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CONVENTION EUROPÉENNE DE LA CONSTRUCTION MÉTALLIQUE
EUROPAISCHE KONVENTION FÜR STAHLBAU

ECCS - Technical Committee 3 - Fire Safety of Steel Structures

Design Manual on the European Recommendations for the Fire Safety of Steel Structures

FIRST EDITION

1985

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FOREWORD

This Design Manual is a supplement to the European Recommendations for the Fire Safety of Steel Structures (Elsevier Scientific Publishing Company, 1983), also prepared by Technical Committee 3 of the European Convention for Constructional Steelwork (ECCS).

The Recommendations, with the particular aim to serve as a reference document for national codes of practice, give the basis for the application 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.

In order to facilitate an analytical fire engineering design in practice, the present Design Manual was prepared offering, besides comprehensive background information, a variety of tabulated design data and worked examples leading to simple and systematic design procedures.

Like the Recommendations this Design Manual only covers traditional structural steelwork exposed to the standard fire. For analytical design methods for composite concrete slabs with profiled steel sheet it is referred to Publication No 32 of ECCS, 1984. A publication on the fire resistance of composite columns is under preparation.

The membership of ECCS Committee TC3 "Fire Safety of Steel Structures" during the preparation of this Design Manual was as follows:

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Delft, July 1985

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I INTRODUCTION

Fires affect the structural performance of buildings, because they change the physical and mechanical properties of materials of construction. As a consequence a fire engineering design system needs to quantify the fire exposure on the one hand and the effects of that exposure on structural behaviour on the other. Presently, the design system is usually based on grading of elements of construction in a standard fire resistance test. In the building regulations structural performance is defined as the minimum time for which each element would survive if it was subjected to a standard fire test.

More generally, a structural fire engineering design includes two components i.e. quantification of the fire exposure (heat exposure model) and the effect of that exposure on the structure (structural response model) [1,2, 3]. Both models can briefly be described as follows:

- a. Heat exposure model H, for the determination of the rise of temperature in a furnace or compartment as a function of time. The heat exposure model is supplemented by factors which take into account the probability of occurrence of a severe fire, reliability of sprinklers, occupancy, height and volume of the building and the consequence of failure for the overall stability of the building.
- b. Structural response model S, for the determination of the heat transfer to and within the structure and the ultimate load bearing capacity of the structure. The structural model may be experimental or analytical.

The design implies a proof that the structure or the structural element under a defined load and subjected to the specified heat exposure, fulfils certain functional

requirements, expressed by relevant limit states. The available heat exposure models (H) (see vertical column in Fig. I.1) and the structural response models (S) (see horizontal row in Fig. I.1) can be characterized with respect to the type of thermal exposure and the type of structural system. The models are listed in a sequence of improved simulation, and consequently also with increased complexity of application.

Heat exposure models

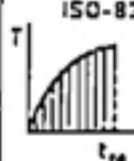
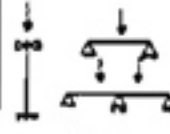
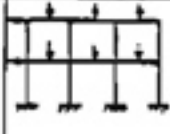
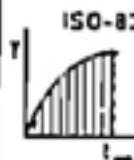
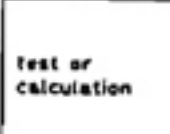
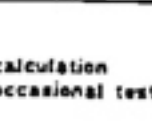
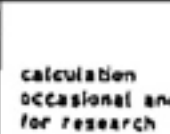
- (H₁) A rise of temperature as a function of time according to the standard temperature time curve.
The duration of the temperature rise is equal to the "required time of fire duration", expressed in building regulations and codes.
- (H₂) A rise of temperature as a function of time according to the standard temperature time curve.
The duration of the temperature rise is equal to the "equivalent time of fire exposure", a quantity which relates a non-standard or natural fire exposure to the standard temperature-time curve.
- (H₃) A rise of temperature as a function of time characterized by an analytical determination of the gas temperature-time curve of a fully developed compartment fire (natural fire).

Structural response models

- (S₁) The load bearing structure is idealized as a series of single members with simplified restraint conditions such as beams and columns. The model can be either experimental (standard fire resistance test) or analytical.
- (S₂) The load bearing structure is idealized as a number of sub-assemblies, such as beam-column systems. Although the model can occasionally be experimental (standard

fire resistance test), an analytical approach will be prevalent.

(S₃) The load bearing structure, such as a building frame or a floor slab system is analysed as a whole. The model is only suitable for analytical design.

Structural Response Model Heat Exposure Model		S ₁	S ₂	S ₃
		Elements	Sub-assemblies	Structures
H ₁				
H ₂				
H ₃				

t_{fd} = required time of fire duration
 t_{ed} = design equivalent time of fire exposure

Matrix of heat exposure and structural response models in sequence of improved simulation [1, 2, 3]. Double lined elements define the scope of the Design Manual (see chapter I.3).

Fig. I.1

Each combination of heat exposure model and structural response model, as an element of the matrix in Fig. I.1,

represents in principle a particular design procedure. It is evident that not all models can be used in all possible combinations. The rule should be to provide a sensible relation in the levels of advancement of both models. In the text in Fig. I.1, reference is made to this aspect [1,2,3].

As discussed, most countries use a fire engineering design in which structural performance is connected to grading of elements of construction by a standard fire resistance test (heat exposure model H_1) [4]. Generally the required time of fire duration is not only related to the estimated fire exposure, but is also differentiated with respect to safety considerations relevant to the structural application and the building in question.

The rise of temperature as a function of time according to the standard fire and the fire duration are a rough approximation of the real gas-temperature-time curve of a fully developed compartment fire. It is possible to calculate a complete gas-temperature-time curve using heat balance equations (heat exposure model H_3) [5, 6, 7, 8]. The amount of combustible material (fire load), the combustion characteristics of the fire load and the geometrical, ventilation and thermal properties of the fire compartment are the important factors. Safety considerations are related to the determination of the design fire load via a set of partial factors [1, 2, 3].

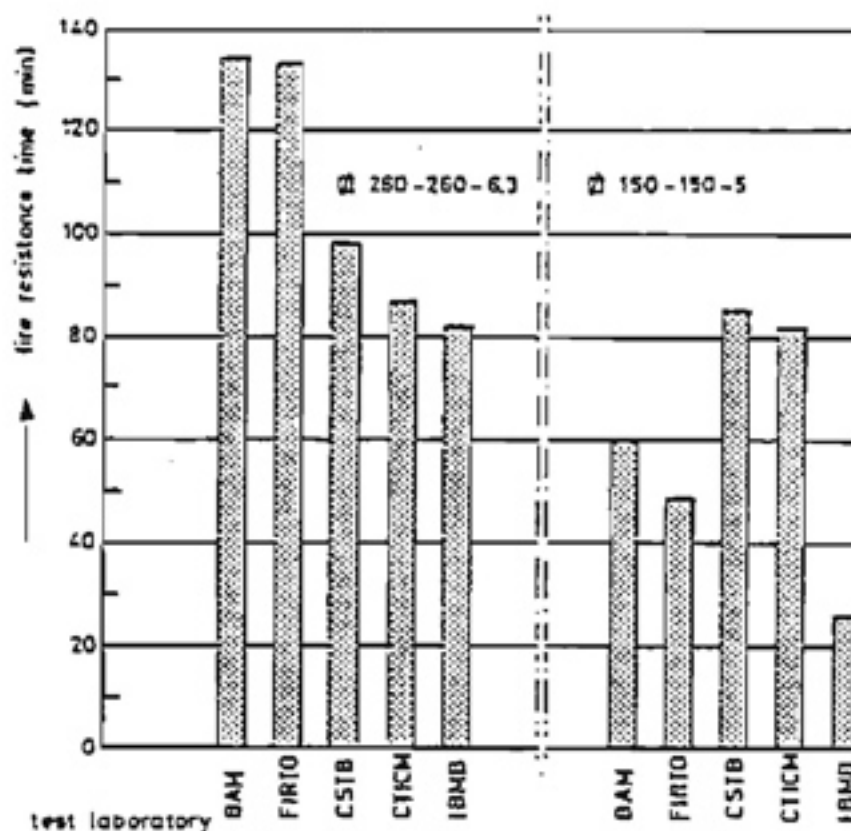
The temperature-time curve of a fully developed compartment fire (heat exposure model H_3) must be calculated, in principle, for any individual application, from the energy and mass balance equations for the fire load and the fire compartment in question. For practical applications this requires a computer or a comprehensive set of design charts for different fire loads, ventilation factors and thermal characteristic of the fire compartment [5, 6]. Moreover, heat exposure model H_3 cannot be used in combination with experimental structural models, which are generally based on the standard temperature-time curve. This is a serious

constraint indeed, because for several structural applications, in particular non-load bearing structures like partitions and doors, the fire resistance test even constitutes the only method of verification. So far only the load bearing capacity of steel structures and, in a more limited sense, concrete structures can be obtained by analytical methods. Therefore, heat exposure model H_2 has been developed, which connects the natural fire according to heat exposure model H_3 , with the standard fire (heat exposure model H_1). The connection between the natural fire and the standard fire comprises a determination of the decisive ultimate state of a reference element, representative for a class of structural systems like steel structures, for a natural fire on one hand and for a thermal exposure according to the standard fire on the other. An equivalent time of fire exposure can be defined as that length of the heating period of the standard curve, which gives the same decisive effect on the reference element with respect to failure as the complete process of a natural fire [6, 7]. Safety considerations relevant to the building in question are related to the equivalent time of fire exposure via a set of partial factors [1, 2, 3, 9, 10, 11].

Most countries use a method of verification based on grading of elements of construction in a standard fire resistance test. Because of limited dimensions of furnaces, only relatively small elements can be tested with simplified end conditions. In the matrix of Fig. 1.1 the method applies to structural model S_1 and occasionally S_2 . Internationally, the standard fire resistance test according to ISO 834 [4] is used very frequently and for many types of structural applications, 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. The specification of the test is insufficient in several aspects, such as heatflow characteristics of furnaces,

material properties and imperfections of the specimen, temperature distribution along members and restraint conditions. Thus, repeated tests in the same furnace, not to mention in different furnaces, may yield a considerable variation in results. The structural element to be tested is supposed to be modelled with respect to actual conditions expected in the structure. However, deviations from conditions in the actual structure are unavoidable because of the limited dimensions of the furnaces, idealized characteristics of the loading device and insufficiently defined support conditions during the test [12, 13]. An illustration is given in Fig. 1.2, which shows some results of a correlation test series on composite columns carried out in different laboratories [14]. It is evident that a considerable and random difference in results does exist. Because of these problems and to achieve solutions with a defined and more uniform safety, there is a strong need to move to analytical structural design systems. Generally these systems include two main steps, viz.:

1. A calculation of the temperature distribution within the fire exposed load bearing element or structure 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 structure at the specified loading.



Some results of fire resistance tests on identical concrete filled hollow steel sections obtained in various test laboratories [14].

Fig. 1.2

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 applies to the structural response models S_1 and S_2 in the matrix of Fig. 1.1. Although in principle an analytical fire engineering design of structural models of the type S_3 is possible, it may be questioned whether the complexity of the model is justified for design in practice, as the structural design at room temperature is usually not performed on entire load bearing structures, but is limited to sub-assemblies of the type S_2 . The structural model S_3 , however, may be a useful tool for research purposes.

Each element in the matrix of Fig. I.1 represents a particular design procedure. The matrix therefore can be considered as a classification system for methods of structural fire engineering design.

The design method H_1-S_1 and occasionally H_1-S_2 with experimental verification of the fire resistance [4] corresponds to a vast majority of national codes. In many countries improved methods based on heat exposure models H_2 and H_3 [5, 8, 9, 10, 11] have occasionally been used, but, except for Sweden, they are not automatically accepted as methods which satisfy the requirements of the building regulations. In contrast to the acceptance of improved heat exposure models, there is a growing acceptance of design methods H_1-S_1 and H_1-S_2 with an analytical verification of the fire resistance. In several countries these methods are now being used as an alternative to the standard fire resistance test.

The European Recommendations [15] to which this document is a Design Manual, apply to design methods based on heat exposure models H_1 and H_2 (standard fire) and analytical structural response models S_1 and S_2 .

II. FIRE EXPOSURE

Terminology

The following terminology is used to describe different types of fire:

A real fire is a developed fire in a building, influenced or not by an extinction system and/or fire brigade action.

An experimental fire is a test fire with specified characteristics and may be small scale or large scale.

