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EUROPÄISCHE KONVENTION FÜR STAHLBAU

Av. Louise 326, Bte. 52, B-1050 Brussels, Belgium

ECCS - Technical Committee 3 - Fire Safety of Steel Structures

European Recommendations for the Fire Safety of Steel Structures

Calculation of the Fire Resistance of Load Bearing Elements and Structural Assemblies Exposed to the Standard Fire

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ECCS General Secretariat
CECM Avenue des Ombrages, 32/36 bte 20
EKS B -1200 BRUSSELS (Belgium)

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FOREWORD

The European Recommendations for the Calculation of the Fire Resistance of Load Bearing Steel Elements and Structural Assemblies Exposed to the Standard Fire have been composed within the European Convention for Constructional Steelwork by Technical Committee 3 - Fire Safety of Steel Structures.

During the last decade there has been important progress in the development of analytical methods for the calculation of the behaviour of fire exposed steelwork. In several countries these methods are now being used where appropriate, as an alternative to the conventional classification directly based on the results of standard fire resistance tests. The Recommendations provide a reference document for national codes of practice with the final objective of achieving a single European code.

The present ECCS-Recommendations deal only with load-bearing steel elements and structural assemblies exposed to the standard fire, providing an alternative to the standard fire resistance test. This corresponds to the majority of national building regulations or codes, in which the requirements are expressed as required time of fire duration, i.e. the fire resistance time, directly related to the standard fire.
A direct analytical design of an element or assembly exposed to a non-standard or natural fire exposure is not covered in the present Recommendations. However, when the natural (non-standard) fire exposure is related to the standard fire by the concept of the "effective fire duration", the Recommendations can be used.

The Recommendations offer a variety of analytical methods, as comprehensive as possible:

(1) tabulated design data
(2) simple calculations, using average values of thermal properties for insulating materials and steel, and variation of yield stress with temperature
(3) more detailed calculations, allowing for variation of thermal properties with temperature and complete stress strain relationships for steel at elevated temperatures.

So far, the Recommendations cover only traditional structural steelwork. It is intended that technical notes on subjects such as composite structures, assemblies with suspended ceilings and external steelwork will follow in due course.

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Drafting and other preparatory work for the Recommendations has been made mainly by the Institute TNO for Building Materials and Building Structures, Delft, The Netherlands.
The calculation rules adopted here have been prepared by a drafting group of ECCS-Committee T3 with the following membership:

J. Witteveen (chairman)
L. Twilt (secretary)
S. Bryl
J. Kruppa
M. Law
O. Pettersson

The membership of Committee T3 - Fire Safety of Steel Structures - during the preparation of the Recommendations was as follows:

J. Witteveen	(chairman)	- Netherlands
L. Twilt	(secretary)	- Netherlands
P. Arnault		- France
W. Bongard		- Germany
S. Bryl		- Switzerland
G.B. Colbridge		- Great Britain
L. Demol		- Belgium
H. Ehm		- Germany
M. Finzi		- Italy
J. Janss		- Belgium
L. Krampf (Mrs.)		- Germany
J. Kruppa		- France
M. Law (Mrs.)		- Great Britain
E. Pedersen		- Danmark
O. Pettersson		- Sweden
F. Pfohl		- Austria
D. Sfintesco		- France
J. Thor		- Sweden
E. Thrane		- Norway
A. Tovar de Lemos		- Portugal
I. Tricario		- Italy
H. Walker		- Great Britain

The Recommendations have been agreed with the Code Advisory Panel of the Fire Commission W14 of the Conseil International du Bâtiment (CIB) in which

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the following international organizations participate:

- Conseil International du Bâtiment (CIB),
- Comité Europeén du Béton (CEB),
- Fédération International de la Précontrainte (FIP),
- European Convention for Constructional Steelwork (ECCS),
- Confédération Européenne de l'Industrie du Bois (CEI-Bois).

Research programmes sponsored by the European Community for Coal and Steel and carried out during the first half of the nineteen seventies should also be acknowledged here.

It is the hope of Technical Committee 3 that these Recommendations will be well used and be of value not only to ECCS-member countries, but also to other nations.

Delft, June 1982

J. Witteveen,

Chairman of the ECCS-Technical Committee 3.

Comments to be received by the secretary of the ECCS-Technical Committee 3:

L. Twilt,
IBBC-TNO,
P.O. Box 49,
2600 AA Delft,
The Netherlands.

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INTRODUCTION

Internationally, the generally accepted method for the fire engineering design of load-bearing elements and structural assemblies uses results of standard fire resistance tests. In such tests, the load bearing element or structural assembly is exposed in a furnace in which the temperature is controlled to vary with time according to the formula [1]:

$$\vartheta_t - \vartheta_o = 345 \log_{10} (8t + 1)$$

where:

t = time in minutes

ϑ_t = furnace temperature in ^oC at time t

ϑ_o = initial furnace temperature in ^oC

The fire resistance of a load-bearing element is defined as the time, expressed in minutes, from commencement of the test until failure occurs at the applied loading. Normally, the test load is the design load at service state.

In practice the requirements of building codes and regulations are expressed directly in terms of a specified fire resistance.

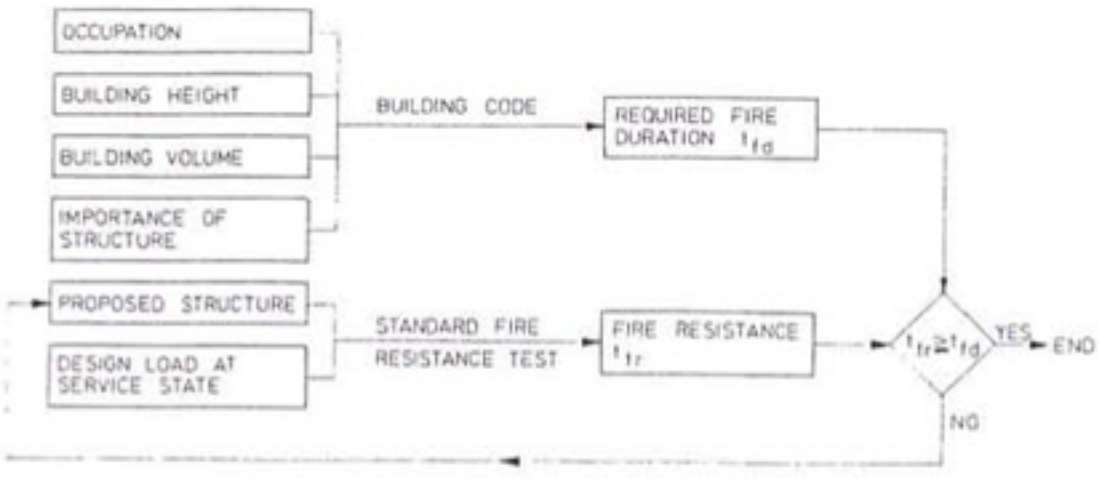

Figure 1: Main international method for the fire engineering design of load bearing elements and structural assemblies, based on classification and results of standard fire resistance tests.

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The design procedure is illustrated by Fig. 1. According to the circumstances, the codes and regulations require a time of fire duration t_{fd}, for which the structure or structural element must fulfil its load bearing function. This period is also related to the standard fire exposure and referred to as "fire resistance time". The design implies a proof that the structure or structural element has a fire resistance t_{fr}, determined in a standard fire resistance test, which meets the required time of fire duration t_{fd}.

Internationally, the standard fire resistance test according to ISO 834 is used very frequently and for many types of structural members it constitutes the only way at present for obtaining the information required for a structural fire engineering design. In spite of this, the standard fire resistance test can be seriously criticized. In its present form, the test procedure is insufficiently specified in several respects, for instance concerning the heating and restraint characteristics, the environment of the furnace, and the thermocouples for measuring and regulating the furnace temperature. Consequently, a considerable variation can arise in the fire resistance for one and the same structural element, when tested in different fire laboratories with varying furnace characteristics and varying support and restraint conditions.

Because of these problems and to achieve solutions with greater economy and more uniform safety, there is a strong need to move towards analytical structural fire engineering design methods. Generally, these methods include two main steps, viz.:

(1) a calculation of the temperature distribution within the fire exposed load-bearing element or assembly during the heating process;
(2) a transformation of these temperature distributions to the variation of the load bearing capacity as a function of time in order to examine whether or not the fire exposure will cause a failure of the structural element or assembly at the specified loading.

During the last decade, considerable progress has been made in developing analytical design methods for fire exposed load bearing elements and structural assemblies and in making these design methods operational, using design diagrams and tables. This approach is most advanced in the field of steel structures and a major contribution has been the extensive information, experience and studies from many countries coordinated by the Fire Committee of the European Convention for Constructional Steelwork (ECCS).

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The three available design methods can be categorized as follows:

- Level (1): Method in which the fire resistance of a load bearing element or structural assembly under standard fire conditions is determined analytically. The design criterion is that the analytically determined fire resistance is equal to or exceeds the time of fire duration required by the building regulations or codes.
- Level (2): Method in which the fire resistance of a load bearing element or structural assembly under standard fire conditions is determined analytically. The design criterion is that the analytically determined fire resistance is equal to or exceeds the "effective fire duration", a quantity which relates a non-standard or natural fire exposure to the standard fire.
- Level (3): Method characterized by a direct analytical design on the basis of a natural (non standard) fire exposure.

The Recommendations concentrate on the analytical design methods according to level (1), i.e. methods for an analytical determination of the fire resistance of load bearing steel elements and structural assemblies under standard fire conditions, providing an alternative for the standard fire resistance test. However the Recommendations can also be used as a component in a level (2) design.

A design procedure according to level (1) corresponds to a vast majority of national building codes, in which the requirements are expressed as a required time of fire duration, i.e. the fire resistance time, directly related to the standard fire. The design procedure is illustrated in Fig. 2. It is necessary to appreciate that the required time of fire duration may be related not only to the estimated fire exposure but also to additional factors such as the type of occupation and the height and the situation of the building. This usually leads to a required time of fire duration which is more severe than the actual fire exposure.

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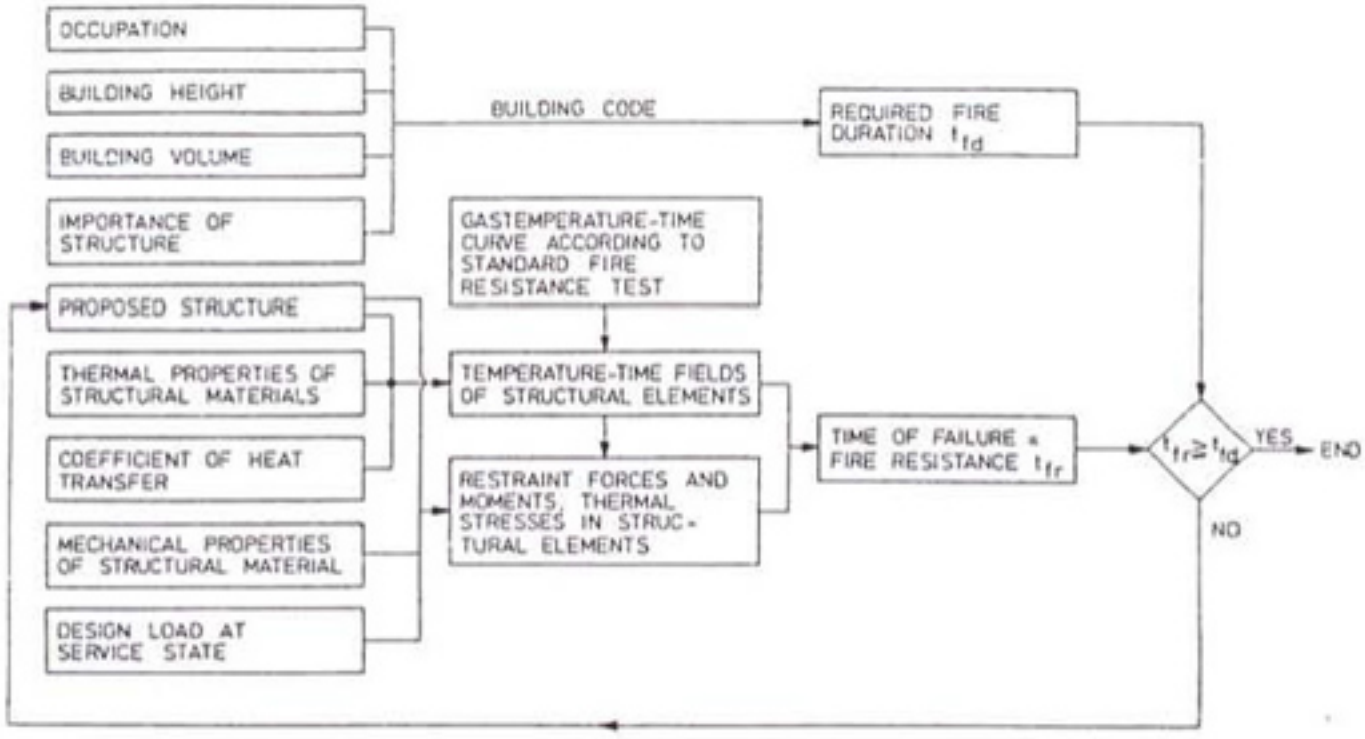

Figure 2: Analytical fire engineering design of load bearing elements and structural assemblies, based on a thermal exposure according to the ISO 834 standard fire resistance test.

For level (2), the natural (non standard) fire exposure is related to the standard fire by the concept of an "effective fire duration", which is a function of the fire load, the geometry and the ventilation conditions of the fire compartment. A level (2) design therefore requires an analytical determination of the fire resistance of load bearing elements and structural assemblies under standard fire conditions in the same way as for level (1).

Level (3) is characterized by a direct analytical design based on calculated gas temperature-time curves of a natural fire and is not covered in the present Recommendations.

In most countries the level (2) and level (3) methods have occasionally been used, but, except in Sweden, they are not yet automatically accepted as methods which satisfy the requirements of the building regulations.

Although the Recommendations offer a variety of analytical methods, the design basis is focussed on one basic method, characterized by the following four assumptions:

- a thermal exposure to the ISO 834 standard fire resistance test,
- constant thermal properties of structural and insulation materials assumed

to be the average for the temperature range,

- mechanical properties of steel at elevated temperature which are assumed to be independent of time, i.e. creep effects are included implicitly,
- a uniform temperature distribution over the height of the cross section and along the members.

These assumptions allow a simplified analytical design procedure in which the structural analysis at elevated temperatures can be dealt with according to conventional methods of structural analysis at room temperature (Fig. 3).

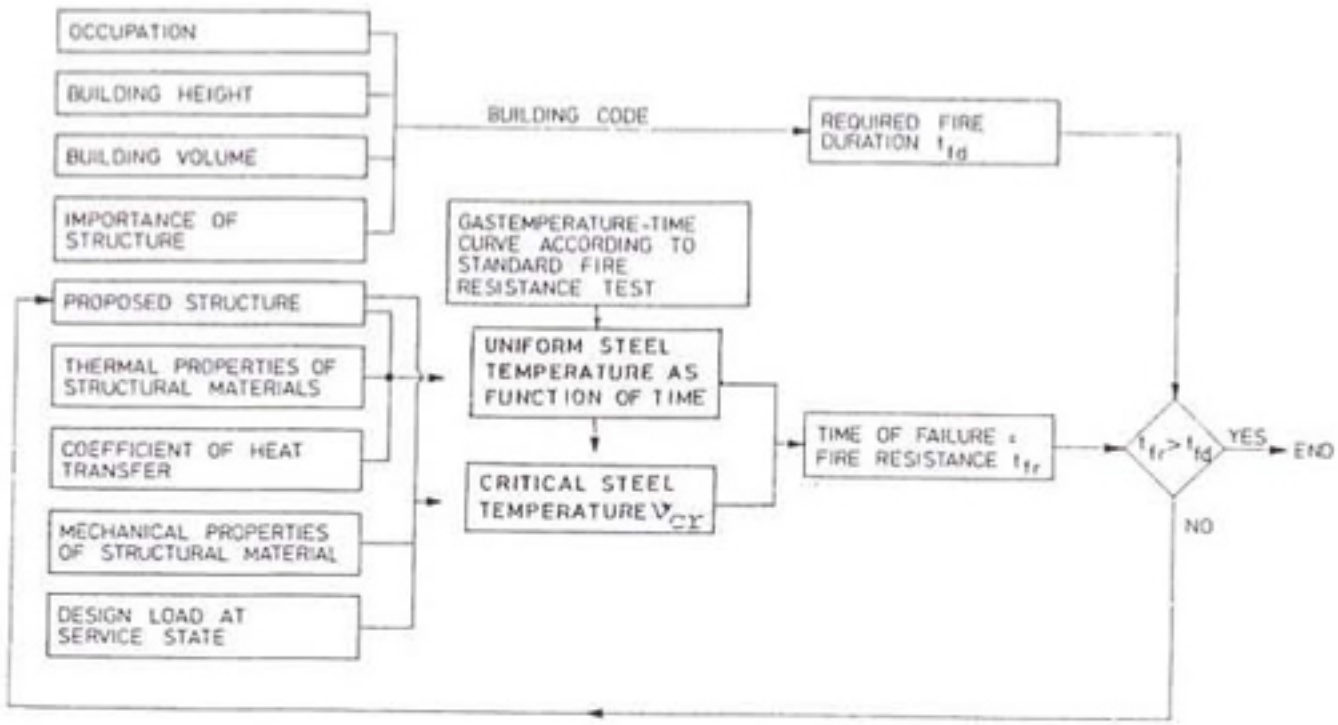

Figure 3: Analytical fire engineering design of load bearing elements and structural assemblies under standard fire exposure; using the concept of critical steel temperature ϑ_{cr} adopted in the ECCS-Recommendations.

In this concept the key-factor is the determination of a unique critical steel temperature ϑ_{cr} at which the structural element collapses under a given load. The procedure starts with the load bearing capacity R of the thermally exposed structural element, put equal to the design load S. This quantity and the mechanical properties of the structural material at elevated temperatures define a critical steel temperature ϑ_{cr}, at which the structural element collapses. Given the thermal properties of the structural and insulation materials and the heat flow characteristics, the uniformly distributed temperature in the steel member as a function of time is calculated. Subsequently the temperature-time relationship in the steel member and the critical temperature ϑ_{cr} define the time of failure t_{fr}.

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The calculated time of failure t_{fr} is theoretically identical to the fire resistance time obtained in a test with the same structural element and design load. The design implies a proof that the calculated time of failure t_{fr} is equal to or exceeds the time of fire duration t_{fd} required by the building code for the structural application in question.

