	Windowless and basement rooms
up to 60	unprotected	30
60 to 120	30	60
120 to 240	60	90
240 to 400	90	120
Over 400	120	180

8. Exceptions

For one-storey buildings and the uppermost storey of multi-storey buildings, fire protection of the structure is, as a rule, not necessary.

9. Calculation of Fire Resistance

The calculation of fire resistance must employ scientifically based calculation methods.* The authority having jurisdiction is empowered to request the calculation, and in the case of new materials and methods of attachment, may also demand verification by testing at a recognized testing laboratory.

10. Submission of Fire Protection Calculation

The fire protection calculations are to be submitted to the agency issuing the building permit.

11. Identification and Control of the Fire Load

In buildings in which the fire protection has been based on a calculated fire load, the latter must be clearly posted on a plaque in each fire compartment.

The fire police are responsible for the inspection of the fire load. If the permissible fire load is exceeded, the fire resistance of the structure must be increased.

Kantonale Gebäudeversicherung
Zürich
Feuerpolizei
Meier

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* For example, see the publication of the technical commission of the Swiss Centre for Steel Construction, "The Calculation of the Fire Resistance of Steel Constructions", March 28, 1969. (Publication on hand).