A design fire is a fire with a specific thermal exposure, to be used in a structural design. If the thermal exposure is given by the standard temperature-time curve according to the ISO Standard 834, the design fire is the standard fire. If the thermal exposure simulates a real fire in a fire compartment in a simplified and idealized way, it is a compartment fire. Such a fire can be specified either by the variation with time of the heat flux to an exposed structure or structural member or by the variation with time of the gas temperature in the compartment.

II.1 Introduction

As stated in chapter I, there has been important progress during the last ten years in the development of computation methods for analytical structural fire engineering design. For steel structures, this has reached a stage where an analytical design can be carried out for most practical applications on the basis of either the standard fire or a compartment fire which takes into account the combustion characteristics of the fire load, the ventilation of the fire compartment and the thermal properties of the structures enclosing the fire compartment.

In spite of this development, the standard fire resistance test still predominates internationally as the basis for fire classification and design. That is the main reason why the ECCS-Recommendations are limited, in first place, to an analytical determination of the fire resistance of load bearing steel elements and structural assemblies thermally exposed to the standard fire.

Where analytical structural design for a compartment fire is concerned, this can be carried out either by a direct use of the time variation of this exposure or, more roughly, by applying the concept of equivalent time of fire exposure, which relates the compartment fire to the standard fire. This concept can give an improved classification for fire ranking or grading of structural elements, when integrated with a probabilistic model, and taking into account, [1] - [12]:

- * the uncertainties of the numerical values of the influence variables (stochastic variables),
- * the uncertainties of the prediction models for the heat exposure and the thermal and structural response,
- * the probability of occurrence of a developed fire - depending on the compartmentation, the fire brigade action and an automatic extinction system, if any, and
- * the consequences of a structural failure - depending on such factors as the occupancy, the height and volume of the building and the importance of the structure or structural member to the overall stability of the building.

The following presentation opens with an outline description of the main characteristics of the fire load with examples of fire load statistics found in recent investigations. The main part of the chapter then deals with analytical models for the design fire exposure with reference to the compartment fire, the standard fire and the concept of equivalent time of fire exposure. The presentation concentrates on the standard fire and the equivalent time of fire exposure, to match the ECSS-Recommendations.

II.2 Fire Load Density

Ordinarily, the fire load in a compartment is defined as the total quantity of heat Q released on complete combustion of all combustible materials, contained inside the fire compartment, inclusive of furnishings, surface material, linings and coverings on walls, roof and floor as well as the load bearing and non-load bearing structure or structural members. Divided by a characteristic reference area, Q gives the fire load density q . Most countries use the floor area A_f as the reference area, while some countries use the total interior area A_c of the surfaces bounding the fire compartment, including all openings. The alternative use of the bounding area A_c originates from the fact that this quantity is a fundamental parameter in the heat balance of the compartment fire.

The two versions of the fire load density are given by the formulae

$$q_f = \frac{1}{A_f} \sum m_i H_{v,i} \quad (\text{II-1a})$$

$$q_c = \frac{1}{A_c} \sum m_i H_{v,i} \quad (\text{II-1b})$$

where

q_f = fire load density, per unit floor area ($\text{MJ} \times \text{m}^{-2}$),

- q_c = fire load density, per unit area of bounding surfaces ($\text{MJ} \times \text{m}^{-2}$),
 m_v = total mass of combustible material (kg),
 H_v = calorific value of combustible material ($\text{MJ} \times \text{kg}^{-1}$),
 A_f = floor area of the fire compartment (m^2),
 A_c = total interior area of the surfaces, bounding the fire compartment, including all openings (m^2).

The calorific values of some solid, liquid and gaseous materials are given in table II.1.

For a fire compartment with a guaranteed permanent use and fittings, the design value of the fire load density can be directly determined according to equation II-1. Such an individual survey is worthwhile doing if the fire load statistics for the relevant type of fire compartment have a large coefficient of variation. The determination may, however, result in certain drawbacks in the event of future alterations and rearrangement in the building. It is therefore more convenient to determine the design fire load density on the basis of statistical investigations for the type of building or premises in question. Such statistical investigations have been carried out in several countries for dwellings, offices, administration buildings, schools, stores, and hospitals - cf., for instance [13] - [19].

Table II.2a gives an example of fire load statistics according to recent Swedish investigations [15], [16]. The reported values are instantaneous values, referred to unit area of the surfaces bounding the fire compartment and determined according to equation II-1b. The table gives the average value, the standard deviation and the characteristic value, defined as the 0.8 fractile value, as authorized in the Swedish Building Code at present [20]. In addition, table II.2b gives the corresponding characteristic value of the fire load density q_f per unit floor area [21], based on the

results of the same statistical survey. Roughly, the values of the fire load density given in tables II.2a and b are on the same level as corresponding values, obtained in German, Swiss [13], [14], [17] and US fire load surveys [18], [19]. This is illustrated in table II.2c [13], which gives the average value and the standard deviation of the fire load density q_f per unit floor area from a very comprehensive investigation carried out by ECCS for various types of fire compartments in office buildings.

As a rule, complete combustion of all combustible materials in a fire compartment does not occur. This can be allowed for by using the following relationships for the fire load density:

$$q_f = \frac{1}{A_f} \sum u_v m_v H_v \quad (\text{II-2a})$$

$$q_c = \frac{1}{A_c} \sum u_v m_v H_v \quad (\text{II-2b})$$

where

u_v = a fraction between 0 and 1, giving the real degree of combustion for each individual component of the fire load.

The non-dimensional factor u_v varies according to the type of fuel, its geometrical properties and position in the fire compartment, and the fire environment (heat flux and duration of heating). Bookcases, stacks of papers and floor coverings are examples of fire load components whose real degree of combustion is usually low, and whose u_v values consequently are well below unity. At present, there is a lack of experimentally verified u_v values for a range of fire environments so that, normally, formula II-2 cannot be used. A test apparatus and a test procedure for evaluating u_v in one specific environment are described in [1] and [2], which include comprehensive lists of values, determined for various materials, under this condition. A more

differentiated procedure for determining u_v , which takes into account the porosity of the fire load and the temperature-time curve of the design fire is presented in [13] and [14]. The procedure is iterative and primarily applicable to stacks of papers in cabinets. However, the values of the fire load density reported in tables II.2a-c generally assume $u_v = 1$.

II.3 Analytical Models of Design Fires

II.3.1 Compartment Fire Exposure

In practice, it is the fully developed regime of the compartment fire which significantly affects the behaviour of the load bearing structure or structural members. The essential influences on this part of the fire process - the post-flashover compartment fire - are

- * amount and type of combustible materials in the compartment (fire load),
- * porosity and particle shape of the fire load,
- * distribution of the fire load in the compartment,
- * amount of air per unit time supplied to the compartment,
- * geometry and openings of the compartment, and
- * thermal properties of the structures, enclosing the compartment.

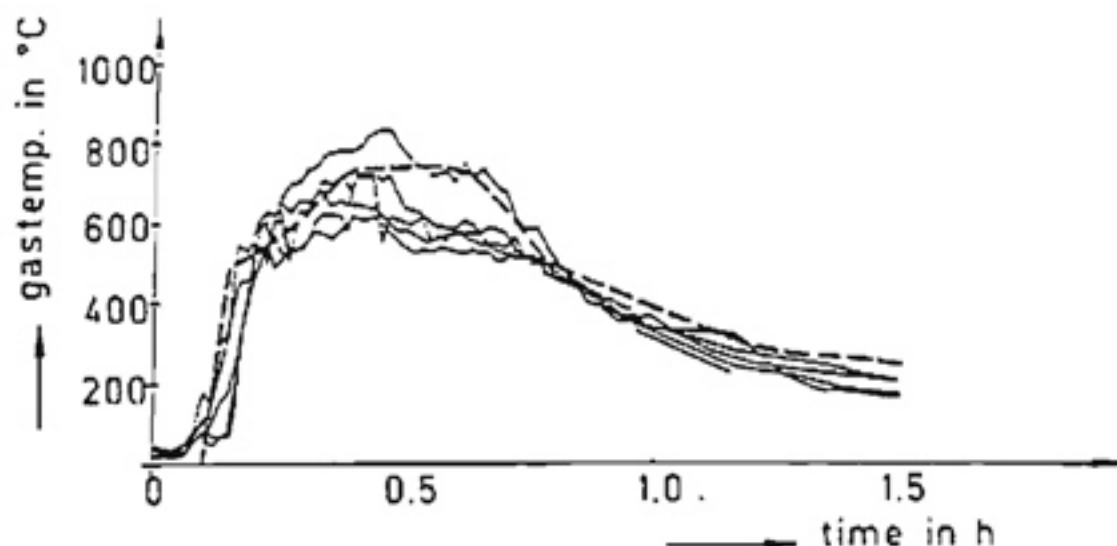
For given combustion characteristics of the fire load, the variation with time of the heat flux to an exposed structure or structural member or the gas temperature-time curve of the fire compartment can be calculated for an individual compartment from the energy and mass balance equations with respect to the size, geometry and ventilation of the compartment, and to the thermal properties of the enclosing structure [15], [16], [20], [22] - [33]. These compartment effects are characterized by the so-called "opening factor", $A_w \sqrt{h_w} / A_c$, and the coefficient, K_f , where A_w is the area and h_w the mean height of the ventilation openings and K_f is

given in table II.3 for different thermal properties of the enclosing structures.

As a simplification, post-flashover compartment fires can be divided into two types of behaviour - ventilation controlled or fuel controlled, [22]. For the first type, the combustion during the active stage of the fire is controlled by the ventilation of the compartment with the burning rate approximately proportional to the air supply through the openings of the fire compartment and not in any decisive way dependent on the amount, porosity and particle shape of the fuel. For the second type, the combustion is mainly controlled by the properties of the fuel (fire load) and is largely independent of the air supply through the openings. The boundary between the two kinds of compartment fires is not clearly marked.

The practical use of the analytical models developed for the post-flashover compartment fire is facilitated by access to well documented computer programmes - cf., for instance [30]. A closed-form approximation, arranged to suit hand-calculations, is presented in [31].

The ability of the models is illustrated in figure II.1 and II.2. In figure II.1, a set of full-line curves is reproduced from a Japanese full-scale test, showing the temperature-time curves of the combustion gases at different points in the fire compartment [34]. The dash-line curve shows the corresponding temperature-time relationship calculated by applying the analytical model derived in [25].



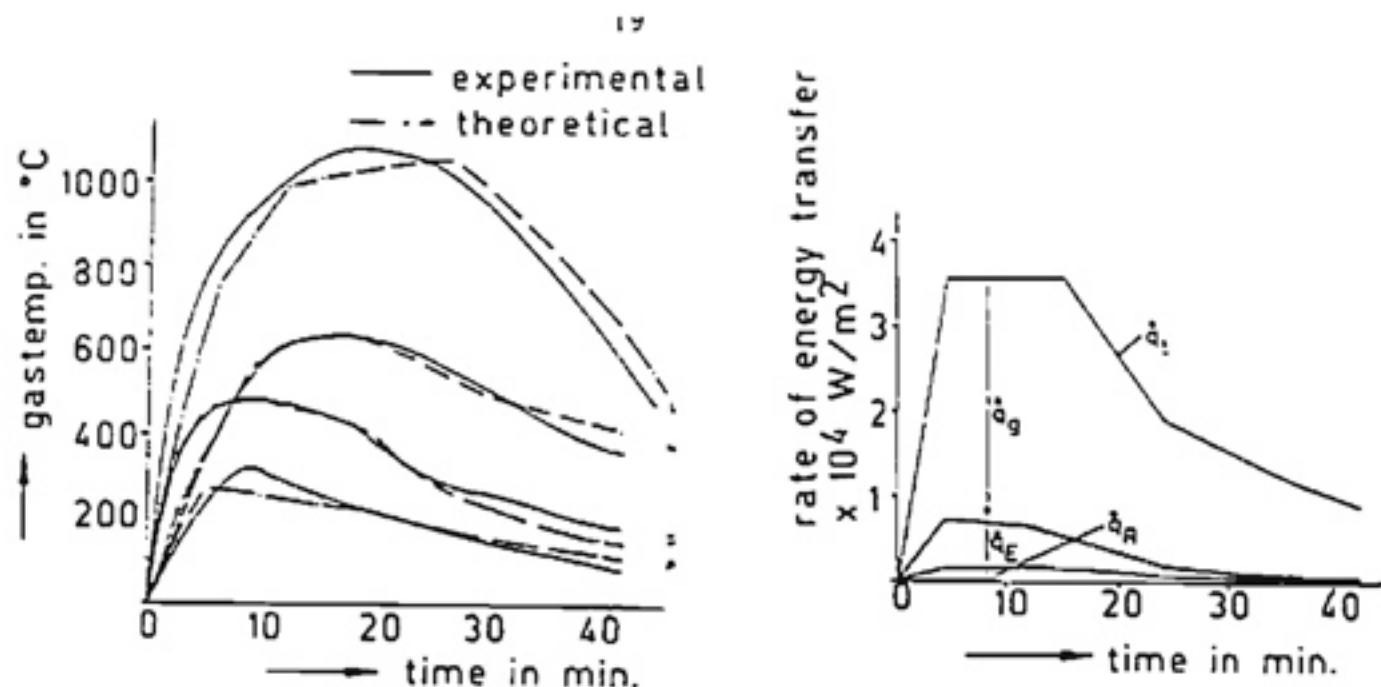
Experimentally determined time curves of the gas temperature at different points in a fire compartment (full-line curves) [34], compared with a theoretical curve calculated according to the analytical model presented in [25] (dash-line curve). Fire load density $q_c = 140 \text{ MJ} \times \text{m}^{-2}$; fire compartment with opening factor $A_w \sqrt{h_w} / A_t = 0.0467 \text{ m}^{1/2}$ and bounding structures of concrete floor, concrete roof and walls made of hollow concrete blocks

Fig. II.1.