An analytical design based upon the four specified assumptions, generally results in a level of fire resistance which is conservative, in comparison with that measured in a standard fire resistance test. Generally, a comparatively favourable non-uniform temperature distribution will exist during the test and the actual yield stress of the test piece will exceed the nominal value used in the calculations. Therefore a correction for the design load is introduced which makes allowance for these discrepancies.

For the experimental determination of the fire resistance of steel structures and structural elements, see ISO 834, and for the general background philosophy and basis of design, see the CIB-Recommendations "The Basis of Design for the Fire Protection of Building Structures".

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RECOMMENDATIONS

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1.1 and 1.2 Traditionally, the fire resistance of building structures has been designed to meet the requirements of national regulations, which vary from country to country, or even from region to region within a country.

The Recommendations give methods and criteria designed to meet these requirements as an alternative to the standard fire resistance test.
The requirements generally specify the time of fire duration during which the structure must fulfil its function. The required time of fire duration depends on many factors, such as the type of occupancy, and the height and volume of the building. In most countries the required time of fire duration is expressed as the time the structure or structural element can resist the standard fire.

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Art. 1 SCOPE.

1.1 Objectives.

These European Recommendations specify methods and criteria for the calculation of the fire resistance of load bearing steel elements and structural assemblies exposed to the standard fire.

1.2 Field of application.

These Recommendations are concerned with the structural behaviour of cased and uncased steel members under standard temperature conditions.
Other types of construction such as composite structures (steel-concrete) or alternative protection systems such as watercooling, heat shields and suspended ceilings are dealt with elsewhere.
See Foreword.

The Recommendations give alternative methods and criteria designed to meet the requirements of national codes and regulations.

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2.3 The mechanical properties of steel given in 2.3 are based on data obtained in The Netherlands and elsewhere. This does not preclude the use of new data, resulting in different stress-strain relationships which of course will have an effect on the critical temperature of beams and in particular of columns.

2.3.1 It appears that the stress induced strains of steel start to be affected by creep at temperatures over about 450°C. This means that, strictly speaking, in estimating the deformation behaviour of steel at temperatures over 450°C, the stress and temperature history should be taken into account. Using creep laws formulated in [2], such an approach has been worked out for beams in [3]. Since the method of calculation is rather complicated, and worked out so far only for beams, it has been decided to give a more approximate method for analysing the deformation behaviour. This method is based on the fact that, as is shown in [4], [5], for practical purposes the heating rate does not significantly influence the deformation behaviour of steel structures under fire conditions. This means that it is possible to define stress-strain relationships in which, for practical applications, the effect of creep is included. The advantage of such an approach is that the stress and temperature history need not to be taken into account explicitly.

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Art. 2 STEEL PROPERTIES AT ELEVATED TEMPERATURES.

2.1 Steel qualities.

The steel properties given in this article are for steel qualities Fe 310, Fe 360, Fe 430, Fe 510 as specified in the Euronorm 25-72. In all other cases the reliability of the given design values must be demonstrated explicitly.

2.2 Density.

The density of steel is considered to be independent of the temperature. In the calculations the following value, valid for all steel qualities, should be taken: $\rho_s = 7850\ kg/m^3$.

2.3 Mechanical properties.

2.3.1 Stress-strain relationships for Fe 360

The stress-strain curves for Fe 360, can, for temperatures in the range between 20°C and 600°C be approximated by the curves given in Fig. r.1.

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Most investigations of the mechanical properties of steel have been made for temperatures up to about 600°C. Steel properties beyond 600°C are not so well known, but there is evidence that creep becomes significant. On the other hand detailed knowledge of the steel properties in this temperature region is not essential, since normally, critical conditions will be reached at temperatures lower than 600°C. Consequently no detailed σ-ε diagrams are given for temperatures beyond 600°C.

To apply the elementary plastic theory, the stress-strain curves are cut off for a certain stress level, as shown by the dashed line in Fig. c.1. This stress level is defined as the effective yield stress, and decreases at increasing temperature. See Fig. r.2.

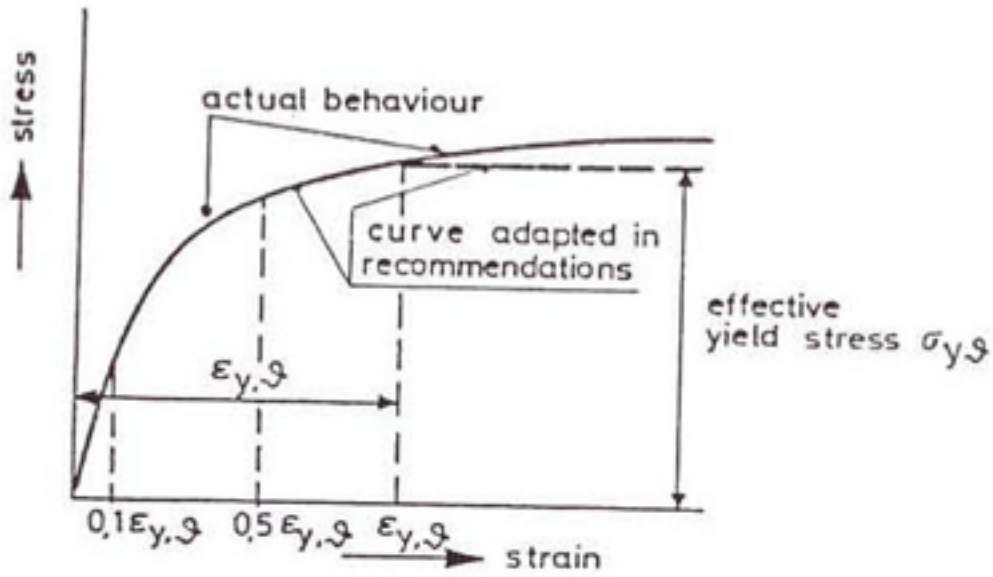

Fig. c-1: Simplification of the stress-strain curves of steel at elevated temperatures.

Tabulated values of the recommended stress strain relations, are given in Appendix I to these Recommendations.

2.3.2 The recommended transformation is based on the following assumptions:

- For a given grade of steel the reduction in the effective yield stress at a temperature ϑ_s is assumed to be the same as for Fe 360.

  i.e. $\Psi = \dfrac{\sigma_{y,\vartheta}}{\sigma_{y,20}}$ for all grades

- For any grade of steel the modulus of elasticity $E_{t\vartheta}$ at zero stress is assumed to be equal to that of Fe 360 throughout the temperature range.

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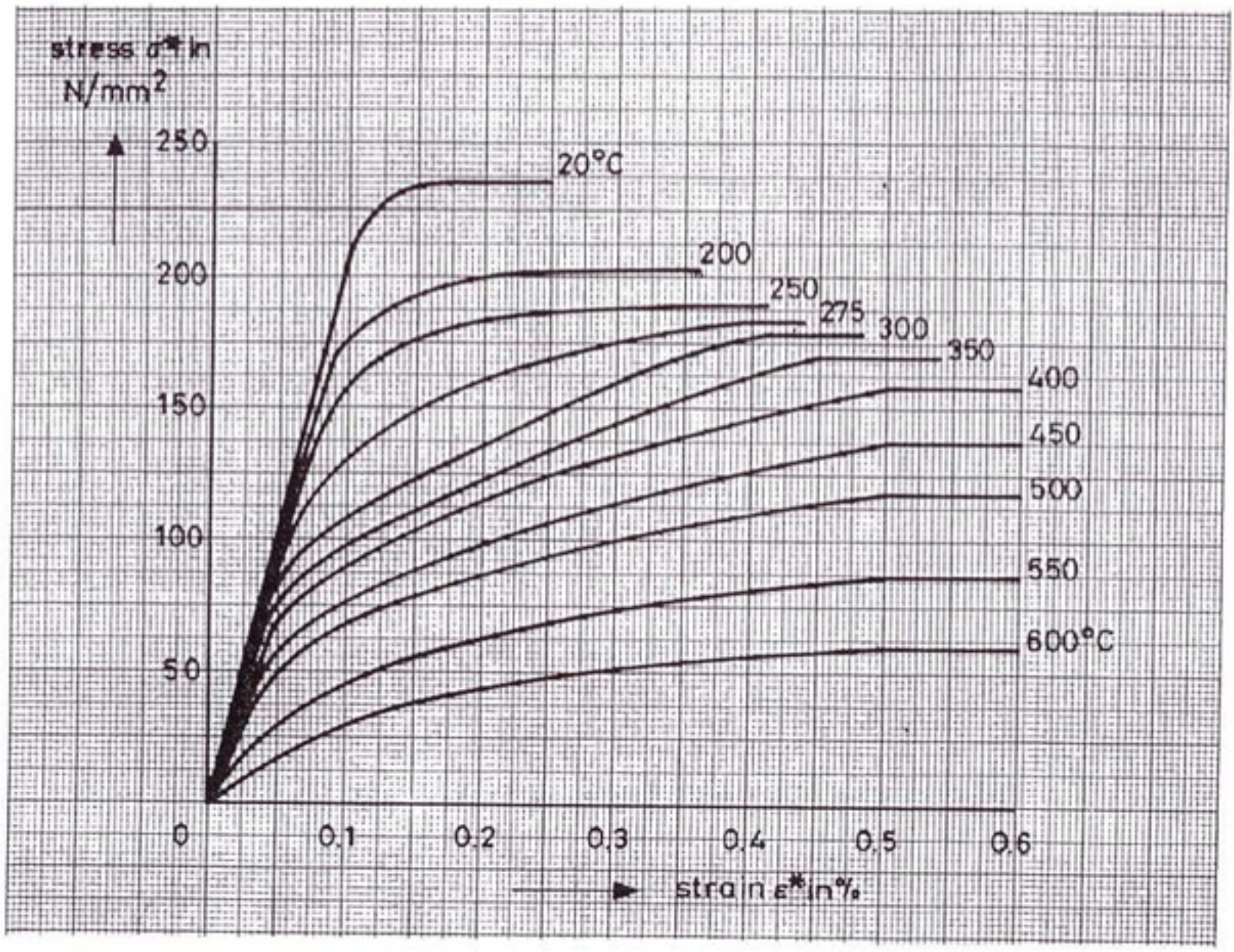

Fig.r-1: Stress-strain relationships at elevated temperatures for steel grade Fe 360.

2.3.2 Stress-strain relationships for other grades of steel.

For grades of steel, other than Fe 360, the stress-strain curves may be derived from the curves given in Fig. r.1 by the following transformation:

$$\sigma = \sigma^{*} + \Psi(\sigma_{y,20} - 235) \qquad [N/mm^2]$$

and

$$\varepsilon = \varepsilon^{*} + \left[\Psi \cdot \frac{(\sigma_{y,20} - 235)}{E_{t\vartheta}}\right] 100 \qquad [\%]$$

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The transformation is illustrated in Fig. c-2

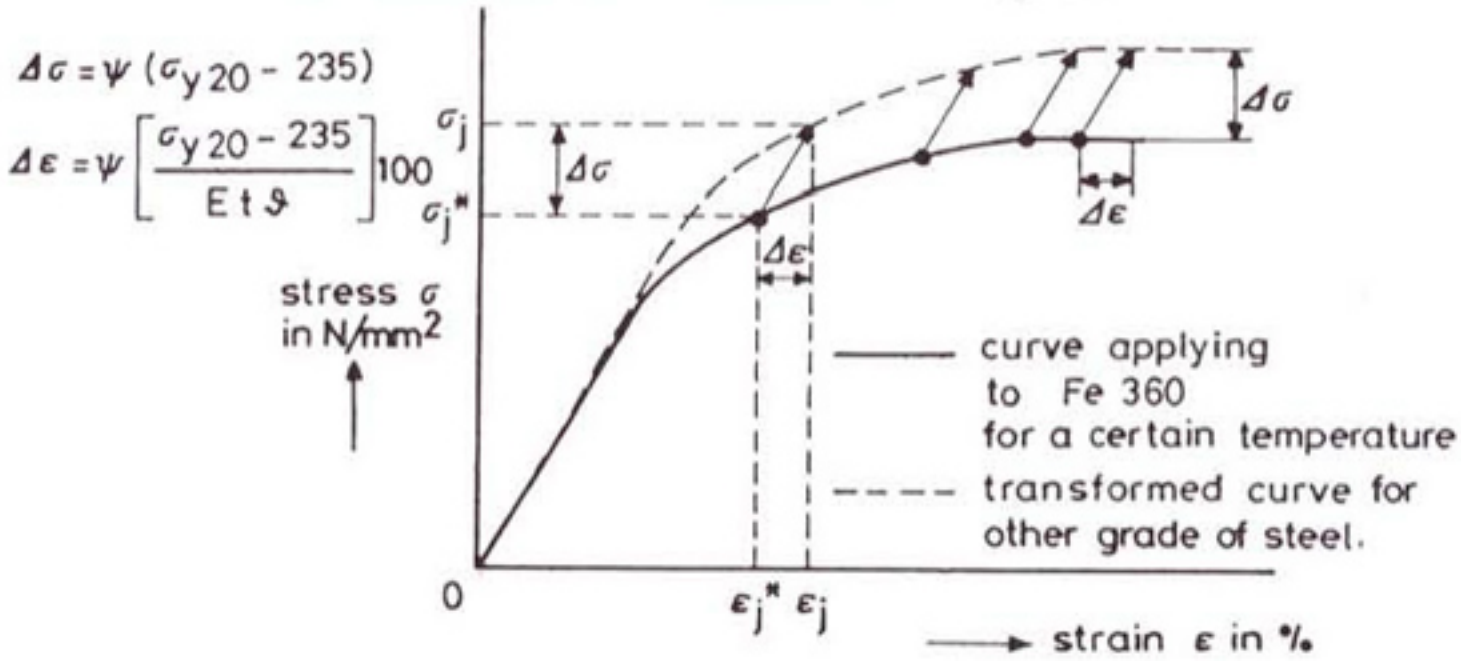

Fig. c-2: Transformation of the stress strain curves for Fe 360 to other grades of steel (schematic)

The stress-strain relationships for Fe 310, Fe 360, Fe 430 and Fe 510 are given in Fig. c-3. Tabulated values are given in Appendix I to these Recommendations.

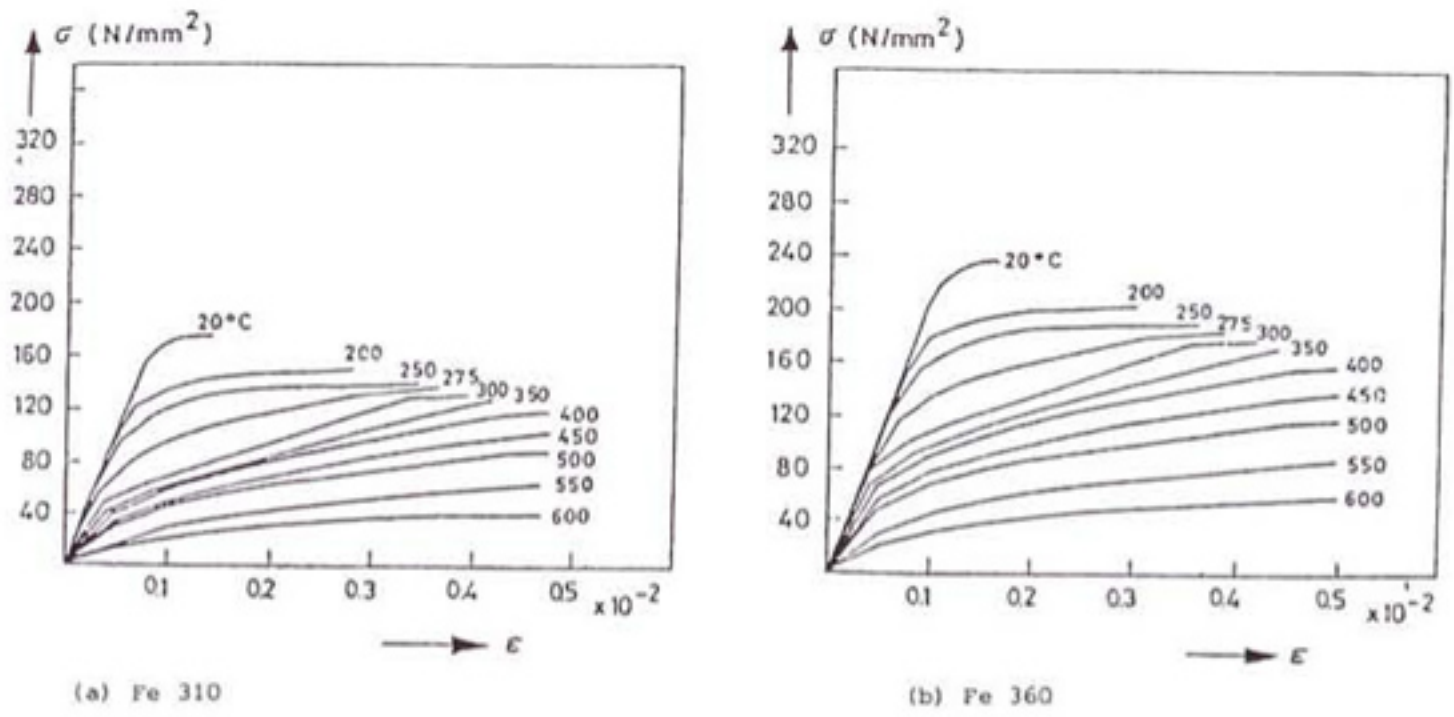

Fig. c-3: Stress strain relationships at elevated temperatures for various grades of steel.

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where:

σ = actual stress $[N/mm^2]$

σ^* = stress parameter applying to Fe 360 and given in Fig. r.1. $[N/mm^2]$

$\psi = \frac{\sigma_{y,\vartheta}}{\sigma_{y,20}}$ = temperature dependent coefficient in accordance to 2.3.3 and given in Fig. r.2.

$\sigma_{y,20}$ = nominal yield stress of the used steel at room temperature as specified in Euronorm 25-72 $[N/mm^2]$

$\sigma_{y,\vartheta}$ = effective yield stress of steel at elevated temperature $[N/mm^2]$

ε = actual strain [%]

ε^* = strain parameter applying to Fe 360 and given in Fig. r.1 [%]

$E_{t\vartheta}$ = temperature dependent modulus of elasticity for $\sigma = 0$ according to 2.3.4 and given in Fig. r.3. $[N/mm^2]$

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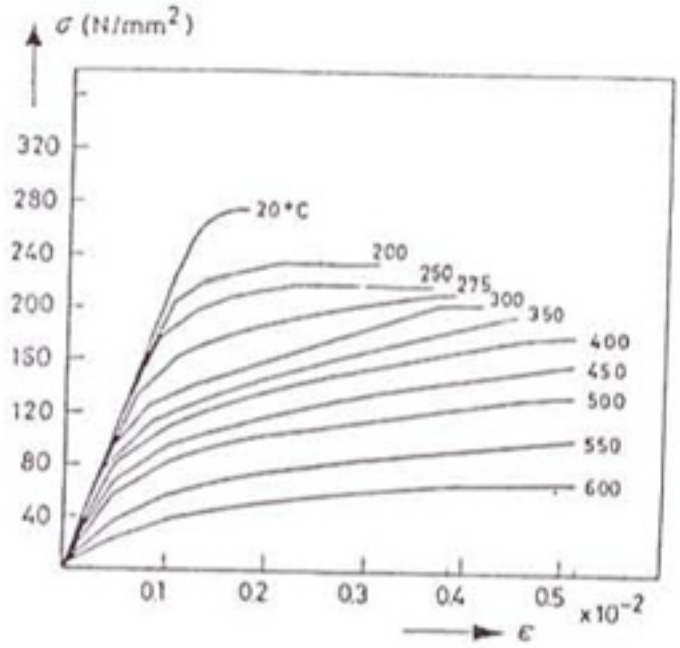
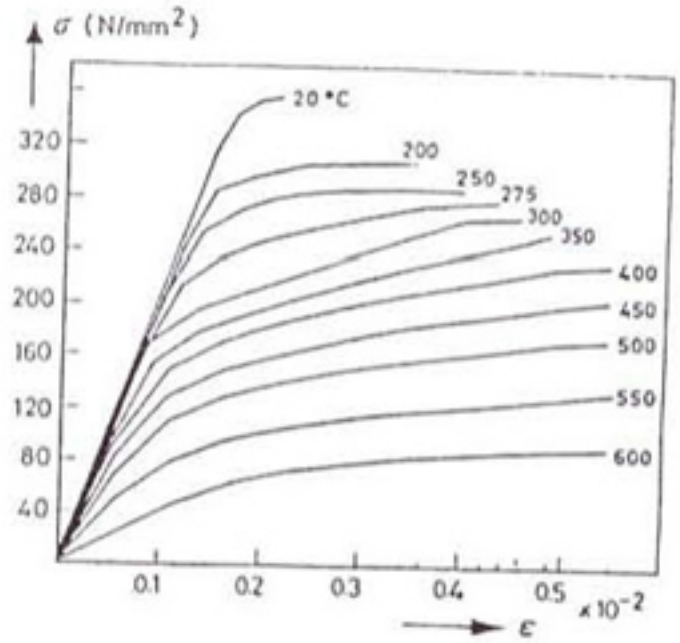


(c) Fe 430



(d) Fe 510

Fig. c-3: continued.

2.3.3 Although for temperatures over 600°C, no stress-strain curves are given (cf. Fig. r-1), it can be shown, that steel only fully loses its strength at temperatures of about 1000°C. For completeness the relationship between effective yield stress and temperature in the region between 600 and 1000°C is shown by the approximate extrapolation in Fig. r.2. This relationship is, because of creep, only suitable for approximate analyses. More detailed analyses must be based on a proper creep theory [2], [3].

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2.3.3 Effective yield stress

The effective yield stress decreases with temperature and may -for all grades of steel- be approximated by the following equations:

$0 \leq \vartheta_s \leq 600^{\circ}C$

$$\Psi = \frac{\sigma_{y\vartheta}}{\sigma_{y20}} = 1 + \frac{\vartheta_s}{767 \ \ln \frac{\vartheta_s}{1750}}$$

$600^{\circ}C \leq \vartheta_s \leq 1000^{\circ}C$

$$\Psi = \frac{\sigma_{y\vartheta}}{\sigma_{y20}} = \frac{108 \ (1 - \frac{\vartheta_s}{1000})}{\vartheta_s - 440}$$

where:

$\sigma_{y\vartheta}$ = effective yield stress at elevated temperature

σ_{y20} = nominal yield stress at room temperature

ϑ_s = steel temperature in $^{\circ}C$

$\Psi = \frac{\sigma_{y\vartheta}}{\sigma_{y20}}$ = temperature dependent coefficient.

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2.3.4 Due to the curved shape of the stress-strain relationships at elevated temperature, the modulus of elasticity in fact will be a function of the stress. It is noted in this respect that the recommended values apply to the tangent modulus for an infinitely small stress, viz.:

$$E_{t\vartheta} = \lim_{\sigma \to 0} \left(\frac{d\sigma}{d\varepsilon}\right)$$

Since the effect of creep must be analysed explicitly for temperatures over 600°C, $E_{t\vartheta}$ is not defined in this temperature range (see also comments to 2.3.1).

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The relation between $\frac{\sigma_{y\vartheta}}{\sigma_{y20}}$ and ϑ_s is given in Fig. r-2.

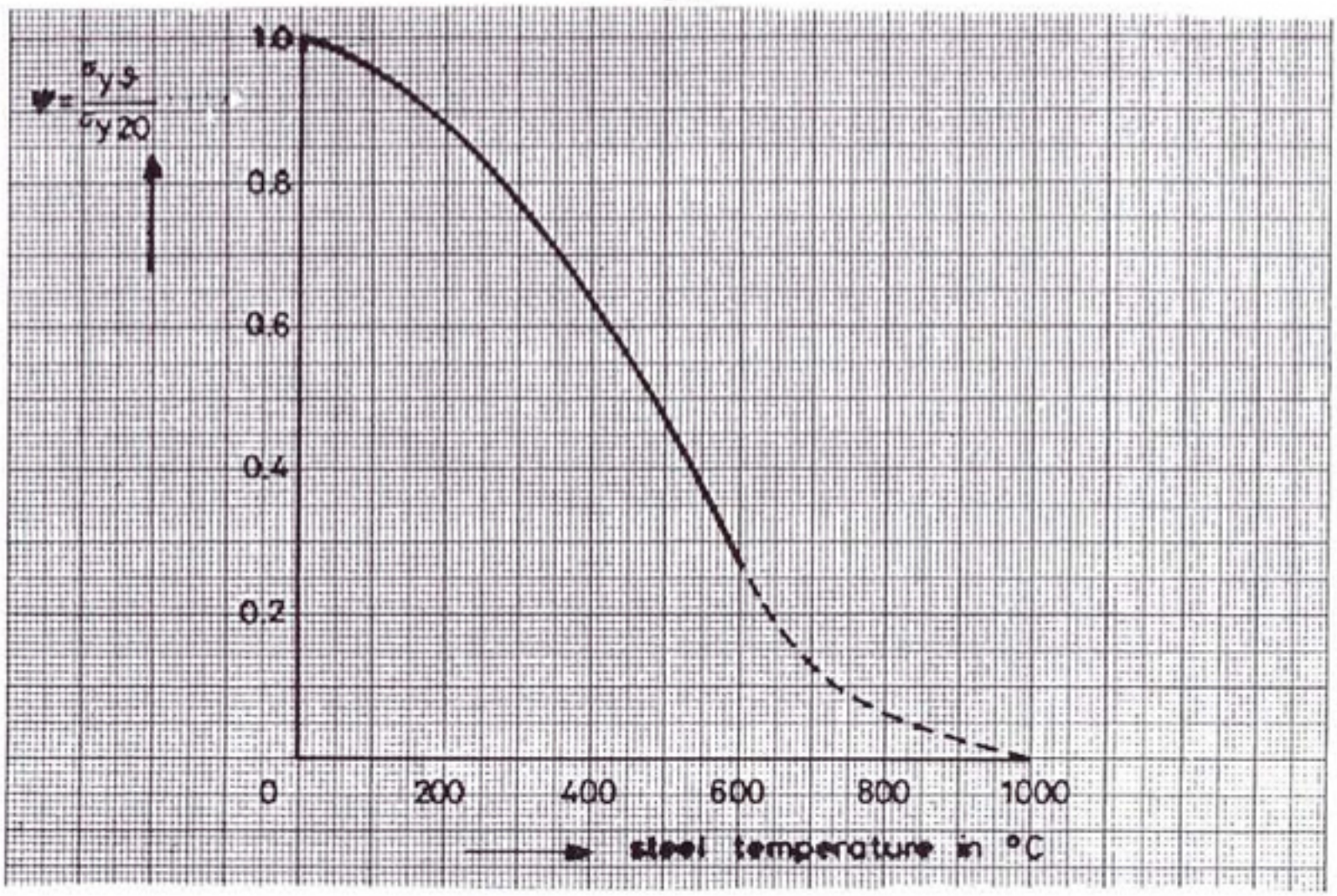

Fig. r-2: Ratio Ψ of the effective yield stress at elevated temperature to the nominal yield stress at room temperature as a function of the steel temperature.

2.3.4 Modulus of elasticity

The modulus of elasticity for $\sigma = 0$ decreases with temperature and may -for all grades of steel- be approximated by the following equation:

$0 \leq \vartheta_s \leq 600^\circ C$

$$E_{t\vartheta} = E\,(-17.2 \times 10^{-12}\,\vartheta_s^4 + 11.8 \times 10^{-9}\,\vartheta_s^3 - 34.5 \times 10^{-7}\,\vartheta_s^2 + 15.9 \times 10^{-5}\,\vartheta_s + 1) \quad [N/mm^2]$$

$\vartheta > 600^\circ C$: $E_{t\vartheta}$ not defined.

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where:

$E_{t\vartheta}$ = modulus of elasticity at elevated temperature for $\sigma = 0$ N/mm^2

E = modulus of elasticity at room temperature (= 2.1×10^5 N/mm^2)

ϑ_s = steel temperature in °C

The relation between $E_{t\vartheta}$ and ϑ_s is given in Fig. r-3.

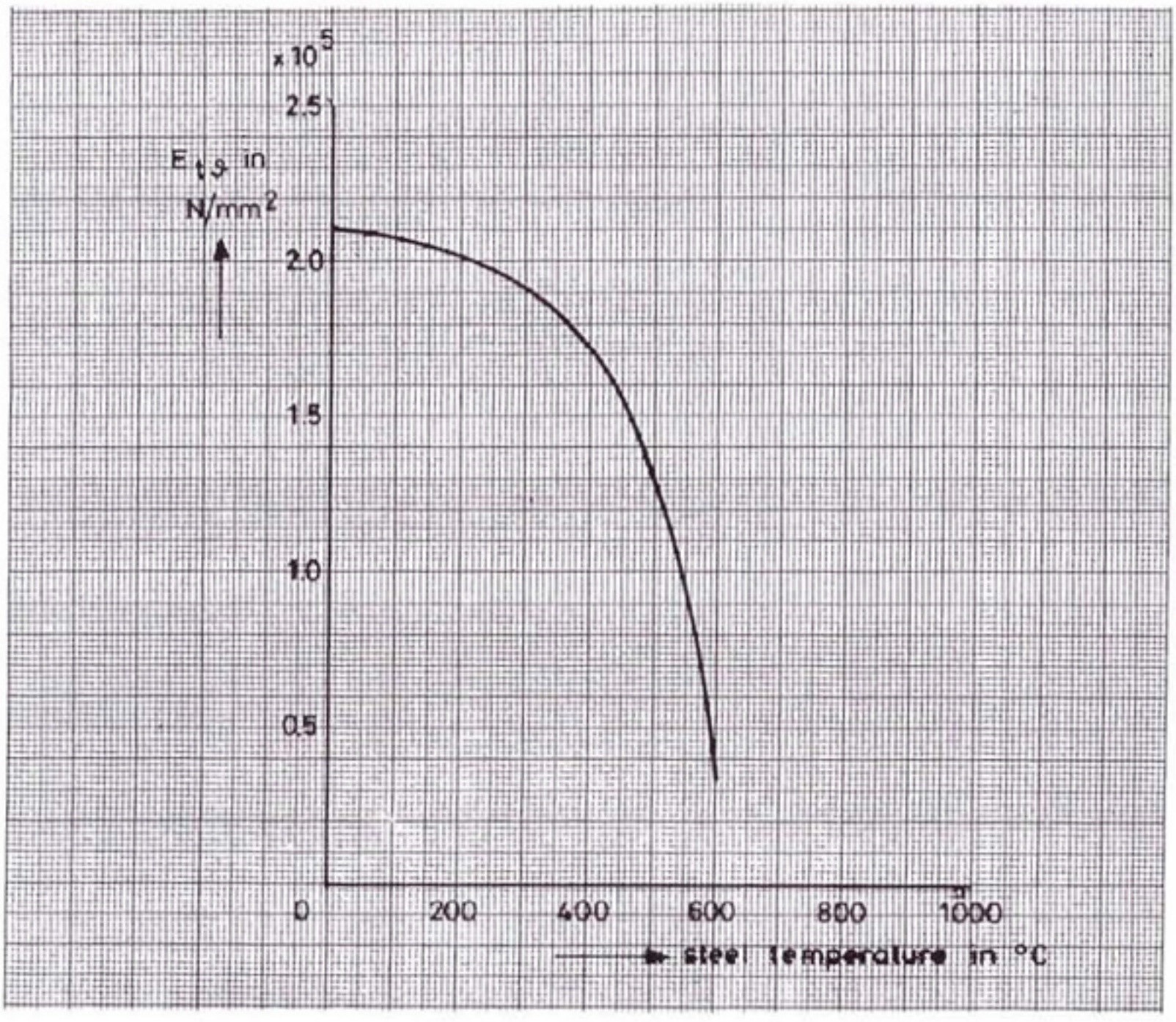

Fig. r-3: Modulus of elasticity $E_{t\vartheta}$ as function of the steel temperature.

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2.4 The coefficient of thermal expansion of steel is a funcion of steel temperature. This is illustrated in Fig. c.4 for a practical temperature range. The dashed line in this figure corresponds to the equation:

$$\frac{\Delta \ell}{\ell} = 0.4 \times 10^{-8}\, \vartheta_s^{\,2} + 1.2 \times 10^{-5}\, \vartheta_s - 3 \times 10^{-4} \qquad [m/m]$$

If necessary this line may be used as an alternative to the approximation recommended in 2.4 and represented by the dashed line in Fig. c.4.

2.5 The specific heat of steel is a function of steel temperature. This is illustrated in Fig. c.5 for a practical temperature range. The drawn line in this figure corresponds to the equation:

$$c_s = 38 \times 10^{-5}\, \vartheta_s^{\,2} + 20 \times 10^{-2}\, \vartheta_s + 470 \qquad [J/kg\,^{\circ}C]$$

If necessary this equation may be used as an alternative for the fairly rough approximation as given in 2.5 and represented by the dashed line in Fig. c.5.

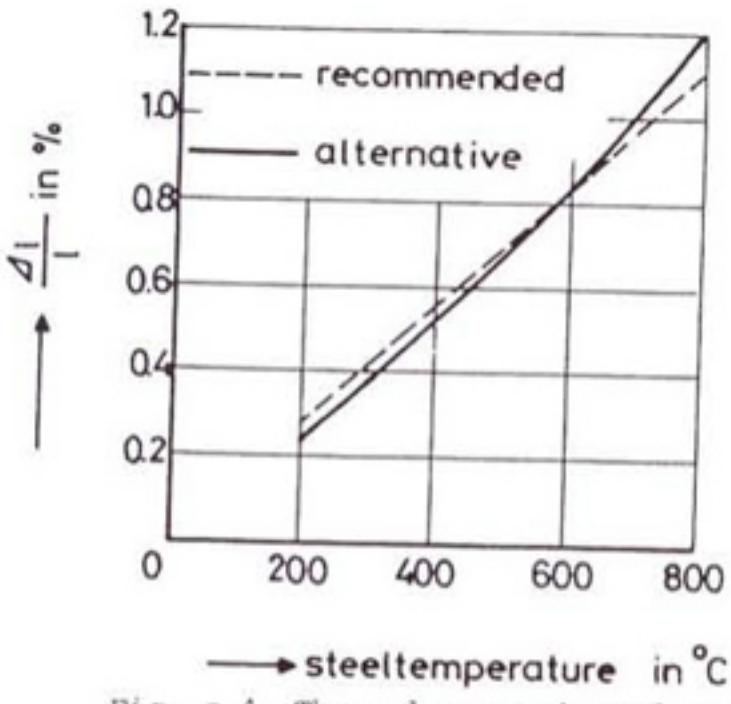

Fig. c-4: Thermal expansion of steel as a function of the steel temperature.

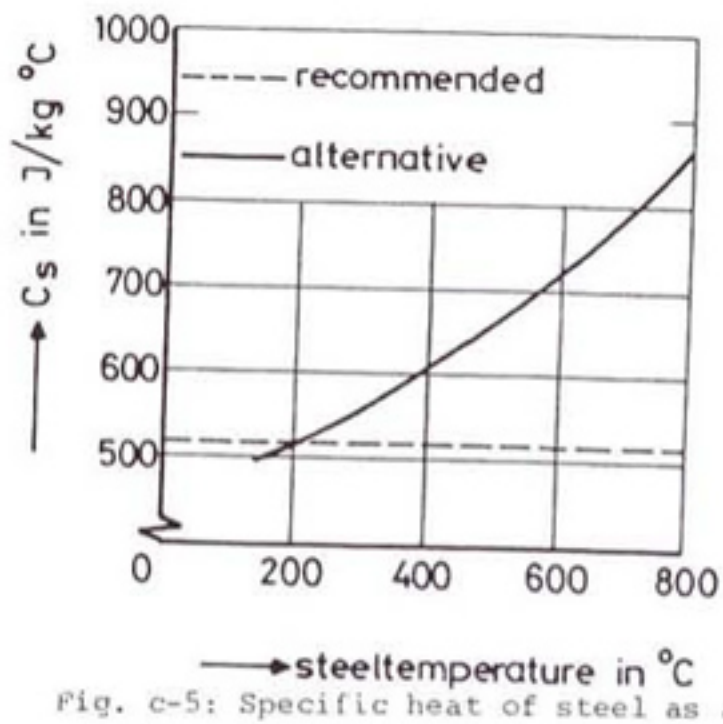

Fig. c-5: Specific heat of steel as a function of the steel temperature.

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2.4 Thermal expansion.