Figure II.2 shows the variation with time of the average gas temperature inside the fire compartment, measured in full-scale tests performed at the Fire Research Station in UK [35], compared with the corresponding curves obtained by the analytical model in [25] - cf. [27]. The figure also illustrates, for one of the tests, the proportions between the terms in the energy balance equation of the fire compartment

$$\dot{q}_c = \dot{q}_g + \dot{q}_e + \dot{q}_R \quad (\text{II-3})$$

where $\dot{q}_c$ = rate of heat release by combustion inside the compartments, $\dot{q}_g$ = rate of heat loss by convection through the openings, $\dot{q}_e$ = rate of heat loss through the bounding walls, floor and ceiling, and



Experimental variation with time of the average gas temperature inside a fire compartment (full-line curves) [35], compared with corresponding theoretical curves calculated according to the analytical model presented in [25] (dash-line curves). Fire load density $q = 31$ (test P), 62 (test D), 124 (test N) and 248 $\text{MJ} \times \text{m}^{-2}$ (test L); fire compartment with opening factor $A_w \sqrt{h_w} / A_t = 0.12 \text{ m}^{1/2}$ and bounding structures of concrete floor, refractory concrete ceiling and walls of common brick. The time curves for the different terms in the energy balance equation for test D are shown at the right [27]

Fig. II.2.

$\dot{q}_R$ = rate of heat loss by radiation through the openings. Generally speaking, the results shown verify that the applied analytical model is adequate for the simulation of experimental full-scale fires.

The available analytical models and computer programmes for the post-flashover compartment fire provide the basis for a systematized design. The gas temperature-time curves $\theta_c - t$ in figure II.3, [15], [16], [20], [25], are an example of such a design basis for use in an analytical design of load bearing structures and partitions exposed to a compartment fire. The curves are approved by the National Swedish Board of Physical Planning and Building for general practical application. They apply to a fire compartment with surrounding

structures, made of a material with a thermal conductivity $\lambda_b = 0.81 \text{ W} \times \text{m}^{-1} \times ^\circ\text{C}^{-1}$ and a heat capacity $pc_b = 1.67 \text{ MJ} \times \text{m}^{-3} \times ^\circ\text{C}^{-1}$ which has $K_f = 1.0$, i.e. fire compartment type A. Such a surrounding material corresponds roughly to an average of brick, concrete and aerated concrete.

The gas temperature-time curves in figure II.3 have been computed from the energy and mass balance equations of the fire compartment under the following simplifying assumptions:

- * the combustion of the fire load takes place entirely within the fire compartment,
- * the fire process is ventilation controlled,
- * the temperature θ is uniform within the fire compartment at any time t ,
- * the coefficient of heat transfer for the internal surrounding surface is uniform over each individual part of the surface at any time t , and
- * the heat transfer to and through the surrounding structures is one-dimensional and except window and door openings, uniformly distributed for each type of structure.

The consequences of the assumptions are analysed in [15], [27] and [36], verifying their reasonableness as a basis for the calculation of the load bearing capacity of fire exposed structures and structural members.

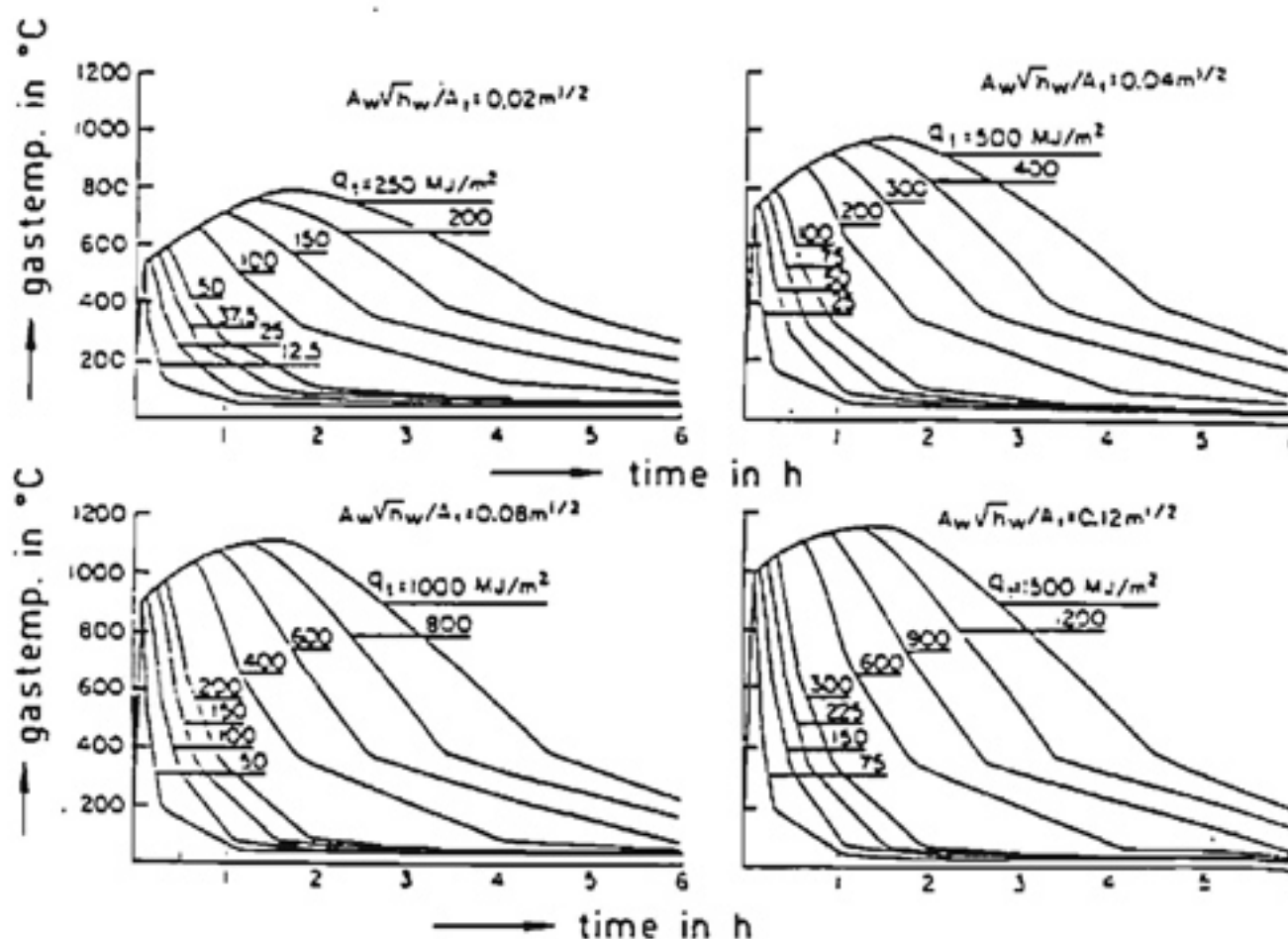
For fire compartments with surrounding structures, whose thermal properties are different from those of fire compartment, type A, an effective fire load density q_{cf} and an effective opening factor $(A_w \sqrt{h_w} / A_c)_f$ can be used in the heat balance, calculated from the real fire load density q_c and the real opening factor $A_w \sqrt{h_w} / A_c$ as follows [15], [16], [20]:

$$q_{cf} = K_f q_c \quad (\text{II-4a})$$

$$(A_w \sqrt{h_w} / A_c)_f = K_f A_w \sqrt{h_w} / A_c \quad (\text{II-4b})$$

where the coefficient K_f is given in table II.3 for seven types of fire compartment.

The gas temperature-time curves illustrated in figure II.3 are applicable to fire compartments of a size representative of dwellings, ordinary offices, schools, hospitals, hotels and libraries. For fire compartments with a very large volume - for instance, large industrial buildings and sports halls - the curves give an unsatisfactory description of the real fire exposure. For such types of buildings, it is confirmed by theory



Example of gas temperature-time curves $\theta - t$ of the post-flashover compartment fire for different values of the fire load density q_f and the opening factor $A_w \sqrt{h_w} / A_c$. Fire compartment, type A - from authorized Swedish Standard Specifications [20]

Fig. II.3.

and experience [37] that flashover will frequently not occur. This pre-flashover compartment fire can give a local fire exposure to a structural member - for instance, a beam, a column or a frame - more severe or less severe than that assumed for the post-flashover compartment fire illustrated in figure II.3. At present, there is no validated analytical model or design basis available for a determination of the fire exposure, which can be generally applied to fire compartments with a very large volume.

II.3.2 Standard Fire Exposure

In the majority of countries, the national requirements for fire engineering design of load bearing structural elements and partitions are directly related to the standard fire resistance test. In the design, the results of the tests have to meet the corresponding requirements specified in the building codes and regulations.

Fire resistance tests are carried out according to national specifications which vary only in minor details from country to country. In principle, the test conditions are in conformity with ISO Standard 834 [38].

In the test, the specimen is exposed in a furnace to a temperature rise, which is controlled so as to vary with time within specified limits according to the relationship:

$$\theta_t - \theta_0 = 345 \log_{10} (8t+1) \quad (\text{II-5})$$

where

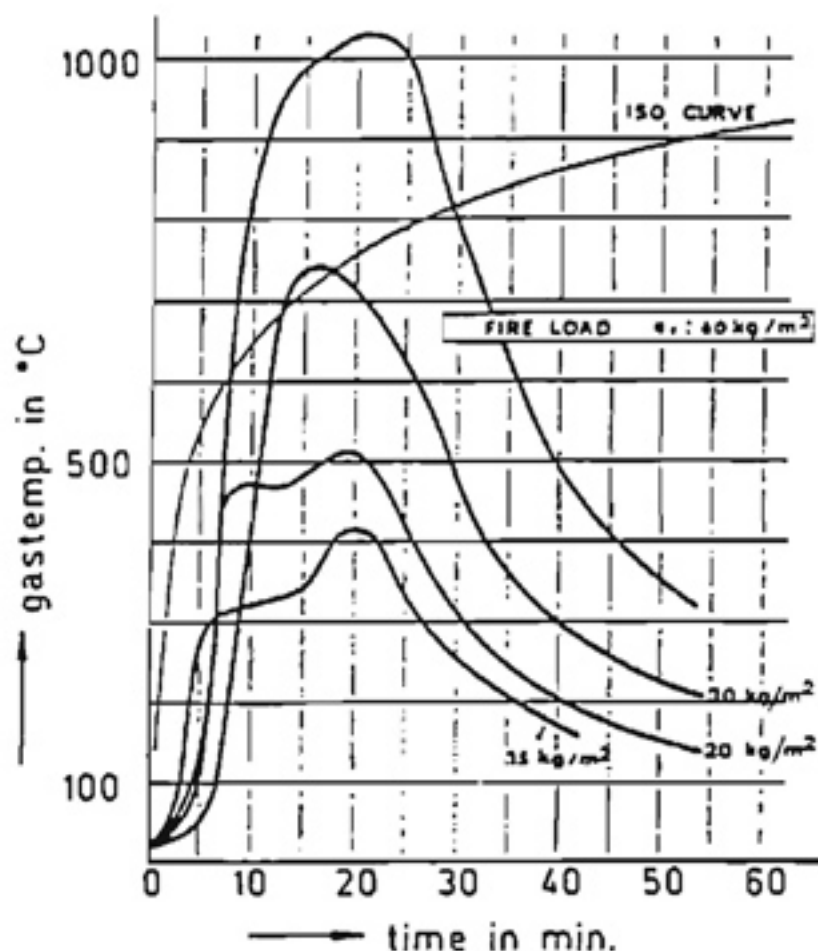
t = time, in minutes,

θ_t = furnace temperature at time t , in $^{\circ}\text{C}$, and

θ_0 = furnace temperature at time $t=0$, in $^{\circ}\text{C}$.