The coefficient of thermal expansion may, as an approximation, be considered to be independent of the steel temperature.
In calculations the thermal expansion, valid for all grades of steel, may be defined by:

$$\frac{\Delta \ell}{\ell} = 1.4 \times 10^{-5} \; \vartheta_s \qquad [m/m]$$

where:

ℓ	= length at room temperature	[m]
$\Delta \ell$	= temperature induced expansion	[m]
ϑ_s	= steel temperature	[°C]

2.5 Specific heat.

The specific heat may as an approximation, be considered to be independent of the steel temperature.
In calculations the following value, valid for all grades of steel may be taken:

$$c_s = 520 \qquad [J/kg\,^{\circ}C]$$

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3.1 Fig. c.6 shows the standard fire curve [1]

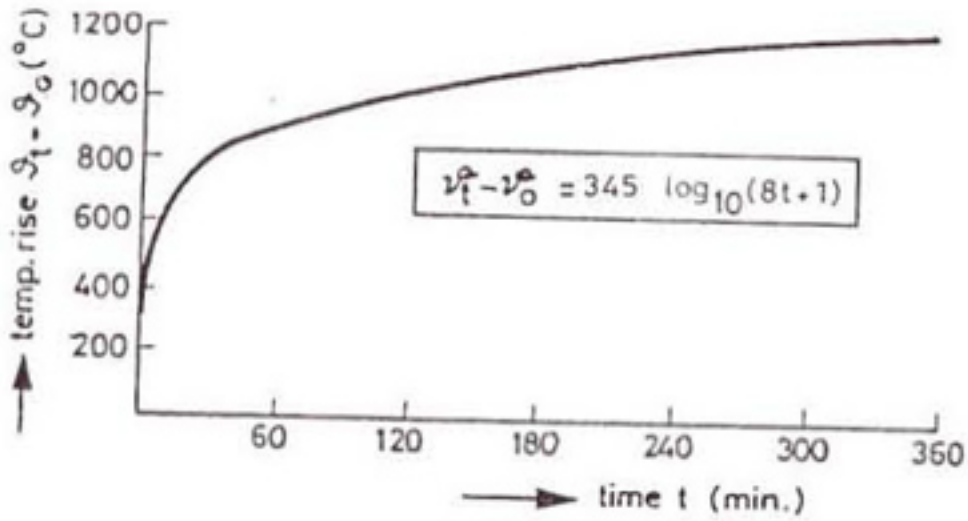

Fig. c-6: The standard fire curve.

3.2.1 The steel is assumed to offer no resistance to heat flow and therefore to be at a uniform temperature. The insulating material is assumed to have negligible thermal capacity and therefore to have a linear temperature gradient between the fire exposed surface and the inner surface next to the steel [11], [12]. The resistance to heat flow between the inner surface of the insulating material and the steel is assumed to be negligible. Heat flow is therefore calculated as one dimensional with a temperature distribution as illustrated in Fig. c.7.

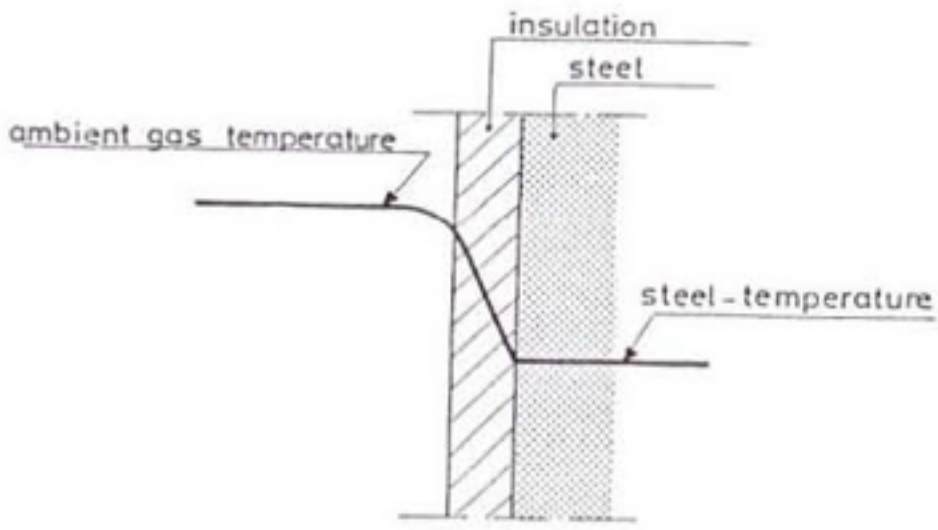

Fig. c-7: Temperature distribution in insulation material and steel.

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Art. 3 STEEL TEMPERATURE DEVELOPMENT UNDER STANDARD FIRE CONDITIONS

3.1 Ambient gas temperature.

The ambient gas temperature used in the calculations is the same as that specified in ISO 834 and given by:

$$(\vartheta_t - \vartheta_o) = 345 \log_{10} (8t + 1)$$

where:

t	= time	[min]
ϑ_t	= furnace temperature at time t	[°C]
ϑ_o	= furnace temperature at time t = 0	[°C]

3.2 Heat flow characteristics.

3.2.1 Basic equations.

3.2.1.1

The heat flow transmitted from the fire compartment to unit length of the steel member is calculated using the follwing equation:

$$\dot{Q} = K.F. (\vartheta_t - \vartheta_s) \qquad [W/m]$$

where:

$\dot{Q}$	= heat flow	[W/m]
K	= coefficient of total heat transfer	[W/m² °C]
F	= surface area of the member per unit length exposed to heating	[m²/m]
ϑ_t	= ambient gas temperature at time t	[°C]
ϑ_s	= temperature of the steel member	[°C]

3.2.1.2

The coefficient of total heat transfer K has three components and is given by :

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3.2.2.2

The recommended value of α_c is based on experimental investigations of standard fire exposure as well as of natural fire exposure.

The recommended value of α_r is based on the Stefan-Boltzmann law of radiation. The value ε_r will depend on the type of fire and the position of the exposed member. The design value of ε_r however, is an approximation which gives a conservative solution. A more accurate evaluation of ε_r is given in [6].

For heat transfer to members outside the facade see [7], [8].

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$$K = \frac{1}{\frac{1}{\alpha_c + \alpha_r} + \frac{d_i}{\lambda_i}} \quad [W/m^2\ {}^oC]$$

where:

α_c = coefficient of heat transfer due to convection from the fire to the exposed surface of the member [$W/m^2\ {}^oC$]

α_r = coefficient of heat transfer due to radiation from the fire to the exposed surface of the member [$W/m^2\ {}^oC$]

λ_i = thermal conductivity of the insulation material [$W/m\ {}^oC$]

d_i = thickness of the insulation [m]

3.2.2 Heat transfer to members without insulation.

3.2.2.1

For non-insulated members the equation for heat flow given in 3.2.1 reduces to:

$$\dot{Q} = (\alpha_c + \alpha_r) \cdot F \cdot (\vartheta_t - \vartheta_s)$$

3.2.2.2

The values of α_c and α_r are as follows:

$$\alpha_c = 25 \quad [W/m^2\ {}^oC]$$

$$\alpha_r = \frac{5.77\ \varepsilon_r}{\vartheta_t - \vartheta_s} \left[\left(\frac{\vartheta_t + 273}{100} \right)^4 - \left(\frac{\vartheta_s + 273}{100} \right)^4 \right] [W/m^2\ {}^oC]$$

where:

ε_r = resultant émissivity of the flames, combustion gases and exposed surface.

For the meaning of the other symbols see 3.2.1.

In calculations the following value for the resultant emissivity may be taken: $\varepsilon_r = 0,5$.

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3.2.3.1

In practice the value of $\frac{1}{\alpha_r + \alpha_c}$ is small compared with the value of $\frac{di}{\lambda i}$. Thus, for insulated members the coefficient of total heat transfer (= K, see 3.2.1.2) reduces to : $K = \frac{\lambda_i}{d_i}$
For very small values of di/λi this simplification may lead to rather conservative solutions. A more precise analysis, accounting for the effect of α_r and α_c on the heat flow is then recommended.

3.2.3.2

It is emphasized that the value of λ_i is not identical with the conventional value of the thermal conductivity as given in the handbooks on heat transfer.
In general λ_i will not only depend om the temperature of the insulation but also on the deformation of the structural element on which the insulation is attached, where cracks and/or chinks may occur in the insulation material.
Type of material, thickness and heating rate play an important role in this respect. For these reasons it is preferable to deduce an effective value of λ_i from the results of fire tests.

To allow for the thermal effects, small scale experiments with unloaded test specimens exposed to standard fire conditions will generally be sufficient. From such tests, for practical values of d_i, the value of λ_i can be determined over a range of temperatures of the insulation material by using the equation described in 3.4.
To cover the effect of the mechanical behaviour, at least one test on a loaded, fire exposed large scale member must be performed. The test load is chosen as the design load at service state.

The range of λ_i - values measured in the small scale tests may then need to be adjusted in order to agree with the result of the loaded large scale test.

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3.2.3 Heat transfer to insulated members.

3.2.3.1.

For insulated members the equation for heat flow given in 3.2.1 reduces to:

$$\dot{Q} = \frac{\lambda_i}{d_i} F \, (\vartheta_t - \vartheta_s)$$

For the meaning of the symbols see 3.2.1.

3.2.3.2

The value of λ_i should be determined experimentally as a function of the mean temperature of the insulation material, taking into account the influence of the thermal and mechanical behaviour of the insulation material under standard fire conditions.

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For a comprehensive description of possible testing procedures, see [6], [9], [10] and [14].

3.2.3.3

As explained under 3.2.3.2, λ_i will normally vary with the temperature of the insulation material. For practical purposes a mean value, selected for a given temperature range, may be used. Since the mean temperature of the insulation material will be a function of the ambient gas temperature as well as the steel temperature, the mean value of λ_i to be taken into account will depend on the time the structure is required to withstand the fire exposure and on the critical steel temperature conditions.

For some simplifying and ready-to-use information see [6] and [10].

3.2.3.4

Some single values of λ_i are given in table c.1. See also [6], [13]. It is emphasized that these are approximate and give conservative results.

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3.2.3.3

A mean value of λ_i, selected for a given temperature range, may be used in calculations.

3.2.3.4

Where no detailed information is available and only an approximate answer is required, the calculation may be based on a single value of λ_i, assumed to be valid for the whole temperature range.

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Material	Density ρ_i kg/m^3	Thermal conductivity λ_i W/m^{o}C
Sprayed mineral fibre	250 - 350	0.10
Perlite or vermiculite plates	300 - 800	0.15
Asbestos silicate sheets	800	0.15
Fibre silicate sheets	450 - 900	0.15
Gypsum slabs	800	0.20
Mineral wool slabs	120 - 150	0.25
Cellular concrete	600	0.30
Cellular concrete	1000	0.45
Cellular concrete	1300	0.65
Light weight concrete	1600	0.80
Clay brick, and lime brick	2000	1.20
Normal weight concrete (mainly amorphous aggregate)	2200	1.30
Normal weight concrete (mainly crystalline aggregate)	2200	1.70

Table c-1: Approximate values for the thermal conductivity of various insulation materials under fire conditions.

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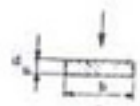
3.3.1 The equation is based on the assumption of quasi-stationary, one dimensional heat transfer, with the steel considered as a heat sink, in which the heat supplied is instantly distributed to give a uniform temperature.

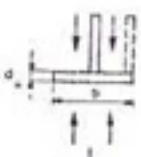
The temperature development in the member is strongly influenced by its geometry, represented by the factor F/V. This factor is called: 'the section factor'. Table c.2 gives some design values of the section factor.



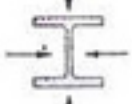
a) flat steel exposed to fire on all sides if $d_s << b$ $\quad F/V = \frac{2b + 2d_s}{b \cdot d_s} \approx \frac{2}{d_s}$

b) folded flat steel or angular steel exposed to fire on all sides $\quad F/V = \frac{2\,(b_1 + b_2)}{d_s\,(b_1 + b_2)} = \frac{2}{d_s}$

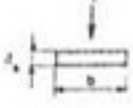
(applies to all open sections with uniform wall thickness)



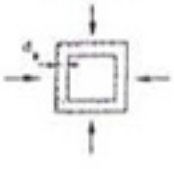
c) flange of an I-beam or channel section exposed to fire on all sides $\quad F/V = \frac{2b + 2d_s}{b \cdot d_s} \approx \frac{2}{d_s}$



d) open section exposed to fire on all sides $\quad F/V = \frac{\text{perimeter}}{\text{steel cross section}}$

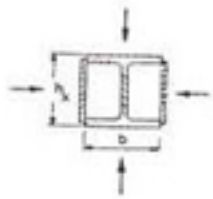


e) flat steel exposed to fire on three sides $\quad F/V = \frac{b + 2d_s}{b \cdot d_s} \approx \frac{1}{d_s}$

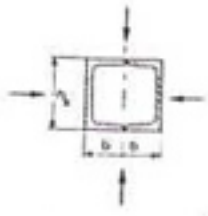


f) hollow section (with uniform wall thickness also corner welded section) exposed on all sides $\quad F/V = \frac{1}{d_s}$

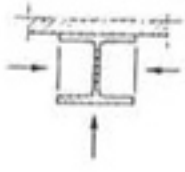
g) tube exposed to fire on all sides $\quad F/V = \frac{1}{d_s}$



h) I-section with box-reinforcement exposed to all sides $\quad F/V = \frac{2(b + h)}{\text{steel cross section}}$



i) box-type welded section exposed to fire on all sides $\quad F/V = \frac{2h + 4b}{\text{steel cross section}}$



k) open section exposed to fire on three sides $\quad F/V = \frac{\text{surface exposed to fire}}{\text{steel cross section.}}$

Table c-2: Section factor $F/_V$ for non insulated steel members.

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3.3 Calculation of the temperature development in non-insulated members.

3.3.1 Calculation rule.

The calculation of the temperature increase $\Delta\vartheta_s$ of a non-insulated member during a time interval Δt is based on the following, simplified equation:

$$\Delta\vartheta_s = \frac{\alpha}{c_s\,\rho_s} \cdot \frac{F}{V} \cdot (\vartheta_t - \vartheta_s) \cdot \Delta t \qquad [^{\circ}C]$$

in which :

α	$= \alpha_c + \alpha_r$ = coefficient of heat transfer in accordance with 3.2.2.	$[W/m^2\ ^{\circ}C]$
c_s	= specific heat of steel in accordance with 2.5	$[J/kg\ ^{\circ}C]$
ρ_s	= density of steel in accordance with 2.2	$[kg/m^3]$
F	= surface area of the member per unit length exposed to the fire	$[m^2/m]$
V	= volume of the member per unit length	$[m^3/m]$
ϑ_t	= ambient gas temperature during the time interval Δt	$[^{\circ}C]$
ϑ_s	= steel temperature during the time interval Δt	$[^{\circ}C]$
Δt	= time interval	[sec]

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For convergence, the value of Δt should have an upper limit as follows:

$$\Delta t < \frac{2.5 \times 10^4}{F/V}$$

where Δt is expressed in seconds and F/V in m^{-1}.

Table c.3 gives the temperature development in a steel member as a function of the section factor for $\varepsilon_r = 0.5$ and c_s -values varying with temperature as specified in the comments on 2.5.

$\varepsilon_r = 0.5$		section factor F/V [m^{-1}]								
time t [min]	gastemp. ϑ_i [°C]	10	20	30	50	100	150	200	250	300
0	20	20	20	20	20	20	20	20	20	20
15	739	95	164	226	333	514	612	664	691	705
30	842	215	367	484	637	781	814	824	829	831
45	902	343	552	682	808	877	888	893	895	896
60	945	467	700	814	897	930	936	939	940	941
75	979	580	809	898	949	967	972	974	975	976
90	1006	678	889	954	985	997	1000	1002	1003	1003

Remarks: - for section factors F/V <10 m^{-1} the assumption of a uniform temperature distribution across the section may be a gross approximation.
- For section factors F/V > 300 m^{-1} the calculated steel temperature is almost the same as the gas temperature.

Table c-3: Calculated temperature of non-insulated steel members as a function of the section factor F/V and time t ($\varepsilon_r = 0.5$, $\Delta t = 30$ secs).

3.4.1.1

If the recommended condition is met, it can be assumed that the heat capacity of the insulation does not affect the heat flow to the member significantly.

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3.4 Calculations of the temperature development in members with light weight insulation.

3.4.1 Scope

3.4.1.1

The insulation may be considered light weight if the following condition is met:

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The values of c_i and to a small degree, also of ρ_i depend on the temperature of the insulation material.
Normally the calculations can be based on mean values selected for a practical temperature range. Some values are given in table c.4; these will give conservative solutions.

For more precise calculations see [6], [10] and [13].

Material	Density ρ_i kg/m^2	Specific heat c_i J/kg °C
Insulation materials on silicate basis (vermiculite, perlite, mineral wool with bonding material)	250-900	1100
Gypsum boards	800	1700
Insulation materials with concrete basis, clay brick	600-2200	1200

Table c-4: Approximate values for the specific heat c_i and the density ρ_i of isulation materials under fire conditions.

3.4.1.2

a. Dry materials.

The heat flow in lightly insulated members using dry insulation materials is essentially governed by the thermal conductivity of the insulation material, represented by λ_i, and the steel temperature can be calculated as indicated in 3.4.2.

b. Moist materials.

Materials often contain water and are then termed moist. Free water in the pores will evaporate at

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$$c_s \cdot \rho_s \cdot V \geq 2\, c_i \cdot \rho_i \cdot d_i \cdot F_i$$

where :

c_s	= specific heat of steel in accordance with 2.5	[J/kg °C]
ρ_s	= density of steel in accordance with 2.2	[kg/m^3]
V	= volume of the member per unit length	[m^3/m]
c_i	= specific heat of the insulation material	[J/kg °C]
ρ_i	= density of the insulation material	[kg/m^3]
d_i	= thickness of the insulation material	[m]
F_i	= area of inner surface of the insulation material per unit length of the steel member	[m^2/m]

If the condition is not met, see 3.5.

3.4.1.2

In calculating the temperature of fire exposed insulated steel members the following types of insulation material should be considered separately:

- dry materials
- moist materials
- intumescent materials

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about 100°C.* Because water has a large latent heat of evaporation, most of the heat supplied to the material at about 100°C will be used for removal of the water and only a small amount is available for raising the temperature of the material. Consequently the temperature increase of a member insulated with moist material will be delayed, the length of the delay depending on the moisture content of the material (see Fig. c.8). The effect of this phenomenon on the temperature development in the steel section should be taken into account separately. See 3.4.3.

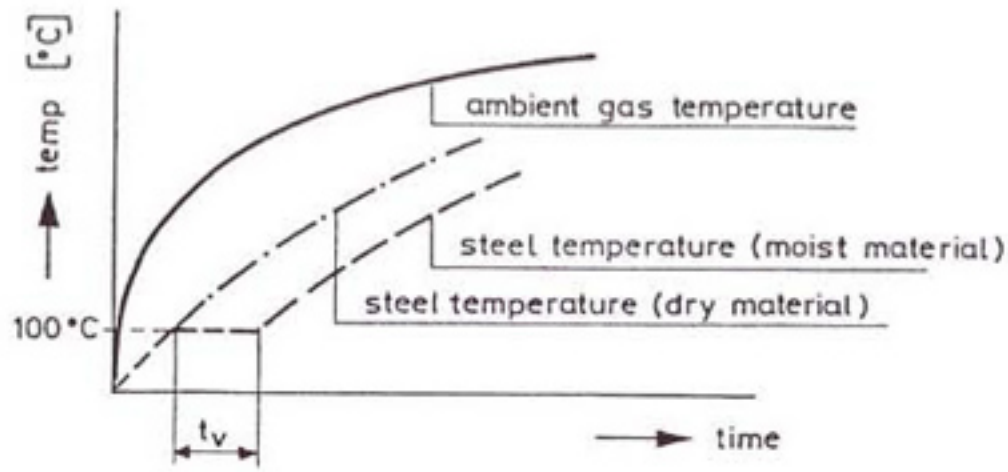

Fig. c-8: Delay time t_v in the temperature increase of a steel member insulated by a moist insulation material.

* For a material like plaster the situation is more complicated, since water of crystallisation is also present, which will be removed at temperatures higher than 100°C, depending on the rate of heating.

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c. Intumescent materials.

When an intumescent material is heated, it swells up to form a foam-like layer or meringue which provides thermal insulation.
The thickness of this layer may vary during fire exposure. Consequently it is not possible to define the thermal conductivity by λ_i.
Instead, the insulation capacity of the material can be expressed in terms of an effective d_i/λ_i value for each nominal thickness of application. See also 3.4.4.

3.4.2 The assumptions are the same as those in the comments on 3.3.1.

Design values of the section factor are given in table c-5.

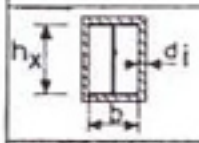
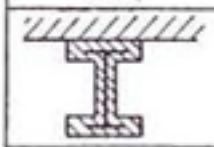
Description		section factor (F_i/V)
	contour encasement of uniform thickness	$\frac{\text{perimeter}}{\text{steel cross section}}$
	hollow encasement of uniform thickness	$\frac{2\ (b+h_x)}{\text{steel cross section}}$
	contour encasement of uniform thickness exposed to fire on three sides.	$\frac{\text{surface exposed to fire}}{\text{steel cross section}}$

Table c-5: Section factors F/V for insulated steel members.

In tables c.6[a] to c.6[d] the temperature development in a lightly insulated steel member is given as a function of the section factor and $\frac{d_i}{\lambda_i}$.
The specific heat c_s was varied with temperature as suggested in the comments to 2.5. The effect of the heat transfer by radiation and convection is, approximately, accounted for by taking:

$$K = \frac{1}{\frac{1}{\alpha_r + \alpha_c} + \frac{d_i}{\lambda_i}}$$

with α_r and α_c as recommended in 3.2.2.2. See also the comment to 3.2.3.1.

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3.4.2 Calculation rule for dry insulation material

Using dry insulation material, the calculation of the temperature increase $\Delta\vartheta_s$ of a lightly insulated steel member during a time interval Δt is based on the following, simplified equation:

$$\Delta\vartheta_s = \frac{\frac{\lambda_i}{d_i}}{c_s\,\rho_s} \cdot \frac{F_i}{V} \cdot (\vartheta_t - \vartheta_s) \cdot \Delta t \qquad [^{\circ}C]$$

where :

λ_i	= thermal conductivity of the insulation material in accordance with 3.2.3	[W/m °C]
d_i	= thickness of the insulation material	[m]
c_s	= specific heat of steel in accordance with 2.5	[J/kg °C]
ρ_s	= density of steel in accordance with 2.2	[kg/m^3]
F_i	= area of the inner surface of the insulation material per unit length of the steel member	[m^2/m]
V	= volume of the steel member per unit length	[m^3/m]
ϑ_t	= mean ambient gas temperature during the time interval Δt	[°C]
ϑ_s	= mean steel temperature during the time interval Δt	[°C]
Δt	= time interval	[sec]

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When a constant value of λ_i can be estimated, these tables enable a quick determination of the steel temperature.
The tables can also be used to estimate λ_i, from test results. Used in this way $\frac{d_i}{\lambda_i}$ includes any effects of an initial moisture content.

$d_i/\lambda_i = 0{,}01$ $\left[\frac{m^2\,{}^oC}{w}\right]$		section factor F_i/V $[m^{-1}]$										
time t [min]	gastemp. ϑ_t [°C]	10	20	30	50	100	150	200	250	300	350	400
0	20	20	20	20	20	20	20	20	20	20	20	20
15	739	68	113	155	230	373	472	540	588	622	647	665
30	842	139	239	323	451	635	721	765	788	802	810	816
45	902	215	360	469	615	780	837	861	873	879	883	886
60	945	289	467	587	731	866	904	918	925	929	932	934
75	979	360	559	682	815	922	948	958	963	966	968	969
90	1006	426	637	758	878	963	982	989	993	995	997	998
105	1029	487	705	820	927	994	1009	1015	1018	1020	1021	1022
120	1049	542	762	871	965	1020	1032	1036	1039	1041	1042	1043

(a)

Table c-6: Calculated temperature of insulated steel members as a function of the section factor F_i/V, thickness and thermal conductivity of insulation material, $\frac{d_i}{\lambda_i}$, and time t (Δt = 30 secs).

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$d_i/\lambda_i = 0{,}05$ $\left[\frac{m^2\,°C}{W}\right]$		section factor F_i/V $[m^{-1}]$										
time t [min]	gastemp. ϑ_t [°C]	10	20	30	50	100	150	200	250	300	350	400
0	20	20	20	20	20	20	20	20	20	20	20	20
15	739	40	60	78	114	193	259	315	363	403	439	469
30	842	68	112	153	226	368	467	540	595	636	669	695
45	902	98	167	228	330	505	612	683	732	767	792	811
60	945	129	221	298	421	611	716	780	821	849	868	882
75	979	160	273	363	499	694	793	850	884	906	921	931
90	1006	191	322	423	568	762	853	902	931	948	960	968
105	1029	222	368	477	628	817	901	943	967	981	991	997
120	1049	251	411	527	680	864	940	977	997	1009	1016	1021

(b)

$d_i/\lambda_i = 0{,}10$ $\left[\frac{m^2\,°C}{W}\right]$		section factor F_i/V $[m^{-1}]$										
time t [min]	gastemp. ϑ_t [°C]	10	20	30	50	100	150	200	250	300	350	400
0	20	20	20	20	20	20	20	20	20	20	20	20
15	739	32	43	54	76	126	172	212	249	283	313	341
30	842	48	74	99	146	247	327	393	447	493	531	564
45	902	65	107	146	216	353	454	529	588	634	671	701
60	945	83	140	192	281	445	555	633	690	733	767	793
75	979	102	173	237	342	523	637	714	768	807	837	860
90	1006	120	206	279	398	591	705	779	829	865	891	910
105	1029	139	238	320	449	649	762	832	878	910	933	949
120	1049	158	268	359	496	700	810	876	919	947	967	981

Table c-6: continued. (c)

$d_i/\lambda_i = 0,2$ $\left[\frac{m^2\,{}^oC}{W}\right]$		section factor F_i/V $[m^{-1}]$										
time t [min]	gastemp. ϑ_t [^oC]	10	20	30	50	100	150	200	250	300	350	400
0	20	20	20	20	20	20	20	20	20	20	20	20
15	739	26	33	39	51	80	108	134	158	181	203	224
30	842	35	50	64	91	154	209	258	302	341	376	407
45	902	44	68	90	133	225	302	366	421	467	507	542
60	945	54	87	118	174	292	385	458	518	568	609	644
75	979	65	106	145	215	354	457	536	598	648	689	723
90	1006	75	125	172	253	410	521	603	666	715	755	787
105	1029	85	145	198	291	461	577	660	723	771	809	839
120	1049	96	164	224	326	508	627	711	772	818	854	882

(d)

$d_i/\lambda_i = 0,3$ $\left[\frac{m^2\,{}^oC}{W}\right]$		section factor F_i/V $[m^{-1}]$										
time t [min]	gastemp. ϑ_t [^oC]	10	20	30	50	100	150	200	250	300	350	400
0	20	20	20	20	20	20	20	20	20	20	20	20
15	739	24	29	33	41	62	82	101	119	136	153	169
30	842	30	40	50	70	115	157	195	230	262	292	320
45	902	37	53	69	99	168	229	282	329	371	408	442
60	945	44	66	88	129	220	296	360	415	463	504	540
75	979	51	80	108	159	269	358	430	490	540	583	620
90	1006	58	93	127	189	316	414	492	555	607	650	687
105	1029	65	107	147	218	360	466	547	612	664	707	743
120	1049	73	121	166	246	401	513	597	662	714	757	792

(e)

Table c-6: continued

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3.4.3 The approach is illustrated in Fig. c-8 (See comments to 3.4.1.2)

The design values of t_v are approximate but give conservative solutions. The use of table r-1 can be illustrated by the following example:

Assumptions:

A steel member is insulated by a light weight insulation material. Moisture content: p = 10% by weight; λ_i = 0,20 W/m °C; ρ_i = 500 kg/m^3.

Required:

The delay time t_v for an insulation thickness of d_i = 0,02 and 0,06 m.

Solution:

For d_i = 0,02 m it follows:

$$\frac{p \cdot \rho_i \cdot d_i^2}{\lambda_i \cdot \rho_w} \cdot 10^2 = \frac{10 \cdot 500\ (0{,}02)^2}{0{,}2 \cdot 1000} \cdot 10^2 = 1\ \frac{m^3\ {}^oC}{W}$$

From table r-1 : t_v = 1 minute

For d_i = 0.06, the parameter $\frac{p \cdot \rho_i \cdot d_i^2}{\lambda_i\ \rho_w} \cdot 10^2$ equals to 9 $\frac{m^3\ {}^oC}{W}$. The corresponding delay time is:

t_v = 17 minutes

Design values for the moisture content are given in table c-7.

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3.4.3 Calculation rule for moist insulation materials.

With moist insulation materials, the calculation of the temperature increase $\Delta\vartheta_s$ is based on an equation, similar to the one under 3.4.2 with the following modification:

At a steel temperature of 100°C, a delay in the steel temperature rise is calculated to allow for evaporation of moisture. The delay time t_v [min] may be determined by using table r-1 where :

p	= moisture content	[% by weight]
d_i	= insulation thickness	[m]
ρ_i	= density of the insulation material	[kg/m^3]
ρ_w	= density of water (= 1000)	[kg/m^3]
λ_i	= thermal conductivity of the insulation material as specified in 3.2.3	[W/m °C]

$\frac{p \cdot \rho_i \cdot d_i^2}{\lambda_i \cdot \rho_w} \times 10^2$ $\lvert m^3\ {}^oC/W \rvert$	4	8	12	16	20	24	28
t_v |min|	5	15	25	30	40	50	55

Table r-1: Design values for the delay time t_v

In calculating the temperature development, the delay in temperature rise due to the initial moisture content may be taken into account up to a moisture level corresponding to the hygroscopic equilibrium moisture at a temperature of 20°C and a relative humidity of 40%.

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Material	Density d_i [kg/m^3]	Hygroscopic equilibrium moisture content [p % by weight]
Sprayed mineral fibre	250-350	1,0
Perlite or vermiculite boards	300-800	15,0
Asbestos silicate sheets	800	5,0
Fibre silicate sheets	450-900	3,0-5,0
Gypsum boards	800	20,0
Mineral wool slabs	120-150	2,0
Cellular concrete	600	2,5
Cellular concrete	1000	2,5
Cellular concrete	1300	2,5
Light weight concrete	1600	2,5
Clay brick, sand lime brick	2000	0,2
Normal weight concrete (mainly amorphous aggregate)	2200	1,5
Normal weight concrete (mainly crystalline aggregate)	2200	1,5

Table c-7: Approximate values for the hygroscopic equilibrium moisture content of various insulation materials.

A more detailed analysis is given in [9], from which the design values have been derived.

3.4.4 Due to possible discontinuity in the behaviour of fire exposed intumescent materials, extrapolation should be used with caution. In the light of present knowledge it should not be assumed that an intumescent material would survive for any significant time beyond that already shown to be satisfactory by a test result. In addition, the insulation provided may not be directly proportional to the coating thickness.

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3.4.4 Calculation rule for intumescent materials.

Using intumescent materials as insulation material, the calculation of the temperature increase $\Delta\vartheta_s$ is based on an equation similar to the one given under 3.4.2, with the value of $\frac{d_i}{\lambda_i}$ derived directly from tests as described in 3.2.3.2.

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3.5 The thermal capacity of the light-weight insulation materials normally used today is very small compared with the thermal capacity of the steel member. In such cases the equations in 3.4 can be used.
If heavier insulation with greater thermal capacity is used, the quantity of heat stored in the insulation should be taken into account when the rise in temperature $\Delta\vartheta_s$ is being calculated. The effect is covered, approximately, by the equations specified in this section.

The use of heavy insulation material is exceptional nowadays since good insulation can be relatively easily achieved with light-weight materials.
For these reasons, the approach recommended for heavily insulated members is more approximate in that no differentiation is made between dry and moist materials. With respect to the thermal properties of the insulation material it will normally be sufficient to use mean values over the temperature range. (See 3.2.3.3 and 3.2.3.4).

If the thermal capacity of the heavy insulation material is not taken into account, the calculated steel temperature will be over-estimated. Thus, if the equations given in 3.4 are used, the result will be conservative.

3.5.1 See comments on 3.4.1.1.

3.5.2 See also [6].

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3.5 Calculation of the temperature development in members with heavy weight insulation.

3.5.1 Scope

If the condition in 3.4.1.1 is not met, the member is considered as heavily insulated.

3.5.2 Calculation rule.

The calculation of the temperature increase $\Delta\vartheta_s$ of a heavily insulated steel member during a time interval Δt may be based on the following, approximate equation.

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3.5.3. Since the equation specified in 3.5.2 gives rise to fairly complex calculations, an alternative, approximate method for determining the temperature development in a heavily insulated steel member is given.
It is emphasized that the method is to be used only for standard fire exposure. For a comprehensive discussion of this method see [15], [16].
The following example illustrates the method:

Assumptions

Steel members are insulated as shown in Fig. c.9. An

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$$\Delta\vartheta_s = \frac{\frac{\lambda_i}{d_i}}{c_s\,\rho_s} \cdot \frac{F_i}{V} \cdot \left[\frac{1}{1+\xi}\right] \cdot (\vartheta_t - \vartheta_s) \cdot \Delta t - \frac{\Delta\vartheta_t}{1+\frac{1}{\xi}} \quad [^oC]$$

and

$$\xi = \frac{c_i d_i \rho_i F_i}{2\, c_s \rho_s V}$$

where :

λ_i	= thermal conductivity of the insulation material as specified in 3.2.3	[W/m °C]
d_i	= thickness of the insulation material	[m]
c_s	= specific heat of steel in accordance with 2.5	[J/kg °C]
ρ_s	= density of steel in accordance with 2.2	[kg/m^3]
c_i	= specific heat of the insulation material	[J/kg °C]
ρ_i	= density of the insualtion material	[kg/m^3]
F_i	= area of inner surface of the insulation material per unit length	[m^3/m^2]
V	= volume of the member per unit length	[m^3/m]
ϑ_t	= mean ambient gastemperature during the time interval Δt	[°C]
ϑ_s	= mean steel temperature during the time interval Δt	[°C]
Δt	= time interval	[sec]
$\Delta\vartheta_t$	= increase of the ambient gastemperature during the time interval Δt	[°C]

3.5.3 Simplified rule

The temperature development in heavily insulated members may be approximated by a simpler calculation method, using the equation for $\Delta\vartheta_s$ as specified in 3.4.2. In this equation the thermal capacity of the steel ($c_s\,\rho_s\,V$) is then increased by one half of the thermal capacity of the insulation material ($\frac{c_i\,\rho_i\,d_i\,F_i}{2}$). It follows:

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insulation of 50 mm concrete is assumed. (d_i = 0,05 m). It is further given that:

c_i	= 1200	[J/kg °C]
c_s	= 520	[J/kg °C]
ρ_i	= 2200	[kg/m^3]
ρ_s	= 7850	[kg/m^3]
λ_i	= 1.7	[W/m °C]
$\frac{F_i}{V}$	= 100	[1/m]

Required:

Steel temperature after 1 hour standard fire exposure on all sides.

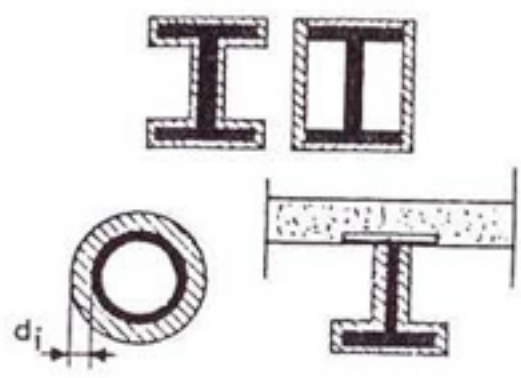

Fig. c-9: Insulated steel members.

Solution:

From the data given:

$$c_s \rho_s = 4.1 \times 10^6 < 2 \cdot c_i \cdot \rho_i \cdot d_i \cdot \frac{F_i}{V} = 26.4 \times 10^6$$

So, as defined in 3.5.1 the steel member is considered as heavily insulated.

The equation given in art. 3.5.3 can be written in the form:

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$$\Delta\vartheta_s = \frac{\dfrac{\lambda_i}{d_i}}{c_s \rho_s V + \dfrac{c_i \rho_i d_i F_i}{2}} \cdot F_i \cdot (\vartheta_t - \vartheta_s) \cdot \Delta t \qquad [^\circ C]$$

For the meaning of the symbols see 3.5.2.