The temperature-time curve given by equation II-5 is

shown in figure II.4 for $\theta_0 = 20^\circ\text{C}$. For comparison, the figure also includes a set of curves giving the average gas temperature inside the fire compartment, obtained in full-scale tests carried out at the fire test station at Maizieres-les-Metz for four different fire load densities in a room with an opening factor



The standard fire according to equation II-5 (ISO curve), compared with the time curves of the average gas temperature determined in four full-scale experimental fires characterized by the same opening factor $A_w/\sqrt{h_w} = 0.157 \text{ m}^{1/2}$ and a varying fire load density $q_{f,t} = 15, 20, 30$ and $60 \text{ kg wood per m}^2 \text{ floor area}$ [39]. Corresponding fire load density $q_{f,t} = 45, 60, 90$ and 180 MJ m^{-2} , respectively. Compartment with bounding structures of floor and one wall of light weight concrete, three walls of common brick and ceiling of refractory concrete - walls and ceiling insulated on the inside by a vermiculite based plaster

Fig. II.4.

$A \sqrt{h_w} / A_w = 0.157 \text{ m}^{1/2}$ [39]. The curves demonstrate that the thermal exposure in an experimental fire can deviate considerably from the thermal exposure in the standard fire.

Internationally, the standard fire resistance test is considered to be one of the most thoroughly understood test methods. In spite of this, the test can be criticized. In its present form, a number of features are not clearly defined. These include the heating and restraint characteristics, the environment of the furnace, and the thermocouples for measuring and regulating the furnace temperature. Consequently, a considerable variation may arise in the value of fire resistance measured for one and the same structural member, when tested in different laboratories, because the furnace characteristics and the support and restraint conditions of the test specimens may be different. See for instance [40] and [41].

The important progress, noted during the last ten years in the development of computation methods for an analytical structural fire engineering design, enables fire resistance to be determined by calculation for many practical applications. Consequently, more and more countries are accepting analytical classification of load bearing structural elements with respect to the standard fire as an alternative to testing. An important purpose of the ECCS-Recommendations is to stimulate more countries to accept this classification method officially and to facilitate the practical application of the procedure.

Irrespective of the fire resistance being determined analytically or by a test for a thermal exposure according to equation II-5, it is important to remember that the standard fire resistance test does not represent the real fire exposure in a building nor does it measure the behaviour of the structural element as a part of an assembly in a building. What the test or the cor-

responding calculations do is to grade structural elements and the building codes and regulations then require different grading levels of elements according to the circumstances.

The grading levels required depend on such influences as the occupancy, the building height and volume and the importance of the structure or structural element for the overall stability of the building. One of the factors included is the severity of fire exposure which might occur, specified as the standard fire duration t_d . This was first quantified by Ingberg who derived the relationship

$$t_d \sim Q/A_f = q_f \quad (\text{II-6})$$

which is still the basis of most current regulations. In later work, the importance of the ventilation and geometry of the compartment was also quantified, as described in section II.3.3.

When applying the standard fire duration t_d , it is essential to remember that it is only a test duration and not a real fire duration.

II.3.3 Equivalent Time of Fire Exposure

The concept of equivalent time of fire exposure was introduced as a way of relating a compartment fire exposure to the heating according to the standard fire - equation II-5. The concept can be used in practice, for instance, to give an improved classification for fire ranking or grading of structural elements. In principle, the equivalent time of fire exposure is defined as that length of the heating period in a standard fire which gives the same, critical effect on a structural element with respect to failure as the complete process of the compartment fire.

For steel structures, two alternative forms of the con-

cepts are given in the literature. The most simple one gives the equivalent time of fire exposure t_e as a function only of the parameters influencing the fire process. In a more developed form, the effect of the structural parameters is also included.

A simple formula, giving the equivalent time of fire exposure as independent of the structural parameters, was derived in [42] and [43] for protected steel structures as follows: A reference element was chosen, with thermal characteristics such that for a given experimental fire exposure it would attain a maximum steel temperature of a fixed value, e.g. 500 °C. The equivalent time of fire exposure t_e was then determined over the temperature-time curve of the standard fire for the same element to attain the same steel temperature. By repeating this procedure for different experimental fires, an empirical formula was obtained giving t_e as a function of only the fire load and the properties of the fire compartment. A similar, but generalized approach, based on the compartment fire exposure illustrated in figure II.3, as presented in [44] and [45], gives the following approximate formula:

$$t_e = 0.067 \frac{q_{tf}}{\left(\frac{A_w \sqrt{h_w}}{A_t} \right)^2} \quad (\text{min}) \quad (\text{II-7a})$$

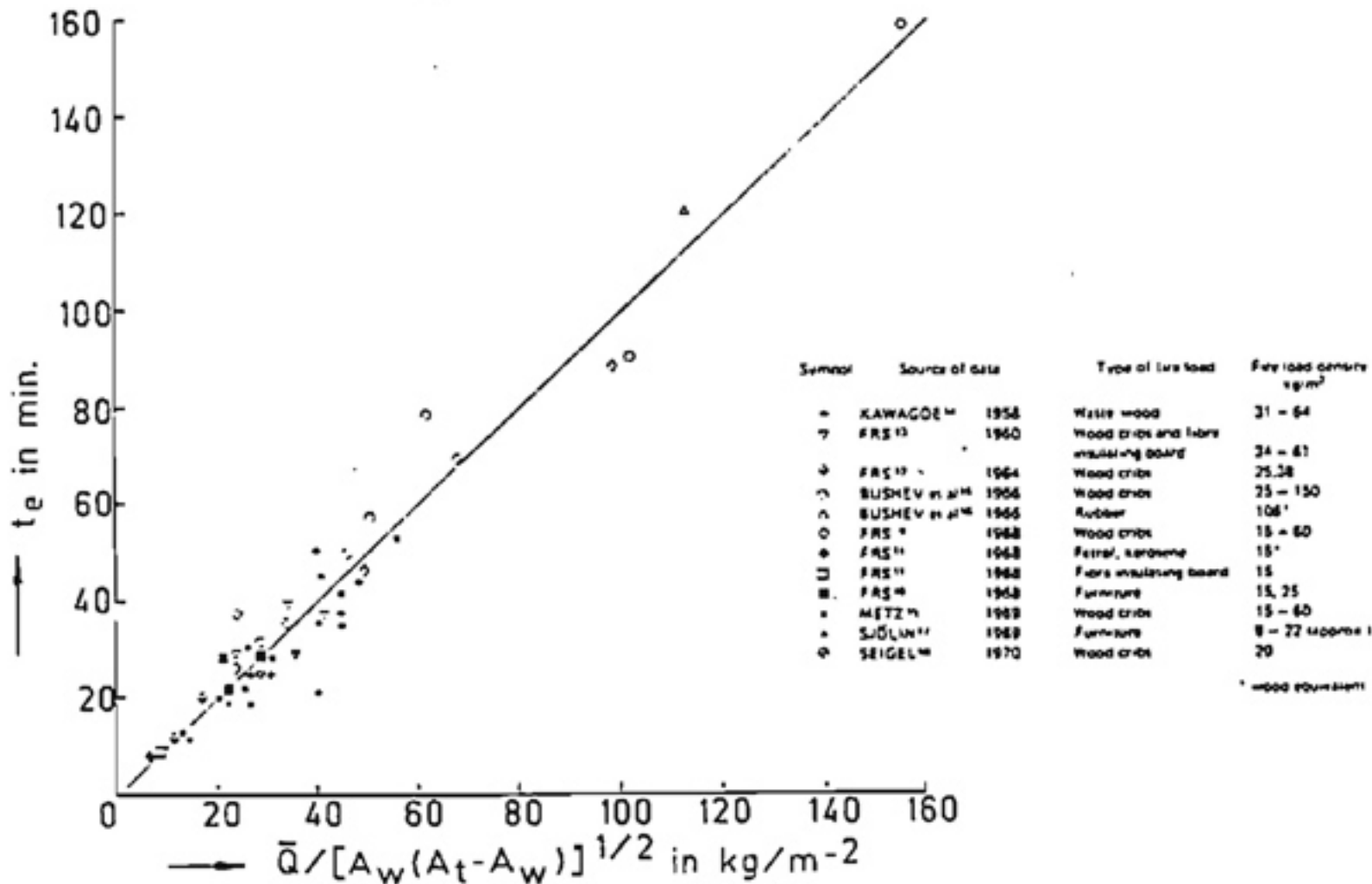
in which q_{tf} is the effective fire load density per unit area of the surfaces bounding the fire compartment ($\text{MJ} \times \text{m}^{-2}$) and $\left(\frac{A_w \sqrt{h_w}}{A_t} \right)^2$ the effective opening factor of the fire compartment ($\text{m}^{1/2}$), calculated according to equation II-4 and table II.3. In this form, the formula can take into account the influence of varying thermal properties of the surrounding structures of the fire compartment.

Alternatively, equation II-7a can be written as

$$t_e = 0.067 \frac{q_{ff} A_f}{A_c \left(\frac{A_w \sqrt{h_w}}{A_c} \right)^2} \quad (\text{min}) \tag{II-7b}$$

with $q_{ff} = K_f q_f$ = the effective fire load density per unit floor area ($\text{MJ} \times \text{m}^{-2}$).

Figure II.5 demonstrates the validity of equation II-7 for protected structural steel members [42]. In the figure, the symbols give values of the equivalent time of fire exposure t_e as a function of $\bar{Q}/[A_w(A_t-A_w)]^{1/2}$, based on gas temperature--time curves determined in experimental fires in larger-scale brick and concrete compartments. $\bar{Q}$ is the total fire load expressed as the equivalent weight of wood in kg. The tests comprise different types of fire



Equivalent time of fire exposure t_e as a function of $\bar{Q}/[A_w(A_t-A_w)]^{1/2}$, derived for insulated steel elements from experimental fires in larger-scale brick and concrete compartments. $\bar{Q}$ is the total fire load in kg wood equivalent [42]

Fig. II.5.

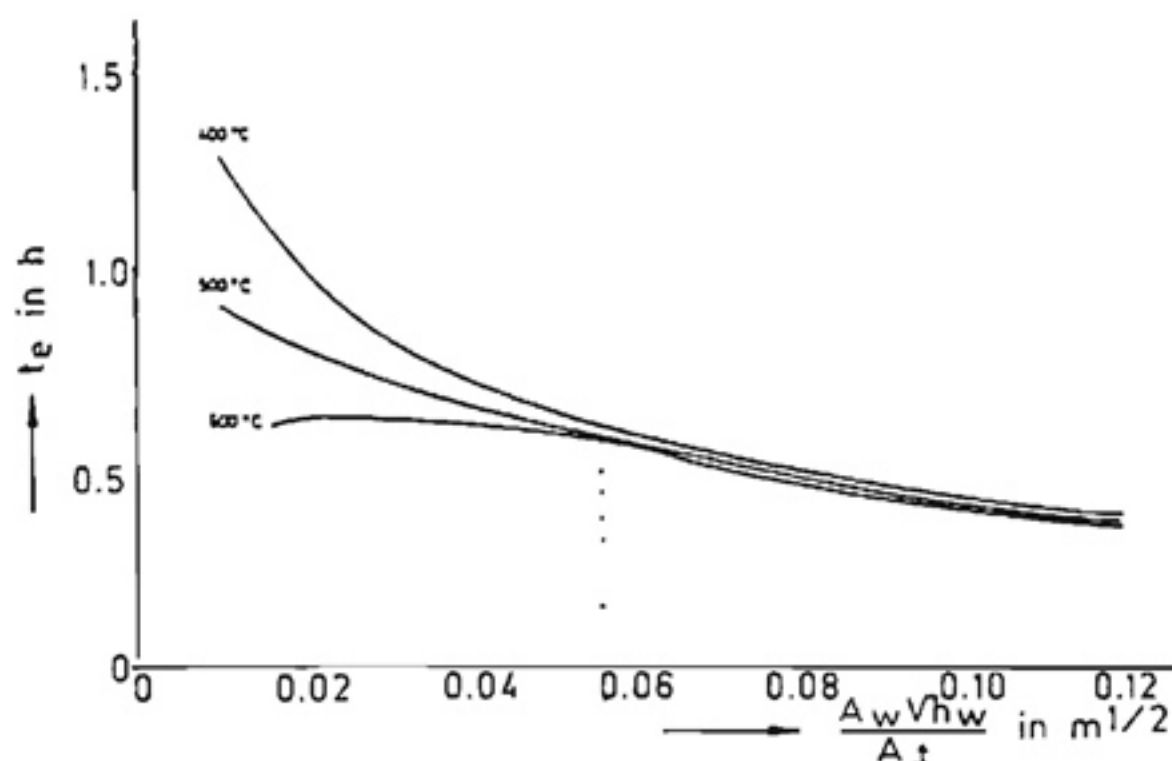
loads according to the table included in the figure with a range for the fire load density q_f from 9 to 150 kg wood equivalent per m^2 floor area. For frequent values of the opening height h_w and the quotient $(A_t - A_w)/A_t$, the straight line in figure II.5 gives values of the equivalent time of fire exposure t_e which are in reasonable agreement with the values obtained by equation II-7.

As the formulae according to equation II-7 are derived from the gas temperature-time curves in figure II.3, this agreement indirectly demonstrates that these curves are a reasonable basis for a fire engineering design of load bearing structures and structural members.

The formulae given by equation II-7 and the relationship shown in figure II.5 are based on a critical steel temperature of 500°C . The influence on the equivalent time of fire exposure of varying the value of the critical steel temperature is illustrated in figure II.6 for protected steel elements, fire exposed according to figure II.3 at a fire load density $q_f = 125 \text{ MJ} \times m^{-2}$ [44]. The figure shows that the influence on t_e of variations in the critical steel temperature is considerable for low values of the opening factor $A_w \sqrt{h_w}/A_t$. For $A_w \sqrt{h_w}/A_t > 0.05 \text{ m}^{1/2}$, this influence is comparatively small. A similar effect is noted for unprotected structural steel elements.

Summarizing, the formulae of equation II-7 have been shown to be applicable to fire exposed unprotected and protected structural steel elements, with a critical steel temperature of about 500°C and they can also be used for other values of the critical steel temperature, provided that the opening factor of the fire compartment $A_w \sqrt{h_w}/A_t > 0.05 \text{ m}^{1/2}$. For smaller values of the opening factor, an improved design basis is given in Appendix A, showing the equivalent time of fire exposure t_e as dependent on the section factor

as well as the parameters influencing the fire process.



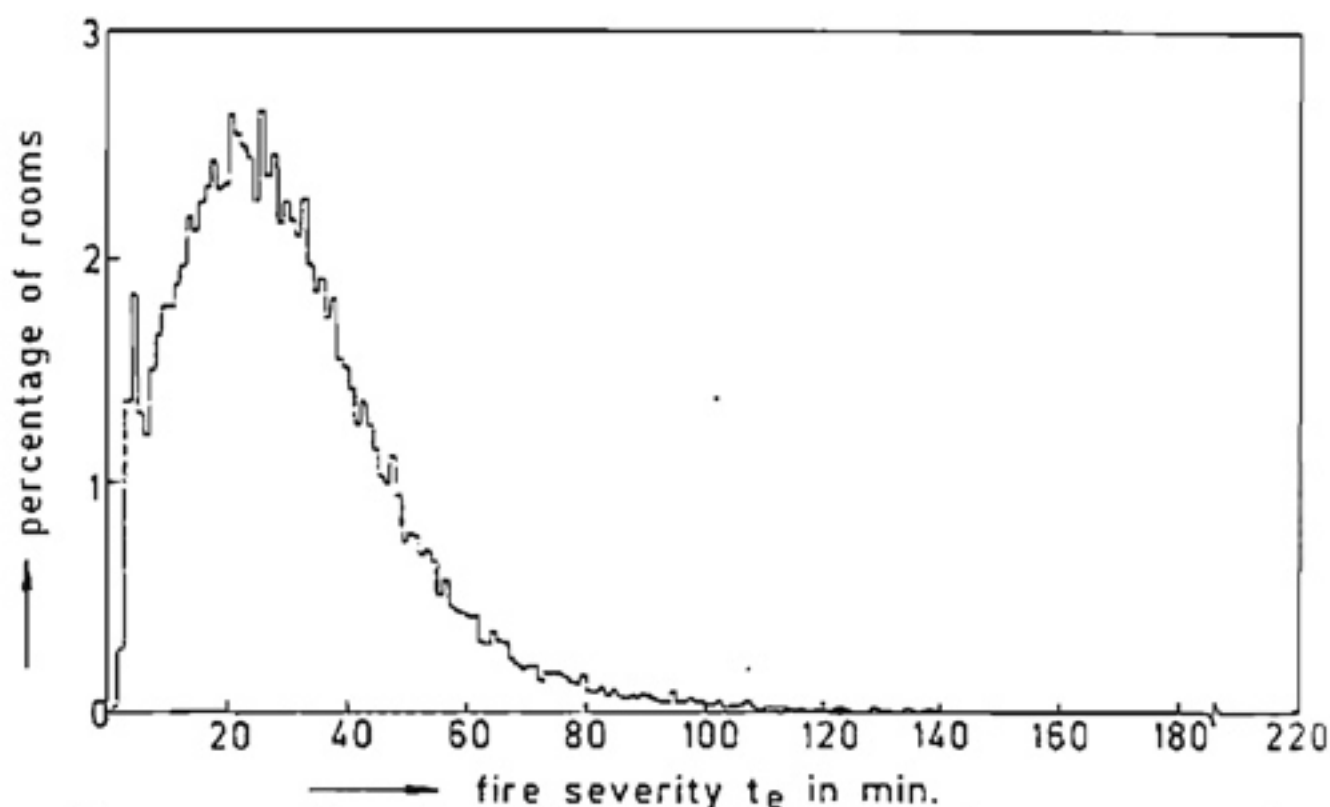
The influence on the equivalent time of fire exposure t_e of varying level of the critical steel temperature for protected steel elements. Compartment fire exposure according to figure II.3; fire load density $q_z = 125 \text{ MJ m}^{-2}$ [44]

Fig.II.6

Although the equivalent time of fire exposure relates the compartment fire exposure and the heating according to the temperature-time curve of the standard fire, the concept does not give any useful information on, for instance, the expected rate of heating or real duration of the real fire. In addition, the formulae derived may not be applicable to large fire compartments and will have the same limitations already discussed in relation to the gas temperature-time curves of the post-flash-over compartment fire reproduced in figure II.3.

In combination with statistical data, the equivalent time of fire exposure can be used to describe the fire severity for different types of fire compartments.

Figure II.7 illustrates this application of the concept [46]. The figure gives the frequency distribution of the fire severity, expressed as the equivalent time of fire exposure t_e in intervals of one minute, and estimated by a Monte Carlo simulation from data, obtained from statistical surveys of modern office rooms. At the simulation, t_e was defined by a formula determined by the straight line in figure II.5.



Frequency distribution of the fire severity, expressed as the equivalent time of fire exposure t_e , for modern office rooms, obtained by simulation [46]

Fig. II.7.

As mentioned in the introduction, an analytical structural design for a compartment fire can be integrated with a probabilistic model. Various levels of ambition can then be distinguished. For ordinary applications, however, such a probability based structural fire design must follow a simplified approach according to a practical design format calculation, specifying characteristic values and related partial safety factors for the action effects and the response capacities.

For a fire design, related to the equivalent time of fire exposure t_e , the practical design format can be given in the following form [2], [8]:

$$\frac{t_{fr}}{\gamma_f} \geq \gamma_n \gamma_e t_e \quad (\text{II-8})$$

in which t_{fr} is the fire resistance of the structural element - determined either experimentally by a standard fire resistance test or by calculation - and γ_f , γ_e , are partial safety factors, related to the fire resistance t_{fr} and the equivalent time of fire exposure t_e , respectively, and γ_n is an adaption factor accounting for the consequences of a structural failure (different safety classes) and special fire fighting provisions, for instance, the effect of a sprinkler system or the efficiency of the fire brigade actions. The partial safety factor γ_e then covers the uncertainties of the fire load density and the fire compartment characteristics, including the uncertainties of the analytical model for a determination of the fire exposure. The partial safety factor γ_f accounts for the uncertainties of the mechanical load and the thermal and mechanical material properties of the structural element, including the uncertainties of the analytical models for a determination of the load effect, the temperature state and the load bearing capacity, if the fire resistance is evaluated analytically.

The appropriate design load for evaluating the fire resistance of the structural element may be determined by considering the following accidental combination:

$$G + \sum \psi_i Q_{k,i} \quad (\text{II-9})$$

wherein

- G = permanent loads,
- $Q_{k,i}$ = variable loads (characteristic value),
- ψ_i = combination coefficients.

The load combination formula implies that the partial safety factors $\gamma_G = 1.0$ and $\gamma_{Q1} = 1.0$. Example values of the combination coefficients ψ_1 can be found in the EUROCODE publication (1984).

The detailed practical application of this probabilistic approach is given in [2] and [8].

Table II.1: Net calorific value H in $\text{MJ} \times \text{kg}^{-1}$ of some solid, liquid and gaseous materials

<u>1. Solids</u>			
Anthracite	32-36	Melamine resin	19
Asphalt	40	Phenol-formaldehyde	28
Cellulose	15	Polyester	30
Charcoal	30	Polyester, fibre	
Clothes	17-21	reinforced	21
Coal	29	Polyethylene	47
Coke	28-34	Polystyrene	42
Cork (grade SP)	35	Polyurethane	24
Cork (grade F)	31	Polyurethane-foam	25-29
Cotton	18	Polyvinyl chloride	18
Grain	17	Urea-formaldehyde	18
Grease	40	Urea-formaldehyde	
Kitchen refuse	8-21	foam	12-15
Leather	20	Rubber	
Linoleum	21	Foam rubber	32
Paper, cardboard	16-18	Gutta-percha	45
Paraffin wax	47	Rubber waste	21
Plastics		Silk	17-21
ABS-plastics	40	Straw	17
Acrylic plastics	27	Wood	17-20
Celluloid	19	Wool	23
Epoxy	34		

The above values apply to materials in a dry state. The calorific value of moist materials H_F can be determined from the formula $H_F = H(1-0.01m)-0.025m$, where m is the moisture content in % by weight.

<u>2. Liquid</u>			
Crude oil	43	Paraffin oil	41
Diesel oil	41-42	Petrol	44
Linseed oil	39	Spirits	33-34
Methanol	23	Tar	38

Table II.1: (continued)

<u>3. Gases</u> (H in MJ/m ³ n)			
Acetylene	57	Coal gas	17
Butane	110	Hydrogen	140
Carbon monoxide	13	Propane	86

Table II.2a: Fire load density q_t per unit area of the surface bounding the fire compartment, according to recent Swedish investigations. q_t according to equation II-1b [15], [16]

Type of fire compartment	Average $\text{MJ}\cdot\text{m}^{-2}$ ($\text{kcal}\cdot\text{m}^{-2}$)		Standard deviation $\text{MJ}\cdot\text{m}^{-2}$ ($\text{kcal}\cdot\text{m}^{-2}$)		Characteristic value (0.5 fractile) $\text{MJ}\cdot\text{m}^{-2}$ ($\text{kcal}\cdot\text{m}^{-2}$)	
1 Dwellings ¹⁾						
1a Two rooms and a kitchen	150	(35.3)	24.7	(5.9)	168	(40.0)
1b Three rooms and a kitchen	139	(33.1)	20.1	(4.8)	149	(35.5)
2 Offices ²⁾						
2a Technical offices	124	(29.7)	31.4	(7.5)	145	(34.5)
2b Administrative offices	102	(24.3)	32.2	(7.7)	122	(31.5)
2c All offices, investigated	114	(27.3)	39.4	(9.4)	138	(33.0)
3 Schools ²⁾						
3a Schools - junior level	84.2	(20.1)	14.2	(3.4)	98.4	(23.5)
3b Schools - middle level	96.7	(23.1)	20.5	(4.9)	117	(28.0)
3c Schools - senior level	61.1	(14.6)	18.4	(4.4)	71.2	(17.0)
3d All schools, investigated	80.4	(19.2)	23.4	(5.6)	96.3	(23.0)
4 Hospitals	116	(27.6)	36.0	(8.6)	147	(35.0)
5 Hotels ²⁾	67.0	(16.0)	19.3	(4.6)	81.6	(19.5)

1) Floor covering excluded

2) Only moveable fire load components included

Table II.2b: Fire load density q_f per unit floor area, according to [21]. Characteristic values (0.8 fractile), calculated on the basis of the same statistical survey as used for the values in table II.2a. q_f according to equation II-1a