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$$\Delta\vartheta_s = \frac{\frac{\lambda_i}{d_i}}{c_s\,\rho_s}\left[\frac{c_s\,\rho_s}{c_s\,\rho_s + \frac{c_i\,\rho_i\,d_i\,F_i}{2V}}\right]\frac{F_i}{V}\,(\vartheta_t - \vartheta_s)\,\Delta t$$

Apart from the factor between the square brackets this equation is identical with the equation for lightly insulated members. For this example the factor is given by:

$$\frac{c_s\,\rho_s}{c_s\,\rho_s + \frac{c_i\,\rho_i d_i F_i}{2V}} = \frac{4.1}{4.1 + \frac{26.4}{4}} \times \frac{10^6}{10^6} = 0.38$$

So, using the equation for lightly insulated members, the effect of the thermal capacity of the insulation can be taken into account by simply modifying the value of the section factor:

$$\left(\frac{F_i}{V}\right)_{mod.} = 0.38 \times 100 = 38$$

Using this value and $\frac{d_i}{\lambda_i} = \frac{0.05}{1.7} = 0.03$, gives by linear interpolation

for t = 1h : $\vartheta_s = 495^{\circ}C$ (cf. tables c-6^{a,b})

If the thermal capacity of the insulation is not taken into account than the equation in 3.4.2 gives

for t =1h : $\vartheta_s = 740^{\circ}C$ (cf. tables c-6^{a,b})

For other approximate methods by which the effect of the thermal capacity of the insulation can be taken into account, see [13].

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4.1 In many cases it will not be necessary to give a complete description of the deformation behaviour in order to meet the requirements. In such cases only the failure conditions have to be determined. The elementary plastic theory (limit state design) can then provide simple solutions.

In the ISO standard 834, the design load (dead load + characteristic live load) is considered to act on the structure throughout the fire This may lead to unduly conservative requirements, since it can be shown that the probability of the occurence of a fully developed fire while, at the same time, the full characteristic load is present, is only very small. This is especially true for live loads. The possibility of reduction of the live load during a fire should be considered [17] [18].

For reasons explained in the introduction, a theoretical analysis based on the specified parameters, gives lower critical temperatures than those measured in conventional fire tests.
From a practical point of view, this cannot be accepted. In the calculation method therefore, rules are given to assure consistency between both approaches (see 5.3 and 5.4).

4.2.1 Since the steel properties given in 2.3 are used, the effect of creep is included implicitly.

Methods are available to determine the deformation behaviour of statically determinate and indeterminate beams, explicitly

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Art. 4 STRUCTURAL BEHAVIOUR AT ELEVATED TEMPERATURE

4.1 Principles.

The required performance of a structural element is defined in terms of a particular failure criterion. This criterion may be either a limit on deformation or the capacity to support a given load.

In determining the structural behaviour of fire exposed steel members, the following principles of structural mechanics are taken into account:

- equilibrium
- Bernouilli's theorem*)
- compatibility theory, resp. yield criteria and associated flow rules.

In general the following parameters are taken into account:

- material properties as stipulated in 2
- ambient gas temperatures as stipulated in 3.1
- characteristics of the heat flow from the fire to the member as stipulated in 3.2
- steel temperature, calculated as described in 3.3, 3.4 or 3.5
- the mechanical load considered to act on the structure during fire; in confirmity with ISO standard 834 [1], this load is assumed equal to the design load (e.g. dead load + characteristic live load).
- static system.

4.2 Deformation behaviour

4.2.1 Basis of calculation

The analysis of deformation behaviour should be based on the steel properties given in 2.3 and 2.4.

---

*) Bernouilli's theorem states that initially plane sections remain plane during the deformation.

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taking into account the effect of creep [3], [6]. Such methods, giving a more accurate description of the deformation for temperatures over about 450°C, may be used as alternative to the method specified in 4.2.1.

4.3.1 and 4.3.2 Elementary plastic theory cannot be used where instability influences the load bearing capacity significantly.

4.4 The specified buckling curves at elevated temperatures are derived from the ECCS buckling curves at roomtemperature [24]. The latter curves are given in Fig. c-10 in non-dimensional form.

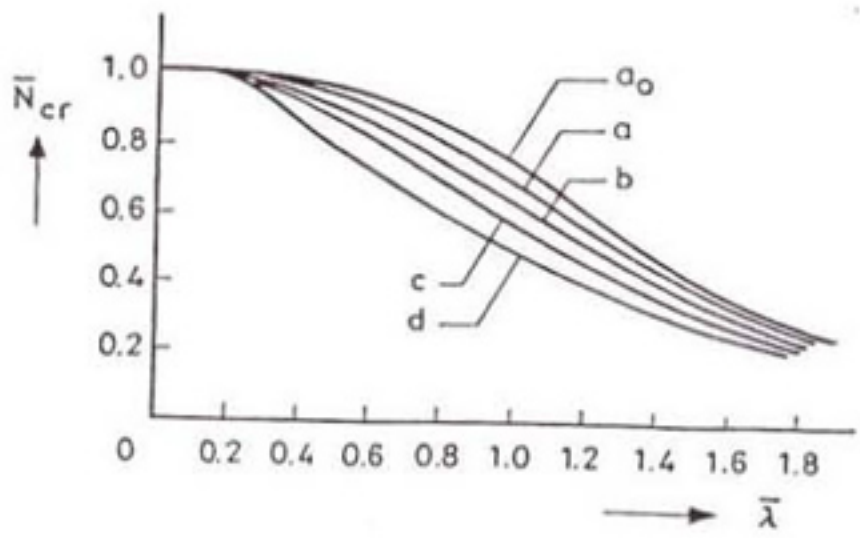

Fig. c-10: ECCS buckling curves at room temperature.

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4.3 Ultimate load bearing capacity

4.3.1 Basis of calculation

If elementary plastic theory can be used at normal temperatures, it can also be used to calculate the load bearing capacity at elevated temperatures.

4.3.2 Additional rule

If the condition given in 4.3.1 is not met, the reliability of the method of calculation must be demonstrated explicitly (see however also 4.4).

4.4 Buckling curves at elevated temperatures.

In calculations, the dimensionless buckling curves, valid for all the specified steel grades, are given by the following approximate equation:

$$\bar{N}_{cr\vartheta} = \psi \left[\frac{1 + 0.489\ (\bar{\lambda} - 0.2) + \bar{\lambda}^{\,2}}{2\bar{\lambda}^{\,2}} - \frac{1}{2\bar{\lambda}^{\,2}} \sqrt{\left[1+0.489(\bar{\lambda}-0.2) + \bar{\lambda}^2\right]^2 - 4\bar{\lambda}^2} \right]$$

for $\bar{\lambda} > 0.2$

and

$\bar{N}_{cr\vartheta} = \psi$ for $\bar{\lambda} \leq 0.2$

where:

$\bar{N}_{cr\vartheta} = \dfrac{\sigma_{cr\vartheta}}{\sigma_{y20}}$ = dimensionless buckling load at elevated temperature.

$\psi = \dfrac{\sigma_{y\vartheta}}{\sigma_{y20}}$ = temperature dependent coefficient given in Fig. r-2 (see also 2.3.3).

$\bar{\lambda} = \dfrac{\lambda}{\pi\sqrt{\dfrac{E}{\sigma_{y20}}}}$ = modified slenderness ratio.

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In Fig. c-10 the dimensionless buckling load $\overline{N}_{cr}$ is defined as the ratio of the buckling stress σ_{cr20} to the nominal value of the yield stress at room temperature σ_{y20} and is given as a function of the modified slenderness ratio $\overline{\lambda}$. The choice of curve a to d depends on the type of cross section and the manufacturing process. These curves can be described by the following general expression:

$$\overline{N}_{cr} = \frac{\sigma_{cr20}}{\sigma_{y20}} = \frac{1+\alpha\ (\overline{\lambda}-0.2)+\overline{\lambda}^2}{2\overline{\lambda}^2} - \frac{1}{2\overline{\lambda}^2}\sqrt{\left[1+\alpha\ (\overline{\lambda}-0.2)+\overline{\lambda}^2\right]^2-4\overline{\lambda}^2}$$

The coefficient α (dimensionless) varies with the selected curve and values are given in table c-8.

ECCS buckling curve	a_o	a	b	c	d
α	0.125	0.206	0.339	0.489	0.756

Table c-8: Buckling coefficient α for various types of cross section.

At room temperature -and for a certain value of α- the dimensionless buckling load is thus a function of three parameters, viz.: the yield stress $\sigma_{y,20}$, the Youngs modulus E and the slenderness ratio λ.

Deriving the specified buckling curves at elevated temperatures, it has been assumed that the equation is similar to the one above for room temperature i.e.:

$$\frac{\sigma_{cr\vartheta}}{\sigma_{y\vartheta}} = \frac{1+\alpha\ (\overline{\lambda}_\vartheta-0.2)+\overline{\lambda}_\vartheta^2}{2\overline{\lambda}_\vartheta^2} - \frac{1}{2\ \overline{\lambda}_\vartheta^2}\sqrt{\left[1+\alpha\ (\overline{\lambda}_\vartheta-0.2)+\overline{\lambda}_\vartheta^2\right]^2-4\overline{\lambda}_\vartheta^2}$$

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with

$\sigma_{cr\vartheta}$ = buckling stress at elevated temperature

σ_{y20} = nominal yield stress of the steel at room temperature

$\sigma_{y\vartheta}$ = effective yield stress of the steel at elevated temperature

λ = slenderness ratio at room temperature

E = modulus of elasticity of steel at room temperature ($= 2.1 \times 10^{+5}$ N/mm^2)

In Fig. r-4 the dimensionless buckling curves are presented graphically.

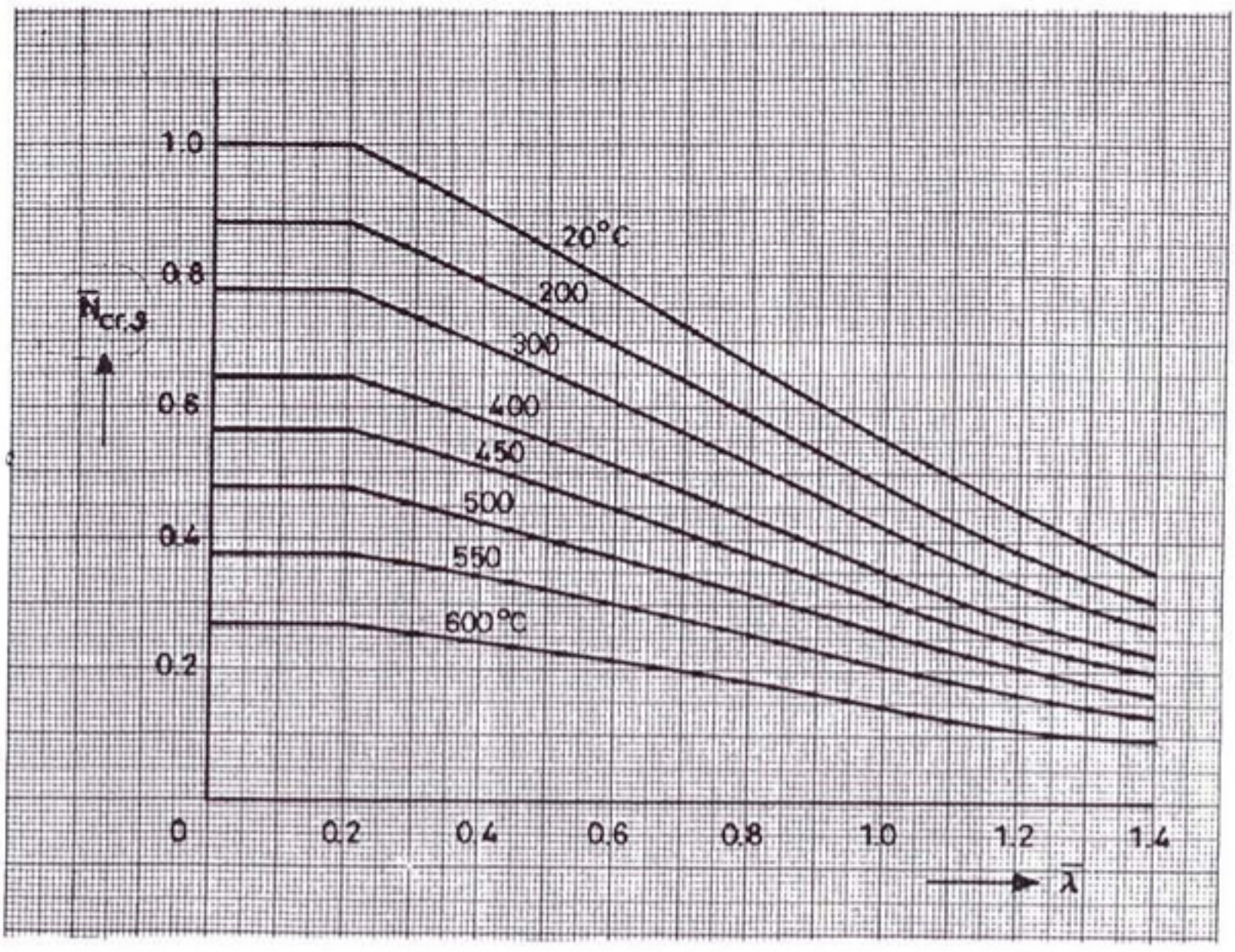

Fig. r-4: Dimensionless buckling curves at elevated temperatures.

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in which the suffix ϑ means that the value of the parameter in question applies to a steel temperature ϑ_s. As a hypothesis, it is further assumed that the effect of ϑ_s on the value of $\bar{\lambda}_\vartheta$ can be quantified by substituting temperature dependent values for the yield stress ($\sigma_{y\vartheta}$) and the modulus of elasticity ($E_{t\vartheta}$) in the equation for the modified slenderness ratio $\bar{\lambda}$, viz.:

$$\bar{\lambda}_\vartheta = \frac{\lambda}{\pi\sqrt{\frac{E_{t\vartheta}}{\sigma_{y\vartheta}}}}$$

or

$$\bar{\lambda}_\vartheta = \bar{\lambda}\sqrt{\frac{\frac{\sigma_{y\vartheta}}{\sigma_{y20}}}{\frac{E_{t\vartheta}}{E}}}$$

Since both $\frac{E_{t\vartheta}}{E}$ and $\frac{\sigma_{y\vartheta}}{\sigma_{y20}}$ vary in a similar way with the steel temperature (cf. art. 2.3.3 and 2.3.4), it follows, as a good approximation:

$$\bar{\lambda}_\vartheta \approx \bar{\lambda}$$

Consequently:

$$\frac{\sigma_{cr\vartheta}}{\sigma_{y\vartheta}} = \frac{\sigma_{cr20}}{\sigma_{y20}} = \bar{N}_{cr}$$

It follows that the buckling load $\bar{N}_{cr\vartheta}$ $(= \frac{\sigma_{cr\vartheta}}{\sigma_{y20}})$ at elevated temperature is given by:

$$\bar{N}_{cr\vartheta} = \frac{\sigma_{y\vartheta}}{\sigma_{y20}}\ \bar{N}_{cr} = \Psi\bar{N}_{cr}$$

The subsequent equation for the buckling curves at elevated temperature is then obtained:

$$\bar{N}_{cr\vartheta} = \Psi\left|\frac{1+\alpha\ (\bar{\lambda}-0.2)+\bar{\lambda}^2}{2\bar{\lambda}^2} - \frac{1}{2\bar{\lambda}^2}\sqrt{\left[1+\alpha\ (\bar{\lambda}-0.2)+\bar{\lambda}^2\right]^2 - 4\bar{\lambda}^2}\right|$$

It is emphasized that the buckling curves in this way are derived for centrical loading and bifurcation buckling. They do not

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take into account other types of instability and axial restraint. Therefore, the room temperature 'c' curve of Fig. c-10 has been used as the basis for the elevated temperature buckling curves recommended in 4.4. This corresponds to a value of 0.489 for α.

The recommended elevated temperature buckling curves are in good agreement with the results of recent full scale tests carried out in Belgium and Denmark [25], [26]. It should be noted however, that at the moment it does not seem possible to carry out an analytical determination of the buckling curves based on the stress strain relationships specified in 2.3.
The reason is that there is insufficient understanding of the other factors which influence the stability of columns at elevated temperatures, such as the unloading characteristics of steel and the effect of residual stresses and geometrical imperfections.

4.5 Since neither in practice nor during tests [19] has failure been caused primarily by the behaviour of connections, it has been decided to omit specific design rules for connections exposed to fire.

If bolted connections are used for insulated steel members the bolts should be as well protected as the member. This will normally lead to a local increase of insulation thickness as illustrated in Fig.c-11 [19]

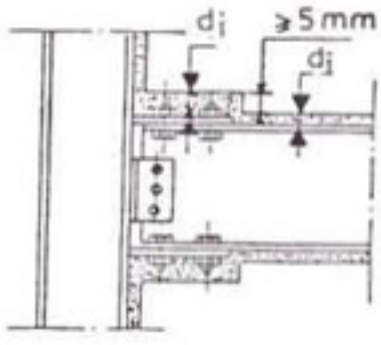

Fig. c-11: Increase of insulation thickness at a bolted connection.

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4.5 Connections.

No analysis of the behaviour of the connections is needed if the following conditions are met:

- the design at room temperature is based on authorized rules
- the insulation covering any fastener is at least as thick as the insulation on the adjacent steel member.

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5.1 The load considered to act on the structure during a fire is discussed in the comments on 4.1. It should be noted that to conform with the ISO 834 standard fire resistance test, the load has to be equal to the design load (dead load + characteristic live load).

The limit states given here correspond to the limit states used in a standard fire resistance test, as may be elucidated as follows.

Strictly speaking, only the limit state of failure is significant and corresponds to an infinitely high rate of deformation. During tests however, it will often be difficult, or even impossible, to decide when the deformation rate is "infinitely high" because of difficulties with the loading equipment or a limited depth of the furnace. However, a maximum permissible deformation rate can be defined which will be on the safe side. The limiting rate frequently used for beams is:

$$\frac{d\delta}{dt} \geq \frac{\ell^2}{15h_x} \qquad [mm/h]$$

where:

δ	= max. deflection	[mm]
ℓ	= span	[mm]
h_x	= height of the steel section	[mm]
t	= time	[h]

The limit state of deformation has been added to avoid impractical situations in which actual failure occurs at extreme deformation. Under such circumstances it must be questioned whether adjacent structural elements can still fulfil their function and/or spread of fire to other rooms can still be contained. Obviously the actual value by which the deformation is limited has an arbitrary character. More generally the recommended deformation criterion can be expressed as:

$$\delta \geq \frac{\ell^2}{800\ h_x} \qquad [mm]$$

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Art. 5. LIMIT STATES AND CRITICAL TEMPERATURE.

5.1 Limit states.

5.1.1 Definition.

A limit state is considered to be attained if it may be assumed that certain, well described functions can no longer be fulfilled by the load bearing element or structural assembly.