Type of fire compartment	Characteristic value (0.8 fractile) $\text{MJ} \times \text{m}^{-2}$ ($\text{Mcal} \times \text{m}^{-2}$)	
1 Dwellings		
1a Bedroom	630	(150)
1b Living-room	510	(120)
2 Offices		
2a Technical offices	720	(170)
2b Administrative offices	640	(155)
3 Schools		
3a Schools - junior level	370	(90)
3b Schools - middle level	400	(95)
3c Schools - senior level	260	(60)
4 Hospitals - bedroom	80	(20)
5 Hotels - bedroom	420	(100)

Only movable fire load components included

Table II.2c: Fire load density q_f per unit floor area, equation II-1a, for various types of fire compartments in German office buildings, according to [13]

Type of fire compartment	Average		Standard deviation	
	$\text{MJ}\cdot\text{m}^{-2}$	$(\text{Mcal}\cdot\text{m}^{-2})$	$\text{MJ}\cdot\text{m}^{-2}$	$(\text{Mcal}\cdot\text{m}^{-2})$
Company management	272	(64.8)	126	(30)
Production management	355	(84.8)	168	(40)
Officials	441	(105)	250	(60)
Office staff	417	(99.6)	210	(50)
Special rooms	1172	(280)	798	(190)
Technical rooms	278	(66.4)	109	(26)
Rooms of communication	168	(40)	240	(57)
All types of compartments	411	(98)	324	(80)

Only moveable fire load components included. In [13], the fire load density is given in kg wood per m^2 . The values, given in the table, are based on a net calorific value of wood $H = 4 \text{ Mcal} \times \text{kg}^{-1}$

Table II.3: Coefficient K_f for transforming a real fire load density q_c and a real opening factor of a fire compartment $A_w \sqrt{h_w}/A_c$ to an effective fire load density q_{cf} and an effective opening factor $(A_w \sqrt{h_w}/A_c)_f$ corresponding to a fire compartment, type A - equation II-4

$$q_{cf} = K_f q_c \quad (A_w \sqrt{h_w}/A_c)_f = K_f A_w \sqrt{h_w}/A_c$$

Type of fire compartment	Opening factor $A_w \sqrt{h_w}/A_c \quad m^{1/2}$					
	0.02	0.04	0.06	0.08	0.10	0.12
Type A	1	1	1	1	1	1
Type B	0.85	0.85	0.85	0.85	0.85	0.85
Type C	3.00	3.00	3.00	3.00	3.00	2.50
Type D	1.35	1.35	1.35	1.50	1.55	1.65
Type E	1.65	1.50	1.35	1.50	1.75	2.00
Type F 1)	1.00-	1.00	0.80-	0.70-	0.70-	0.70-
	0.50	0.50	0.50	0.50	0.50	0.50
Type G	1.50	1.45	1.35	1.25	1.15	1.05
Type H	3.00	3.00	3.00	3.00	3.00	2.50

1) The lowest value of K_f applies to a fire load density $q_c > 500 \text{ MJ} \times m^{-2}$, the highest value to a fire load density $q_c < 60 \text{ MJ} \times m^{-2}$. For intermediate fire load densities, linear interpolation gives sufficient accuracy.

For an explanation of the various types of fire compartment, specified in the table, it is referred to pages 39 and 40.

Fire compartment, type A: Bounding structures of a material with a thermal conductivity $\lambda_b = 0.81 \text{ W m}^{-1} \text{ }^{\circ}\text{C}^{-1}$ and a heat capacity $pc_b = 1.67 \text{ MJ m}^{-3} \text{ }^{\circ}\text{C}^{-1}$.

Fire compartment, type B: Bounding structures of concrete.

Fire compartment, type C: Bounding structures of aerated concrete (density $\rho = 500 \text{ kg m}^{-3}$).

Fire compartment, type D: 50% of the bounding structures of concrete, and 50% of aerated concrete (density $\rho = 500 \text{ kg m}^{-3}$).

Fire compartment, type E: Bounding structures with the following percentage of bounding surface area:

50% aerated concrete (density $\rho = 500 \text{ kg m}^{-3}$),
33% concrete, and
17%, from the interior to the exterior, of plasterboard panel (density $\rho = 790 \text{ kg m}^{-3}$) 13 mm in thickness, diabase wool (density $\rho = 50 \text{ kg m}^{-3}$) 10 cm in thickness, and brickwork (density $\rho = 1800 \text{ kg m}^{-3}$) 20 cm in thickness.

Fire compartment, type F: 80% of the bounding structures of sheet steel, and 20% of concrete. The compartment corresponds to a storage space with a sheet steel roof, sheet walls, and a concrete floor.

Fire compartment, type G: Bounding structures with the following percentage of bounding surface area:

20% concrete, and
80%, from the interior to the exterior, of double plasterboard panel (density $\rho = 790 \text{ kg m}^{-3}$) 2 x 13 mm in thickness, air space 10 cm in thickness, and double plasterboard panel (density $\rho = 790 \text{ kg m}^{-3}$) 2 x 13 mm in thickness.

Fire compartment, type H: Bounding structures of sheet steel on both sides of diabase wool (density $\rho = 50 \text{ kg m}^{-3}$) 10 cm in thickness.

For fire compartments, not directly represented in the table, the coefficient K_f can either be determined by a linear interpolation between applicable types of fire compartment in the table or be chosen in such a way as to give results on the safe side. For fire compartments with surrounding structures of both concrete and lightweight concrete, then different values can be obtained of the coefficient K_f , depending on the choice between the fire compartment types B, C and D at the interpolation. This is due to the fact that the relationships, determining K_f , are non-linear. However, the K_f -values of the table are such that a linear interpolation always gives results on the safe side, irrespective of the alternative of interpolation chosen. In order to avoid an unnecessarily large overestimation of K_f , that alternative of interpolation is recommended which gives the lowest value of K_f . In determining K_f , it is not acceptable to combine types of fire compartments in such a way as to give a negative contribution to K_f .

III TEMPERATURE DEVELOPMENT IN FIRE-EXPOSED STEEL MEMBERS

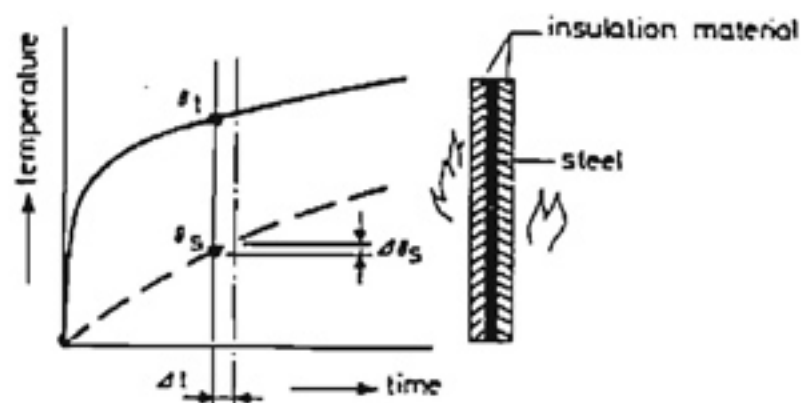
III.1 Introduction

When estimating structural behaviour in building fires, it is generally necessary to calculate transient heat conduction to the steel section. Where an analytical solution is not available, it is possible to adopt a quasi-stationary approach, iterated for successive time intervals of $\Delta t(s)$. This calculation method can be simplified by considering the steel to be a heat sink, with negligible resistance to heat flow; thus any heat supplied to the steel section is considered to be instantly distributed to give a uniform steel temperature.

A further simplification is to consider the insulating material, if any, to have negligible thermal capacity and to provide only resistance to heat flow.

III.2 Basic heat conduction equation

Consider a steel plate protected on two faces by a material of thickness $d_i(m)$, this material being exposed to gases heated to follow the standard temperature-time curve given by equation II.5. The variation of gas temperature and steel temperature with time is illustrated in Fig. III.1:



Temperature development in a fire exposed insulated steel plate.

Fig. III.1

Assuming no heat flow at the edges of the plate, then the heat flow is one-dimensional and total heat ΔQ (J) transferred to the steel in time Δt is given by:

$$\Delta Q = K \cdot F \cdot (\theta_t - \theta_s) \Delta t \quad (\text{III.1})$$

where K = total heat transfer coefficient ($\text{W/m}^2 \cdot ^\circ\text{C}$)

F = surface area of steel plate
(excluding edges) (m^2)

θ_t = ambient gas temperature during time
interval Δt ($^\circ\text{C}$).

θ_s = steel temperature during time
interval Δt ($^\circ\text{C}$)

The temperature rise, $\Delta\theta_s$ ($^\circ\text{C}$), of the steel is given by:

$$\Delta\theta_s = \frac{\Delta Q}{V c_s \rho_s} \quad (\text{III.2})$$

Where V = volume of steel plate (m^3)

c_s = specific heat of steel ($\text{J/kg} \cdot ^\circ\text{C}$)

ρ_s = density of steel (kg/m^3)

Combining (III.1) and (III.2) gives

$$\Delta\theta_s = \frac{K}{c_s \rho_s} \frac{F}{V} (\theta_t - \theta_s) \Delta t \quad (\text{III.3})$$

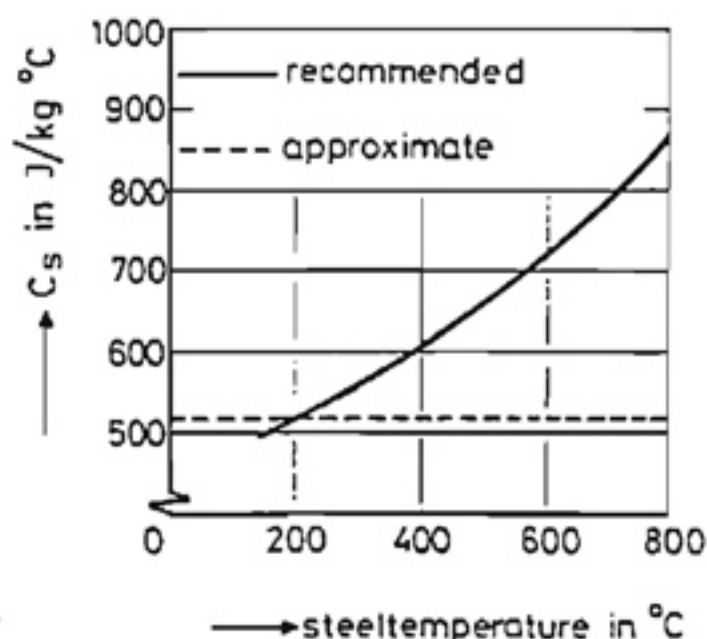
The value of K depends on the heat transfer coefficients for radiation and convection, α_r and α_c ($\text{W/m}^2 \cdot ^\circ\text{C}$) respectively, and on the thermal conductivity, λ_1 ($\text{W/m} \cdot ^\circ\text{C}$), and the thickness, d_1 (m), of the protective material i.e.

$$K = \frac{1}{\left(\frac{1}{\alpha_c + \alpha_r}\right) + \left(\frac{d_1}{\lambda_1}\right)} \quad (\text{III.4})$$

The value of ρ_s is constant and may be taken as 7850 kg/m^3 . The value of c_s varies with temperature, as illustrated in Fig. III.2. The curve which is recommended for use in calculations, is given by:

$$c_s = 38 \times 10^{-5} \times \theta_s^2 + 20 \times 10^{-2} \times \theta_s + 470 \text{ (J/kg} \cdot ^\circ\text{C)} \quad (\text{III.5})$$

For approximate calculations, a conservative value for c_s of 520 (J/kg °C) may be used. For a tabulated review of c_s as function of steel temperature, it is referred to Table D-1^a.

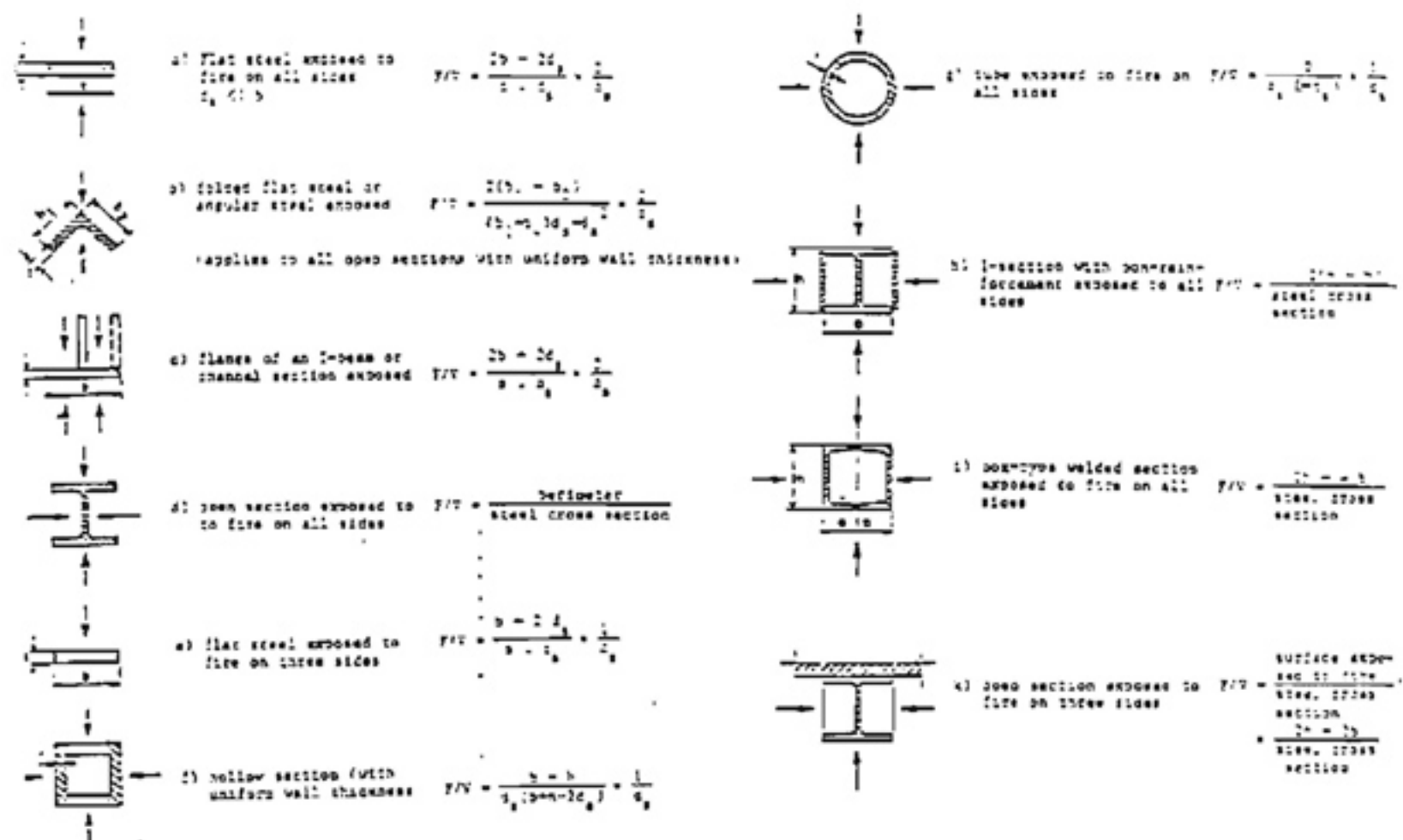


Specific heat of steel as a function of the steel temperature.

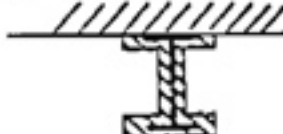
Fig. III.2

In practice, the steel section will be heated on three sides (beams) or four sides (columns and beams). The heat conduction equation can still be one-dimensional if it is assumed that there is no heat loss at the ends of the sections and the heat conduction from upper flange of a beam to the slab above can be neglected. If the exposed surface area per unit length is denoted by F (m²/m) and the volume per unit length by V (m³/m) then F/V (m⁻¹) becomes the section factor. The determination of section factor is illustrated in Fig. III.3 for non insulated steel sections.

For protected steel sections, it is appropriate to define the exposed area as the fire exposed area of inner surface over the insulation material per unit length denoted as F_1 (m²/m). Fig. III.4 illustrates how F_1/V can be determined.



Section factor F/V for non insulated steel members.
Fig. III.3

Description		section factor (F_1/V)
	contour encasement of uniform thickness	$\frac{\text{steel perimeter}}{\text{steel cross section}}$
	hollow encasement of uniform thickness	$\frac{2(b + h)}{\text{steel cross section}}$
	contour encasement of uniform thickness exposed to fire on three sides	$\frac{\text{steel perimeter excl. width of one flange}}{\text{steel cross section}}$

Section factor F_1/V for insulated steel members.
Fig. III.4

III.3 Heat transfer to unprotected sections

III.3.1 Heat transfer coefficient

When there is no cladding, i.e. $d_1 = 0$, the heat transfer coefficient is:

$$K = \alpha = \alpha_c + \alpha_r \quad (\text{III.6})$$

where, for the standard test, convection transfer is given by

$$\alpha_c = 25 \quad \begin{matrix} \cdot \\ \cdot \\ \cdot \\ \cdot \end{matrix} \quad (\text{III.7a})$$

and radiation transfer by:

$$\alpha_r = \left(\frac{5.67 \epsilon_r}{(\theta_c - \theta_s)} \right) \left[\left(\frac{\theta_c + 273}{100} \right)^4 - \left(\frac{\theta_s + 273}{100} \right)^4 \right] \quad (\text{III.7b})$$

The resultant emissivity of the flames, combustion gases and steel surface is denoted by ϵ_r , and its value depends on the type of fire and the position of the exposed member. An approximate value, which gives a conservative solution is:

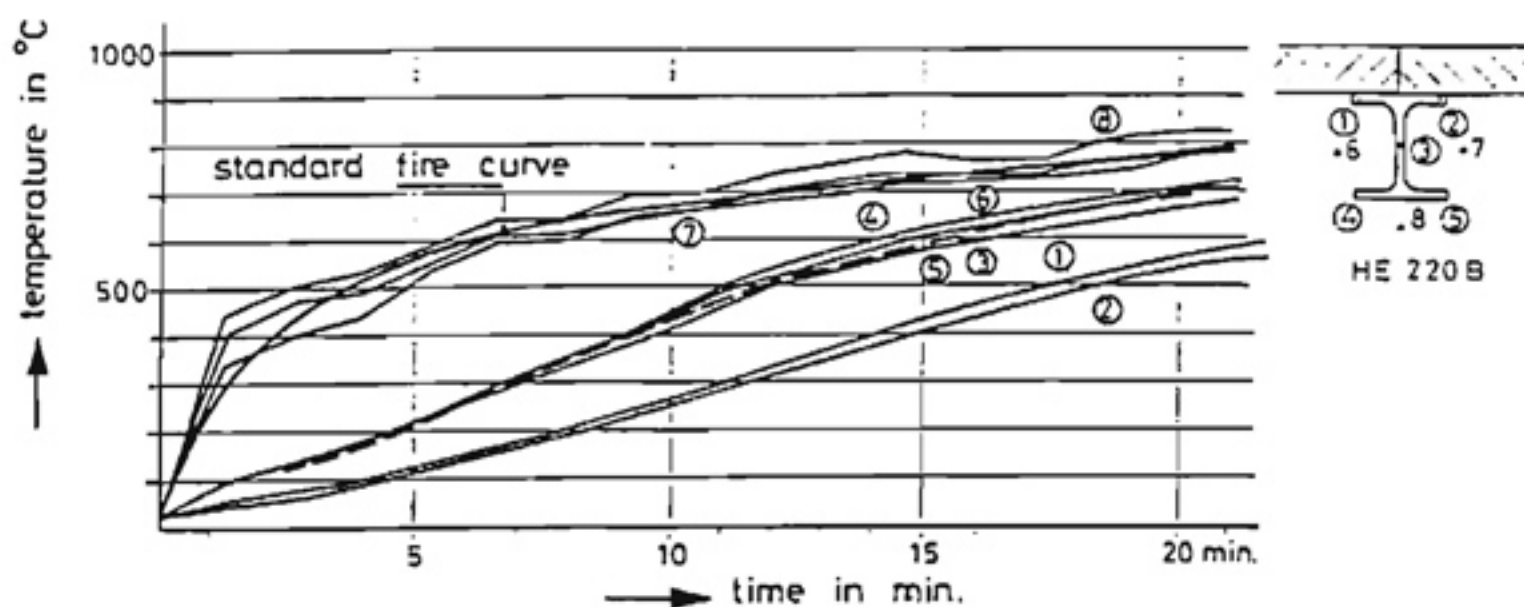
$$\epsilon_r = 0.5$$

Equation III.3 therefore becomes:

$$\Delta \theta_s = \frac{\alpha}{c_s \rho_s} \cdot \frac{F}{V} (\theta_c - \theta_s) \Delta \tau \quad (\text{III.8})$$

III.3.2 Background to calculations

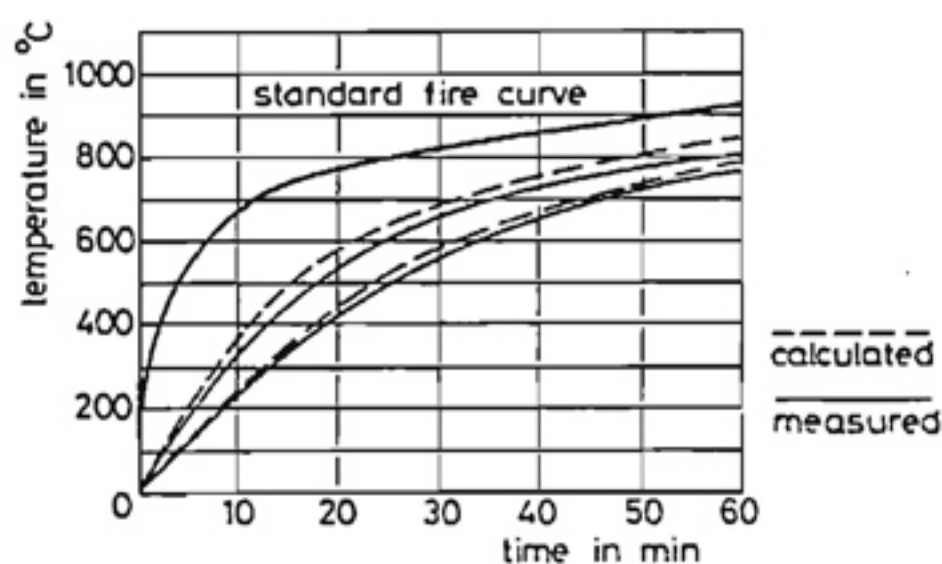
The temperature development of an unprotected steel beam in a standard test carried out in one laboratory is illustrated in Fig. III.5.



Measured (—) and calculated (---) steel temperature development in a non-insulated steel girder.

Fig. III.5

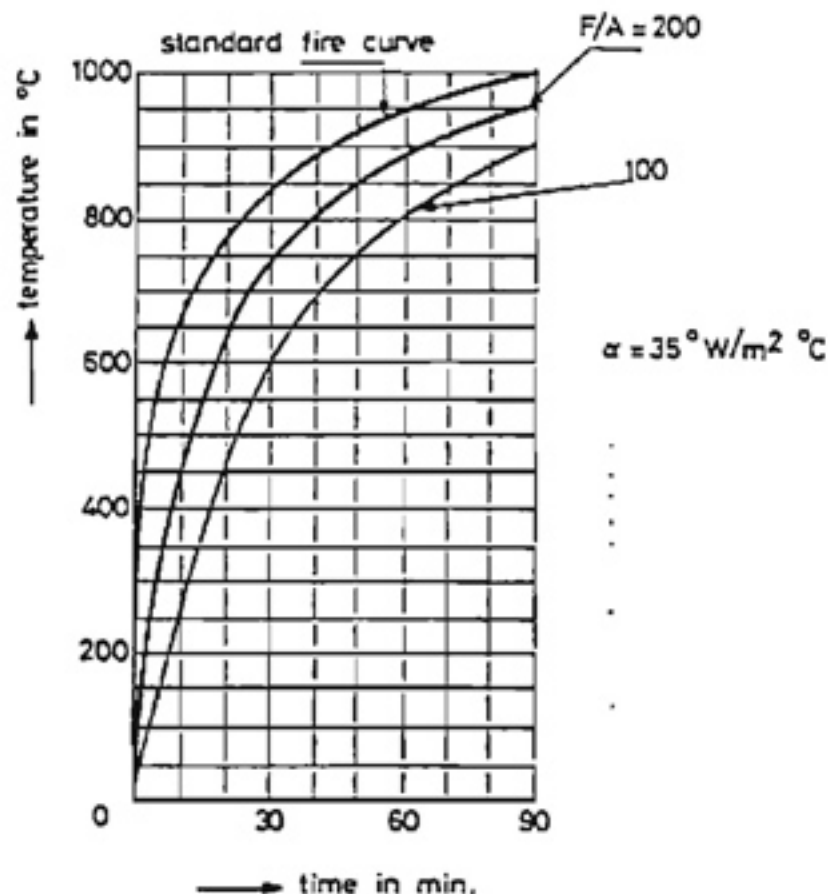
The calculated curve uses equation III.6 for α , with $c_r = 0.5$. In some test furnaces the radiative contribution to heat transfer is small. This is illustrated in Fig. III.6, which shows a test result from another laboratory. The calculated curve is based on a constant value for α of $35 \text{ W/m}^2 \text{ } ^\circ\text{C}$, which gives better agreement than equation III.6.