5.1.2 Limit state of failure.

The limit state of failure is reached when the load bearing capacity of the structural load bearing element or structural assembly decreases to the level corresponding to the load considered to act on the structure under fire conditions.

5.1.3 Limit state of deformation.

5.1.3.1

The limit state of deformation should be considered to be attained if the maximum deflection exceeds $\frac{1}{30}$ th of the characteristic dimension of the structural member under consideration (see Fig. r-5).

5.1.3.2

If it can be demonstrated explicitly that a limit to the deformation is not justified on functional grounds this criterion may be dispensed with.

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where δ, ℓ and h_x are already defined above. For a practical range of the span-to-height ratio (that is to say $15 > \frac{\ell}{h_x} > 25$) this general criterion tends to $\frac{\delta}{\ell} \geq \frac{1}{30}$ (cf. 5.1.3.1).

The combination of the criteria:

$$\frac{d\delta}{dt} \geq \frac{\ell^2}{15h_x} \qquad [mm/h]$$

and

$$\delta \geq \frac{\ell^2}{800h_x} \qquad [mm]$$

is known as the Robertson-Ryan criterion and is used for beams and slabs in fire tests. For columns such criteria are not relevant.

In an analytical approach, confined to structural steelwork, it is more convenient to use the criteria as given in 5.1.2 and 5.1.3.

If the analysis is based on a determination of the complete deformation behaviour (4.2), both the recommended failure and deformation criterion can easily be checked.
If the analysis is based on a determination of the ultimate load bearing capacity (4.3), the deformation will generally be unknown. Consequently the deformation cannot be checked using such analysis. However, for practical values of the height-to-span ratio of structural steel members and if no membrane forces are involved, it is not necessary to check the deformation criterion, since in these circumstances it can be shown that the collapse criterion and the deformation criterion lead to practically the same fire resistance.

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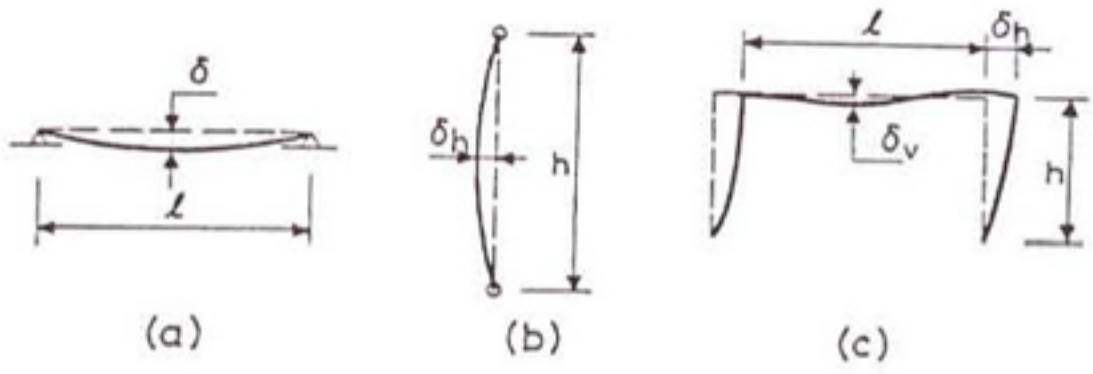

characteristic dimension: (a) = ℓ

(b) = h

(c) = h (lateral deflection)

ℓ (vertical deflection)

Fig. r-5: 'Characteristic dimensions' referred to in 5.1.2.

5.2 Definition of the critical temperature.

The critical temperature of a load bearing steel element or structural assembly is that temperature at which the limit state of failure is expected to be attained.

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5.3.1 to 5.3.3

The rules given are based on the assumption of a uniformly distributed temperature over the length and the cross-section of the beam. Moreover, the nominal value of the yield stress is adopted. As explained in the Introduction, calculations based on such assumptions give substantially lower critical temperatures than those measured in conventional fire tests. See also the comments on 4.1. Since this discrepancy is unacceptable in practice, a load multiplier κ is introduced, which allows for, approximately, the effect of both a non-uniform temperature distribution and the discrepancy between nominal and actual yield stress. This is discussed more fully in [22], which is the source of the recommended values of κ.

The factor Θ will be equal to one for statically determinate beams, since it can be assumed that there will be no redistribution of moments. For statically indeterminate beams however: $\Theta \geq 1$.

The following example illustrates the determination of critical temperature.

Assumptions.

Consider a uniformly loaded beam as shown in Fig. c-12.

Required.

a. critical temperature of the beam if the design at room temperature is based on the theory of elasticity.

b. critical temperature of the beam if the design at room temperature is based on the theory of plasticity.

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5.3 Critical temperature of beams.

5.3.1 Basis of calculation.

If the analysis can be based on a determination of the ultimate load bearing capacity according to 4.3.1, the critical temperature of beams is derived using the temperature dependent coefficient Ψ $(= \frac{\sigma_{y\vartheta}}{\sigma_{y20}})$ according to the following rules:

5.3.2 Calculation rules.

5.3.2.1

If the design at room temperature is based on the theory of elasticity, the limit state of failure is given by the following equation:

$$\Psi = \frac{1}{\Theta} \cdot \frac{W}{Z} \cdot \frac{\kappa . q^*}{q_e} \quad (= \frac{\sigma_{y\vartheta}}{\sigma_{y20}})$$

where:

Ψ = temperature dependent coefficient specified in 2.3.3 and given in Fig. r-2

Θ = factor ≥ 1, representing the plastic reserve of the beam due to redistribution of moments

W = elastic section modulus

Z = plastic section modulus

q^* = load considered to act on the beam during fire and specified in 4.1

q_e = ultimate elastic load at room temperature

κ = load multiplier in accordance with 5.3.3

$\sigma_{y\vartheta}$ = effective yield stress at elevated temperature

σ_{y20} = nominal yield stress at room temperature

5.3.2.2

If the design at room temperature is based on the theory of plasticity, the equation given in 5.3.2.1 reduces to:

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Comments

static system :

collapse mechanism :

moment distribution at failure :

elastic moment distribution :

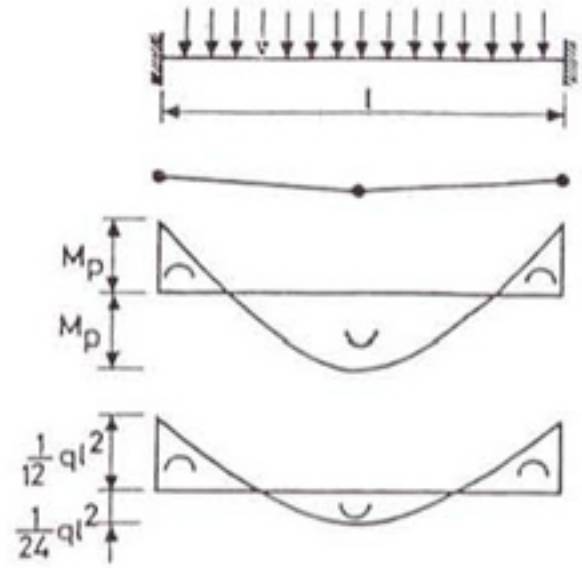

Fig. c-12: Structural behaviour of a uniformly loaded beam with built-in support conditions.

Solution :

ad a Design condition at room temperature : (theory of elasticity)

$$1/12\ q_e \ell^2 = W \cdot \sigma_{y20} \quad \text{----------(a)}$$

Failure condition at elevated temperature :

$$1/8\ q^* \ell^2 = 2Z \cdot \sigma_{y\vartheta} \quad \text{----------(b)}$$

Combining (a) and (b), and introducing the load multiplier κ :

$$\frac{\sigma_{y\vartheta}}{\sigma_{y20}} = \frac{\kappa}{4/3} \cdot \frac{W}{Z} \cdot \frac{q^*}{q_e}$$

Note that the factor $\frac{4}{3}$ is, in general, denoted as Θ. Thus in this example : $\Theta = \frac{4}{3} = 1{,}33$.

Taking $\frac{Z}{W} = 1.15$ then : $\frac{\sigma_{y\vartheta}}{\sigma_{y20}} = 0.65 \frac{q^*}{q_e} \cdot \kappa$

For a certain value of $\frac{q^*}{q_e}$, κ can be obtained from table r-2.

Thus, $\frac{\sigma_{y\vartheta}}{\sigma_{y20}} = \Psi$ can be calculated and the corresponding critical temperature can be obtained from Fig. r-2.

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$$\psi = \frac{\kappa q^*}{q_p} \quad \left(= \frac{\sigma_{y\vartheta}}{\sigma_{y20}}\right)$$

in which

q_p = the ultimate plastic load at room temperature.

5.3.3 Load multiplier κ for beams.

The load multiplier κ is a function of the load level and the static system. Design values of κ are given in table r-2.

	Load level $\frac{q^*}{q_e}$ resp. $\frac{q^*}{q_p}$								
	0,2	0,3	0,4	0,5	0,6	0,7	0,8	0,9	1,00
Statically determinated beam	0,80	0,83	0,85	0,88	0,90	0,93	0,95	0,98	1,00
Statically indeterminated beam with one redundancy	0,60	0,65	0,70	0,75	0,80	0,85	0,90	0,95	1,00
Statically indeterminated beam with two or more redundancies	0,40	0,48	0,55	0,63	0,70	0,78	0,85	0,93	1,00

Table r-2: Design values for the load multiplier κ.

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ad b Design condition at room temperature : (theory of plasticity)

$$1/8\ q_p\ \ell^2 = 2\ Z\ \sigma_{y,20} \quad \text{----------} (c)$$

Failure conditions at elevated temperature :

$$1/8\ q^*\ \ell^2 = 2\ Z\ \sigma_{y,\vartheta} \quad \text{----------} (d)$$

Combining (c) and (d) and introducing the load multiplier κ,

$$\frac{\sigma_{y,\vartheta}}{\sigma_{y,20}} = \kappa\ \frac{q^*}{q_p}$$

Table c-9 shows the value of θ in combination with the critical temperature for some practical examples. ($\frac{Z}{W} = 1.15$).

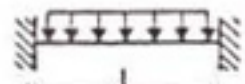
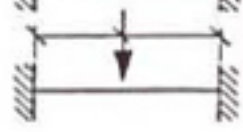
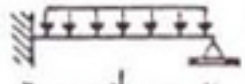
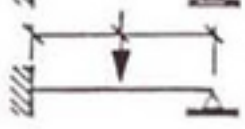
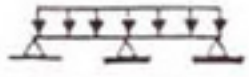
base of structural design at room temperature			factor $\frac{\kappa q^*}{q_e}$ resp. $\frac{\kappa q^*}{q_p}$				
		static system	0,3	0,4	0,5	0,6	0,7
	theory of plas-ticity	statically determined statically indeter-mined	585	540	490	430	360
	theory of elasticity	statically determined	605	565	525	475	425
		statically indeter-mined					
		$\Theta =$ 1,33 	640	605	575	545	510
		$\Theta =$ 1,0 	605	565	525	475	425
		$\Theta =$ 1,47 	650	615	590	560	535
		$\Theta =$ 1,12 	615	580	545	505	465
		$\Theta =$ 1,47 	650	615	590	560	535

Table c-9 : Critical temperature of beams

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5.4.1 It is common practice to specify the performance of a fire exposed column in terms of the result of a conventional fire resistance test, in which the test specimen is centricly loaded. It has been decided therefore to give approximate rules for calculation the critical temperature in these circumstances. Rules which implicitly take into account the statical system involved are given in Appendix II.
For further information on the elevated temperature buckling curves see the comments on 4.4.

5.4.2 Column failure is assumed to occur if the design load at elevated temperature ($= \kappa N^*$) equals the critical buckling load ($\overline{N}_{cr\vartheta} . A . \sigma_{y20}$).

i.e. $\kappa N^* = \overline{N}_{cr\vartheta} . A \sigma_{y20}$

From 4.4:

$$\overline{N}_{cr\vartheta} = \Psi \overline{N}_{cr}$$

Therefore:

$$\kappa . N^* = \Psi \overline{N}_{cr} . A \sigma_{y20}$$

With $N_{cr} = \overline{N}_{cr} . A \sigma_{y20}$, the specified calculation rule can be derived easily.

The following example illustrates the determination of the critical temperature:

Assumptions:

Consider a column in a braced frame, with the following specifications:

grade of steel : Fe 360
profile : HE 200 A

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5.4 Critical temperature of columns classified according to standard fire resistance test conditions of loading.

5.4.1 Basis of calculation.

The critical temperature of columns, under the loading conditions of the standard fire resistance test, may be based on the buckling curves specified in 4.4. Under these conditions the critical temperature is given by the temperature dependent coefficient Ψ (= $\frac{\sigma_{y\vartheta}}{\sigma_{y,20}}$) according to the following rule.

5.4.2 Calculation rule.

The limit state of failure is given by the equation

$$\Psi = \frac{\kappa \cdot N^*}{N_{cr}} \quad (= \frac{\sigma_{y\vartheta}}{\sigma_{y20}})$$

where:

Ψ = temperature dependent coefficient specified in 2.3.3 and given in Fig. r-2.

κ = load multiplier given in 5.4.3.

N^* = normal load considered to act on the column during fire and specified in 4.1.

N_{cr} = ECCS buckling load at room temperature corresponding to the so called c-curve(= $A.\sigma_{y20}.\overline{N}_{cr}$).

with:

A = cross section area of the column

σ_{y20} = nominal yield stress of the steel at room temperature.

$\overline{N}_{cr}$ = dimensionless buckling load at room temperature.

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From the design at room temperature:

λ = 100 (see however also 5.4.4)

N^* = 400 kN

Required:

The critical temperature of the column.

Solution:

Note that $\sigma_{y,20}$ = 235 N/mm^2 (grade of steel:Fe 360) and calculate $\bar{\lambda}$:

$$\bar{\lambda} = \frac{100}{\Pi\sqrt{\frac{2.1\times10^5}{235}}} = 1.07$$

Substitute this value of $\bar{\lambda}$ in the specified equation for the dimensionless buckling load at room temperature and obtain:

$$\bar{N}_{cr} = 0.50$$

With A = 5.38 x 10^3 mm^2 (HE 200 A) and σ_{y20} = 235 N/mm^2 :

$$N_{cr} = 5.38 \times 10^3 \times 235 \times 0.50 = 632 \text{ kN}$$

With N^* = 400 N and κ = 0.85 (see 5.4.3) it follows for Ψ :

$$\Psi = \frac{0.85 \times 400}{632} = 0.54$$

For Ψ = 0.54, Fig.r-2 gives a critical temperature of 460°C.
Note that this value of the critical temperature can also be obtained directly from the buckling curves given in Fig. r-4 for

$$\bar{N}_{cr\psi} = \frac{0.85 \times 400 \times 10^3}{5.38 \times 10^3 \times 235} = 0.269 \text{ and } \bar{\lambda} = 1.07.$$

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where:

$$\bar{N}_{cr} = \frac{1+0.489\ (\bar{\lambda}-0.2)\ +\ \bar{\lambda}^2}{2\ \bar{\lambda}^{\ 2}} - \frac{1}{2\ \bar{\lambda}^2}\sqrt{\left[1+0.489\,(\bar{\lambda}-0.2)+\bar{\lambda}^{\ 2}\right]^{2}\ -4\bar{\lambda}^{\ 2}}$$

for $\bar{\lambda} > 0.2$

or

$\bar{N}_{cr} = 1$ for $\bar{\lambda} \leq 0.2$

$\bar{\lambda}$ = modified slenderness ratio

$$= \frac{\lambda}{\Pi\sqrt{\frac{E}{\sigma_{y20}}}}$$

λ = slenderness ratio at room temperature, in accordance with 5.4.4

E = modulus of elasticity at room temperature (2.1×10^5 N/mm^2).

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Table c-10 gives the critical temperature as a function of the factor $\frac{\kappa N^*}{N_{cr}}$

$\frac{\kappa N^*}{N_{cr}}$	0.3	0.4	0.5	0.6	0.7
ϑ_{cr}	585	540	490	430	360

Table c-10: Critical temperature for columns classified according to standard fire resistance test conditions of loading.

5.4.3 The load multiplier κ varies with the load level and the static system [22], but because of the other approximations, a single value for κ has been adopted for calculating the critical temperature of columns.

5.4.4 The specified slenderness ratio depends on the fire resistance test procedure. In some countries columns are tested under pin ended support conditions, which means that the buckling length is equal to the system height of the column. In other countries there is one fully built-in and one pin ended support condition so that the buckling length equals 0.7 system height.

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5.4.3 Load multiplier for columns.

In calculations the load multiplier κ may be taken equal to 0.85.

5.4.4 Slenderness ratio.

In calculations the slenderness ratio λ should be the value, specified in the standard fire resistance test procedure.

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APPENDIX I: Tabulated values of stress strain relationships at elevated temperatures.

In tables A-1[a] to A-1[d] numerical information is provided on stress strain relationships at elevated temperatures for steel grades Fe 310, Fe 360, Fe 430 and Fe 510. For an explanation of these relationships see section 2.3.

The use of the tables may be illustrated by reviewing the stress strain curve for Fe 510 at 500°C (see table A-1[d]).

- effective yield stress : $\sigma_{y\vartheta}$ = 175.6 N/mm^2
- effective yield strain : $\varepsilon_{y\vartheta}$ = 0.544 %

This combination of stress and strain determines the point at which -for increasing strain- the curve is horizontal. For strains smaller than $\varepsilon_{y\vartheta}$, the corresponding stress can be read from the table. E.g. : for ε = 0.4 $\varepsilon_{y\vartheta}$ = 0.218 %, it follows : σ = 141.1 N/mm^2.

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Fe 310	Stress N/mm^2										
ϑ_s [°C] / $\varepsilon_{y\vartheta}$ [%] / strain ε [%]	20	200	250	275	300	350	400	450	500	550	600
	0.132	0.284	0.345	0.365	0.386	0.417	0.478	0.478	0.478	0.475	0.456
0.1 $\varepsilon_{y\vartheta}$	25.4	56.8	62.7	56.5	49.4	43.7	42.4	34.0	30.4	14.6	13.5
0.2 $\varepsilon_{y\vartheta}$	53.1	105.5	103.5	83.3	62.3	55.6	56.7	48.4	43.6	27.8	21.3
0.3 $\varepsilon_{y\vartheta}$	80.2	129.5	121.8	97.1	73.0	66.6	68.9	57.8	53.3	35.5	26.1
0.4 $\varepsilon_{y\vartheta}$	107.4	137.4	130.1	107.1	83.2	77.3	79.0	66.3	61.0	41.0	30.1
0.5 $\varepsilon_{y\vartheta}$	133.8	142.6	134.4	113.6	93.6	86.7	88.2	74.8	66.6	46.5	33.2
0.6 $\varepsilon_{y\vartheta}$	154.6	146.1	136.4	119.1	103.7	96.0	95.3	82.0	72.1	51.1	35.9
0.7 $\varepsilon_{y\vartheta}$	165.9	147.5	137.3	124.9	113.2	104.3	101.2	87.7	76.9	54.6	38.7
0.8 $\varepsilon_{y\vartheta}$	171.2	148.4	138.2	130.1	122.3	111.9	108.9	93.5	81.8	57.6	41.1
0.9 $\varepsilon_{y\vartheta}$	174.5	149.3	139.1	132.8	128.6	119.4	115.4	99.1	87.4	61.4	41.9
1.0 $\varepsilon_{y\vartheta}$	175.0	150.2	140.1	135.6	131.3	127.0	118.2	102.9	89.2	65.2	42.8

(a) Grade Fe 310

Table A-1: Stress strain relationships at elevated temperatures for various grades of steel ($\varepsilon_{y\vartheta}$ is the strain at which the effective yield stress is reached, see Fig. c-1).

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Fe 360	Stress N/mm^2										
ϑ_s [°C] / $\varepsilon_{y\vartheta}$ [%] / strain ε [%]	20	200	250	275	300	350	400	450	500	550	600
	0.160	0.310	0.370	0.390	0.410	0.440	0.500	0.500	0.500	0.500	0.500
0.1 $\varepsilon_{y\vartheta}$	33.0	62.0	72.0	75.0	78.0	73.0	70.0	58.0	50.0	30.0	18.0
0.2 $\varepsilon_{y\vartheta}$	66.0	124.0	134.0	120.0	102.0	93.0	90.0	78.0	68.0	46.0	32.0
0.3 $\varepsilon_{y\vartheta}$	99.0	171.0	164.0	139.0	115.0	105.0	104.0	89.0	79.0	56.0	39.0
0.4 $\varepsilon_{y\vartheta}$	132.0	186.0	177.0	152.0	126.0	117.0	115.0	98.0	88.0	62.0	44.0
0.5 $\varepsilon_{y\vartheta}$	165.0	193.0	183.0	160.0	137.0	127.0	125.0	107.0	94.0	68.0	48.0
0.6 $\varepsilon_{y\vartheta}$	197.0	198.0	186.0	166.0	148.0	137.0	133.0	115.0	100.0	73.0	51.0
0.7 $\varepsilon_{y\vartheta}$	220.0	200.0	187.0	172.0	158.0	146.0	139.0	121.0	105.0	77.0	54.0
0.8 $\varepsilon_{y\vartheta}$	229.0	201.0	188.0	178.0	168.0	154.0	147.0	127.0	110.0	80.0	57.0
0.9 $\varepsilon_{y\vartheta}$	234.0	202.0	189.0	181.0	175.0	162.0	154.0	133.0	116.0	84.0	58.0
1.0 $\varepsilon_{y\vartheta}$	235.0	203.0	190.0	184.0	178.0	170.0	157.0	137.0	118.0	88.0	59.0

(b) Grade Fe 360

Table A-1:

(Continued)

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Fe 430	Stress N/mm^2										
ϑ_s [°C] / $\varepsilon_{y\vartheta}$ [%] / strain ε [%]	20	200	250	275	300	350	400	450	500	550	600
	0.179	0.327	0.387	0.407	0.426	0.456	0.515	0.515	0.515	0.517	0.530
0.1 $\varepsilon_{y\vartheta}$	37.7	65.8	76.5	80.5	84.6	82.1	80.8	68.9	58.5	38.1	19.4
0.2 $\varepsilon_{y\vartheta}$	74.6	132.2	147.1	140.1	129.1	119.1	113.6	98.2	84.7	57.9	36.8
0.3 $\varepsilon_{y\vartheta}$	111.6	190.5	190.4	167.3	143.2	131.1	127.3	109.9	96.3	69.3	47.4
0.4 $\varepsilon_{y\vartheta}$	148.5	218.4	207.9	181.5	154.6	143.2	139.0	119.2	105.8	76.2	53.2
0.5 $\varepsilon_{y\vartheta}$	185.5	226.7	215.4	190.8	166.0	154.0	149.5	128.4	112.4	82.2	57.7
0.6 $\varepsilon_{y\vartheta}$	222.4	232.3	218.9	197.3	177.4	164.3	158.0	136.9	118.5	87.6	61.1
0.7 $\varepsilon_{y\vartheta}$	253.1	235.0	220.2	203.5	188.0	173.8	164.3	143.2	123.8	91.9	64.2
0.8 $\varepsilon_{y\vartheta}$	267.7	236.1	221.2	209.8	198.4	182.1	172.4	149.4	128.9	95.0	67.5
0.9 $\varepsilon_{y\vartheta}$	273.6	237.1	222.3	213.2	205.9	190.4	179.7	155.6	135.0	99.1	68.7
1.0 $\varepsilon_{y\vartheta}$	275.0	238.2	223.3	216.3	209.1	198.7	182.9	159.7	137.2	103.2	69.8

(c) Grade Fe 430

Table A-1:

(Continued)

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Fe 510	Stress N/mm²										
ϑ_s [°C] / $\varepsilon_{y\vartheta}$ [%] / strain ε [%]	20	200	250	275	300	350	400	450	500	550	600
	0.217	0.362	0.421	0.440	0.459	0.487	0.545	0.544	0.544	0.550	0.589
0.1 $\varepsilon_{y\vartheta}$	46.1	73.2	83.3	85.9	88.3	90.3	93.9	83.0	69.5	49.2	21.4
0.2 $\varepsilon_{y\vartheta}$	91.7	145.9	165.5	170.6	173.5	164.1	156.1	134.7	114.8	79.6	43.0
0.3 $\varepsilon_{y\vartheta}$	136.7	218.9	235.7	219.6	198.1	182.5	173.4	151.1	130.2	95.2	61.4
0.4 $\varepsilon_{y\vartheta}$	181.4	276.1	267.1	239.3	211.7	195.5	186.9	161.6	141.1	104.3	71.0
0.5 $\varepsilon_{y\vartheta}$	226.2	293.0	279.3	252.1	223.9	207.8	198.3	171.3	149.1	110.7	76.8
0.6 $\varepsilon_{y\vartheta}$	271.0	300.5	284.6	259.8	236.2	218.8	208.0	180.5	155.5	116.8	81.2
0.7 $\varepsilon_{y\vartheta}$	314.9	304.8	286.6	266.5	247.9	229.2	215.0	187.7	161.3	121.6	84.7
0.8 $\varepsilon_{y\vartheta}$	343.5	306.3	287.6	273.4	259.2	238.3	223.1	194.1	166.6	125.1	88.4
0.9 $\varepsilon_{y\vartheta}$	352.7	307.4	288.8	277.5	267.7	247.1	231.1	200.7	173.1	129.2	90.2
1.0 $\varepsilon_{y\vartheta}$	355.0	308.6	289.9	280.8	271.4	256.0	234.6	205.2	175.6	133.7	91.3

(d) Grade Fe 510

Table A-1:

(Continued)

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APPENDIX II: Critical temperature of columns in frames.

II-1 Motivation.

The standard fire resistance test does not provide a realistic input for the design of steel columns under fire conditions, which are generally loaded by both axial force and moment and have variable restraint conditions. In a level 1 design i.e. a pure classification system of components connected to the standard fire, the rough approximation of the complex structural model by a simplified fire test is acceptable, because the required fire resistance generally accounts for a large safety margin and the classification system is supported by many years of experience. However, in more advanced levels of design, the column test becomes questionable as the uncertainty of the structural model may become too large in comparison to the fire development and heat-flow models.
The deficiencies of the standard fire resistance test have certainly stimulated the development of analytical methods, which potentially give better possibilities to achieve solutions with greater economy and more uniform safety. For steel structures, this development has now arrived at a point which enables an anlytical fire engineering design to be carried out in most practical applications. As an alternative for the calculation rules given in 5.4 which are derived for standard fire resistance test conditions of loading, the following, still simplified method for assessing the critical temperature of columns in frames, is therefore recommended. Distinction is made between braced and unbraced frames.

II-2 Critical temperature of columns in braced frames.

II-2.1 Basis of calculation.

The critical temperature of columns in braced frames may be based on the buckling curves specified in 4.4, taking into account the average stress only. Under these conditions the critical temperature is given by the temperature dependent coefficient ψ ($= \frac{\sigma_{y,\vartheta}}{\sigma_{y,20}}$), according to the following rule.

The critical temperature derived in this way should not exceed 550°C.

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In fact, the critical temperature will be a function of both the normal force and the moment distribution. Since the interaction of normal force and moment distribution at elevated temperatures is not very well known, simplified rules are given, derived for centric loading (see also 5.4). If the stresses due to the normal force are small compared to the stresses due to the bending moments, unrealistic high values for the critical temperature may be calculated. To avoid this, an upper limit is recommended for the caculated value of critical temperature. For further information on the elevated temperature buckling curves see comments on 4.4.

II-2.2 Calculation rule.

The limit state of failure is given by the same equation as specified in 5.4.2. For the load multiplier K and the slenderness ratio λ to be adopted see II-2.3 and II-2.4 respectively.

For information and a calculation example see the comments on 5.4.2.

II-2.3 Load multiplier K for columns in braced frames.

In calculations the load multiplier K may be taken equal to 0.85.

See comments on 5.4.3.

II-2.4 Slenderness ratio.

The slenderness ratio should be taken equal th the value used in the design at room temperature, unless it can be shown that under fire exposure more favourable conditions hold.

Initially the fire will occupy one room and in general it is likely to be limited to one storey. Consequently the structural members adjacent to the column under consideration will remain relatively cold. If the column is rigidly connected to the adjacent members, calculations show that the assumption of built-in end conditions can be assumed. This cannot be assumed at room temperature when it is usual to base the slenderness ratio on

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a system length of the column equal to the column length. Thus, under fire conditions a smaller slenderness ratio may occasionally be taken into account. See also Fig. II-1. For a column on the top floor, this condition will not generally be met as far as the connection of the column with the roof girders is concerned: at the top, the column itself and all the adjacent members are normally affected by the fire in a similar way. Consequently, at the top of the column, a hinge must be assumed.

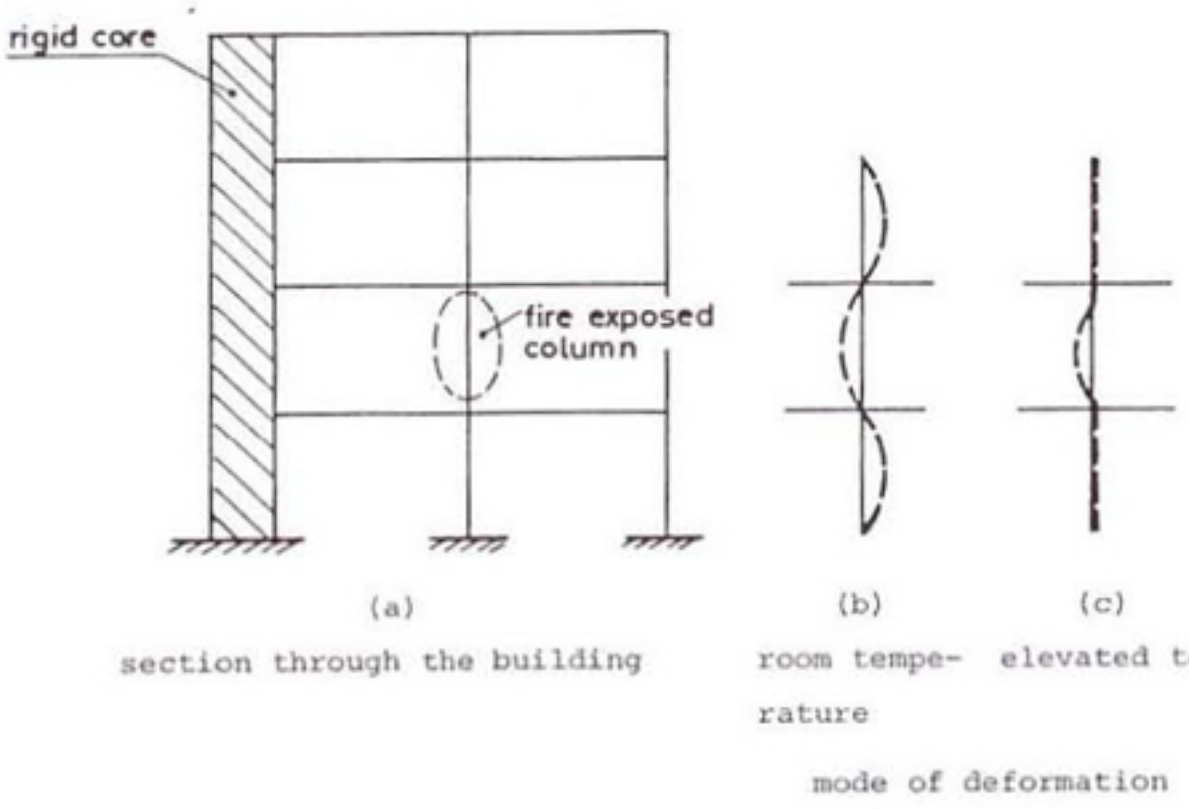

Fig. II-1: Structural behaviour of columns in braced frames (schematic)

II-3 Critical temperature of columns in unbraced frames.

II-3.1 Basis of calculation.

The calculation of the critical temperature of columns in unbraced frames should be based on a full analysis of the structural behaviour at elevated temperature, taking into account possible effects of frame instability. In the absence of such an analysis, a critical temperature of 300°C should be taken. See however II-3.2.

A theoretical analysis of the structural behaviour of fire exposed unbraced frames would be complex especially if frame instability is involved. So far, there only has been some

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limited investigation in this area [21], which indicates that second order effects can significantly influence the stability behaviour under fire conditions. For this reason, a low critical temperature has been recommended, if a full analysis is not carried out.

II-3.2 Calculation rule.

In the case of unbraced frames with no more than two storeys the critical temperature of the columns may be determined by applying the rules given in II-2.

The rule is based on the fact that, under the specified condition, frame instability will be of minor importance. The columns will mainly be loaded on bending.

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Notation

A	= cross section area of the steel member	[mm^2]
E	= modulus of elasticity of steel at room temperature	[N/mm^2]
$E_{t\vartheta}$	= tangent modulus of elasticity of steel at elevated temperature for $\sigma = 0$ N/mm^2	[N/mm^2]
F	= fire exposed surface per unit length of the steel member	[m^2/m]
F_i	= fire exposed area of inner surface over the insulation material per unit length of the steel member	[m^2/m]
K	= coefficient of the total heat transfer	[$W/m^2{}^oC$]
M_p	= plastic moment	[Nm]
N^*	= normal load considered to act on the column during fire	[N]
N_{cr}	= ECCS-buckling load at room temperature	[N]
$\bar{N}_{cr}$	= dimensionless ECCS-buckling load at room temperature	[-]
$\bar{N}_{cr\vartheta}$	= dimensionless buckling load at elevated temperature	[-]
$\dot{Q}$	= heat flow	[W/m]
R	= mechanical resistance	[N]
S	= mechanical action effect	[N]
V	= volume of the steel member per unit length	[m^3/m]
W	= elastic section modulus	[mm^3]
Z	= plastic section modulus	[mm^3]

b	= flange width of the steel member	[m]
c_i	= specific heat of the insulation material	[J/kg^oC]
c_s	= specific heat of steel	[J/kg^oC]
d_i	= thickness of the insulation	[m]
d_s	= web or flange thickness of the steel member	[m]

h	= height of the column, storey	[mm]
h_x	= height of the steel member	[mm]
l	= length, span of beam or frame	[mm]
p	= moisture content by weight of the insulation material	[%]
q^*	= load considered to act on a steel member during fire	[N] or [N/m^1]
q_e	= ultimate elastic load at room temperature	[N] or [N/m^1]
q_p	= ultimate plastic load at room temperature	[N] or [N/m^1]
t	= time	[sec], [min], [h]
t_{fd}	= required fire duration	[min], [h]
t_{fr}	= fire resistance	[min], [h]
t_v	= delay time due to moisture content	[min]
Δ	= difference, interval	[-]
Θ	= factor ≥ 1, representing the plastic reserve of a beam at room temperature due to redistribution of moments	[-]
Ψ	= temperature dependent coefficient equal to the ratio between the effective yield stress at elevated temperature and the nominal yield stress at room temperature	[-]
α	= coefficient of heat transfer	[W/m^2 ^{o}C]
or	= coefficient in the ECCS-buckling curves dependent on the type of cross section	[-]

α_c	= coefficient of heat transfer due to convection from the fire to the exposed surface of the member	[W/m^2 °C]
α_r	= coefficient of heat transfer due to radiation from the fire to the exposed surface of the member	[W/m^2 °C]
δ	= maximum deflection	[mm]
δ_h	= horizontal deflection	[mm]
δ_v	= lateral deflection	[mm]
ε	= actual strain	[%]
ε^*	= strain parameter applying to Fe 360	[%]
$\varepsilon_{y\vartheta}$	= strain at which the effective yield stress at elevated temperatures is reached	[%]
ε_r	= resultant emissivity of the flames, combustion gases and exposed surface	[-]
ϑ_{cr}	= critical steel temperature	[°C]
ϑ_o	= initial ambient gas temperature	[°C]
ϑ_s	= steel temperature	[°C]
ϑ_t	= ambient gas temperature at time t	[°C]
κ	= load multiplier $\leq$ 1, introduced in order to assure consistency between the analytical approach and conventional fire tests	[-]
λ	= slenderness ratio at room temperature	[-]
$\bar{\lambda}$	= modified slenderness ratio at room temperature	[-]
$\bar{\lambda}_\vartheta$	= modified slenderness ratio at elevated temperature	[-]
λ_i	= thermal conductivity of the insulation	[W/m°C]
ρ_i	= density of the insulation material	[kg/m^3]
ρ_s	= density of steel	[kg/m^3]
ρ_w	= density of water	[kg/m^3]
σ	= actual stress	[N/mm^2]
σ^*	= stress parameter applying to Fe 360	[N/mm^2]

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σ_{cr20}	= ECCS buckling stress at room temperature	$[N/mm^2]$
$\sigma_{cr\vartheta}$	= ECCS buckling stress at elevated temperature	$[N/mm^2]$
σ_{y20}	= nominal yield stress at room temperature	$[N/mm^2]$
$\sigma_{y\vartheta}$	= effective yield stress at elevated temperature	$[N/mm^2]$

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