Measured (—) and calculated (---) steel temperature development in two non-insulated steel sections.

Fig. III.6

Fig. III.7 illustrates the effect of the value of the section factor F/V on the temperature attained for $\alpha = 35 \text{ W/m}^2 \text{ } ^\circ\text{C}$.



Temperature development in non-insulated steel sections for various values of the section factor F/V . ($\alpha = 35 \text{ W/m}^2 \text{ } ^\circ\text{C}$)

Fig. III.7

The calculated variation of steel temperature with time, t , for different values of section factor F/V is given in Appendix D, Table D.2. In these calculations $\epsilon_r = 0.5$, $\Delta t = 30\text{s}$, α is given by equations III.7a and III.7b and c_s by equation III.5.

III.4 Heat transfer to protected sections

III.4.1 Heat transfer coefficient

When there is a cladding applied to the steel section the heat transfer coefficient K is:

$$K = \frac{1}{\frac{1}{\alpha} + \frac{d_1}{\lambda_1}} \quad (\text{III.9})$$

where

$$\alpha = \alpha_r + \alpha_c$$

In practice, the value of $(1/\alpha)$ is usually small compared with (d/λ_1) and K is then given by:

$$K = \frac{\lambda_1}{d_1} \quad (\text{III.10})$$

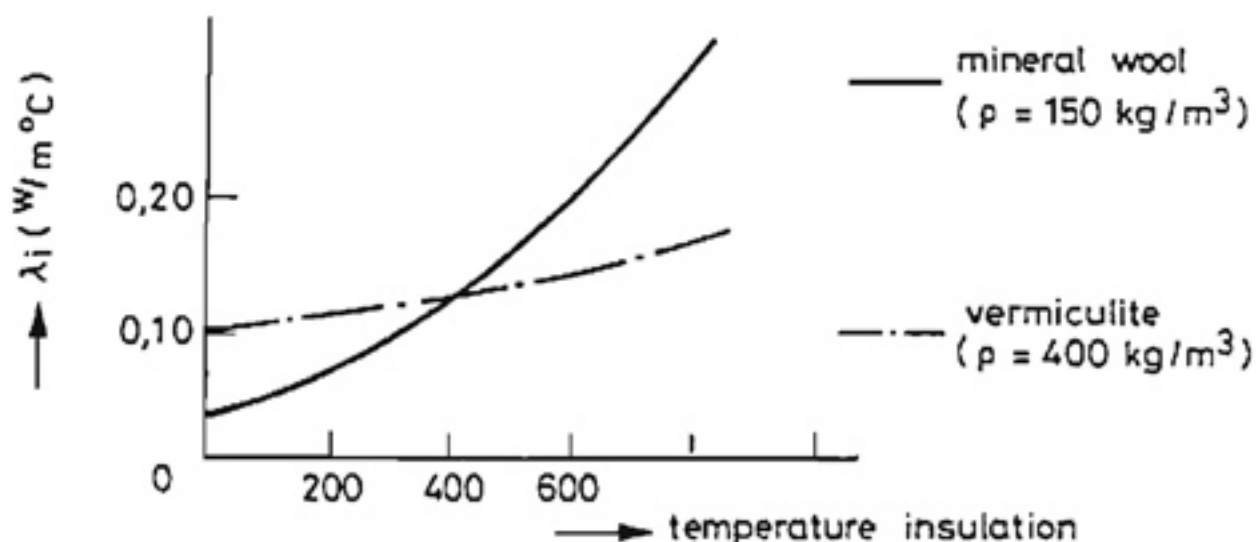
Equation III.4 then becomes:

$$\Delta\theta_s = \frac{d_1/\lambda_1}{c_s \rho_s} \cdot \frac{F_1}{V} (\theta_c - \theta_s) \Delta t \quad (\text{III.11})$$

The calculated variation of steel temperature with time t for different values of section factor F_1/V and different values of d_1/λ_1 is given in Table D.3. In these calculations $\Delta t = 30s$ and c_s is given by equation III.5.

III.4.2 Background to calculations

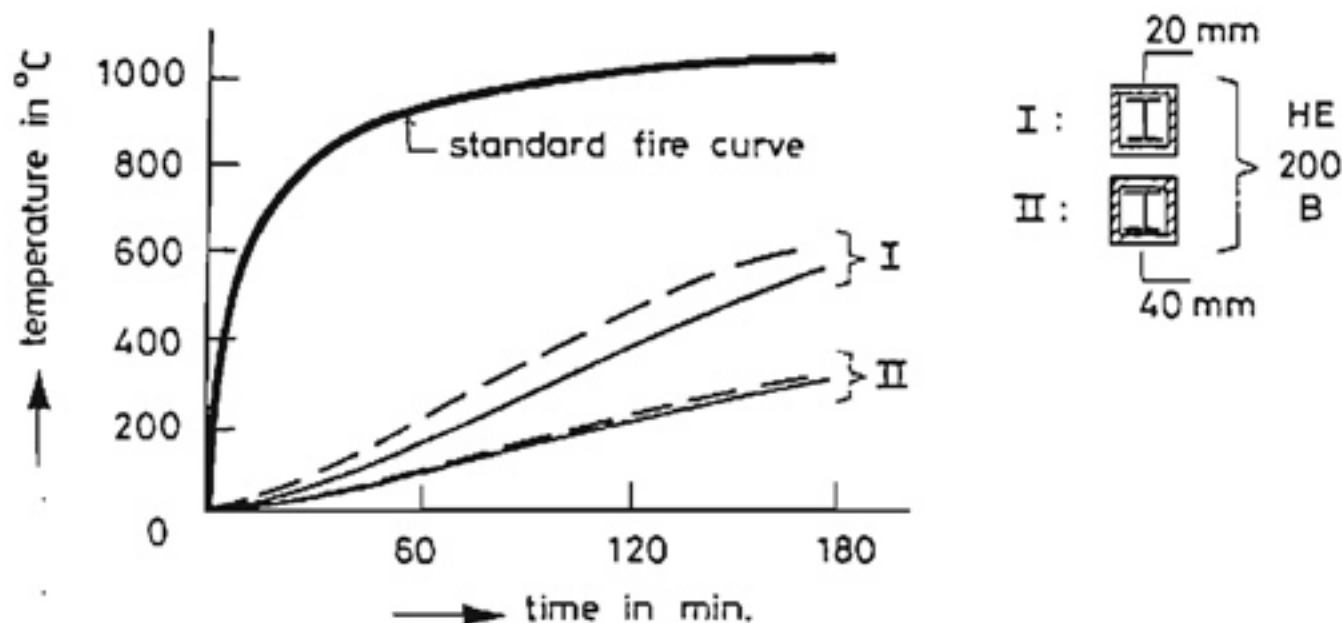
For most insulating materials the value of thermal conductivity λ_1 varies with temperature, as illustrated in Fig. III.8



The thermal conductivity (λ_1) of two insulation materials as a function of the temperature of the insulation [2].

Fig. III.8

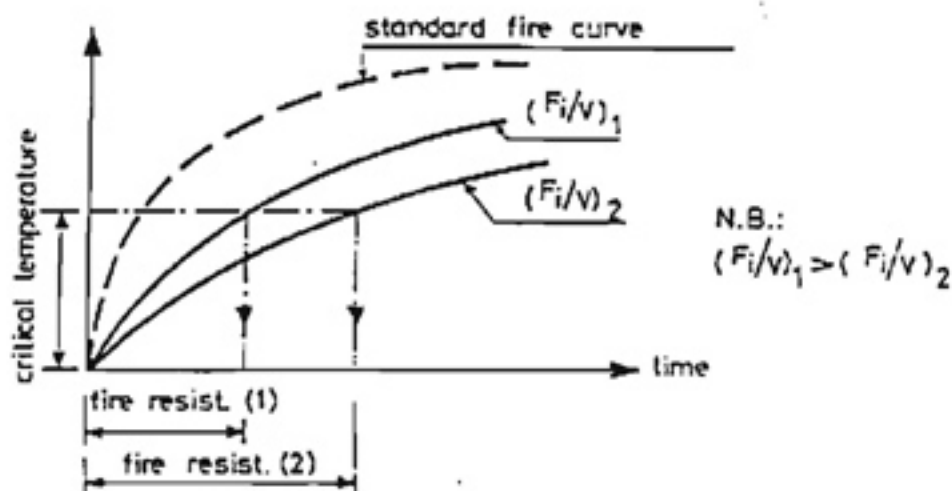
Equation III.11 can be used taking into account this variation of λ_i with temperature and an example of a calculation of steel temperature for two thicknesses of an insulating material is given in Fig. III.9, compared with the values measured in a standard test.



Measured (—) and calculated (---) steel temperature in insulated steel sections for different values of the insulation thickness.

Fig. III.9

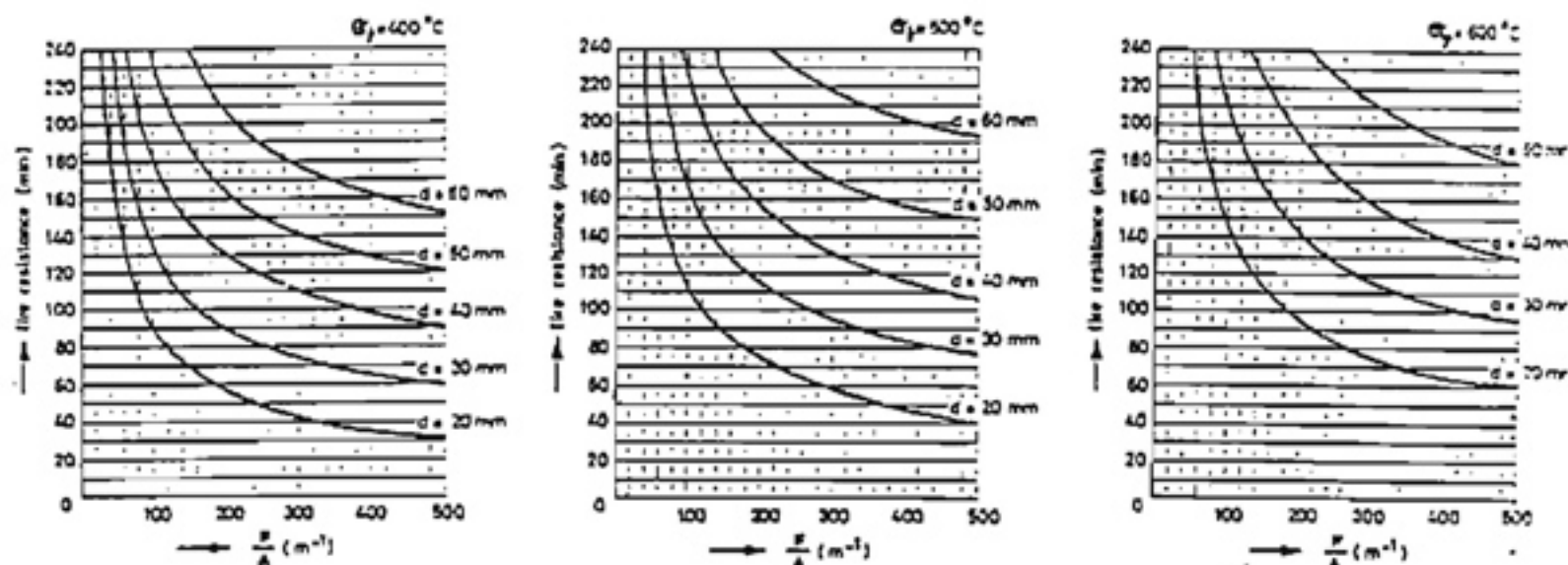
For a given thickness of an insulating material, the steel temperature attained will depend on the section factor, F_i/V , as illustrated in Fig. III.10: the higher value of section factor heats up more quickly.



Development of steel temperature for one insulation thickness and different section factors. (qualitative).

Fig. III.10

It is possible, therefore, to construct charts which show how steel temperature varies with d_i , F_i/V and t for a given insulating material, as illustrated in Fig. III.11.



Fire resistance of insulated steel profiles as function of the insulation thickness and the section factor for critical temperatures of 400, 500 and 600°C [3].

Fig. III.11

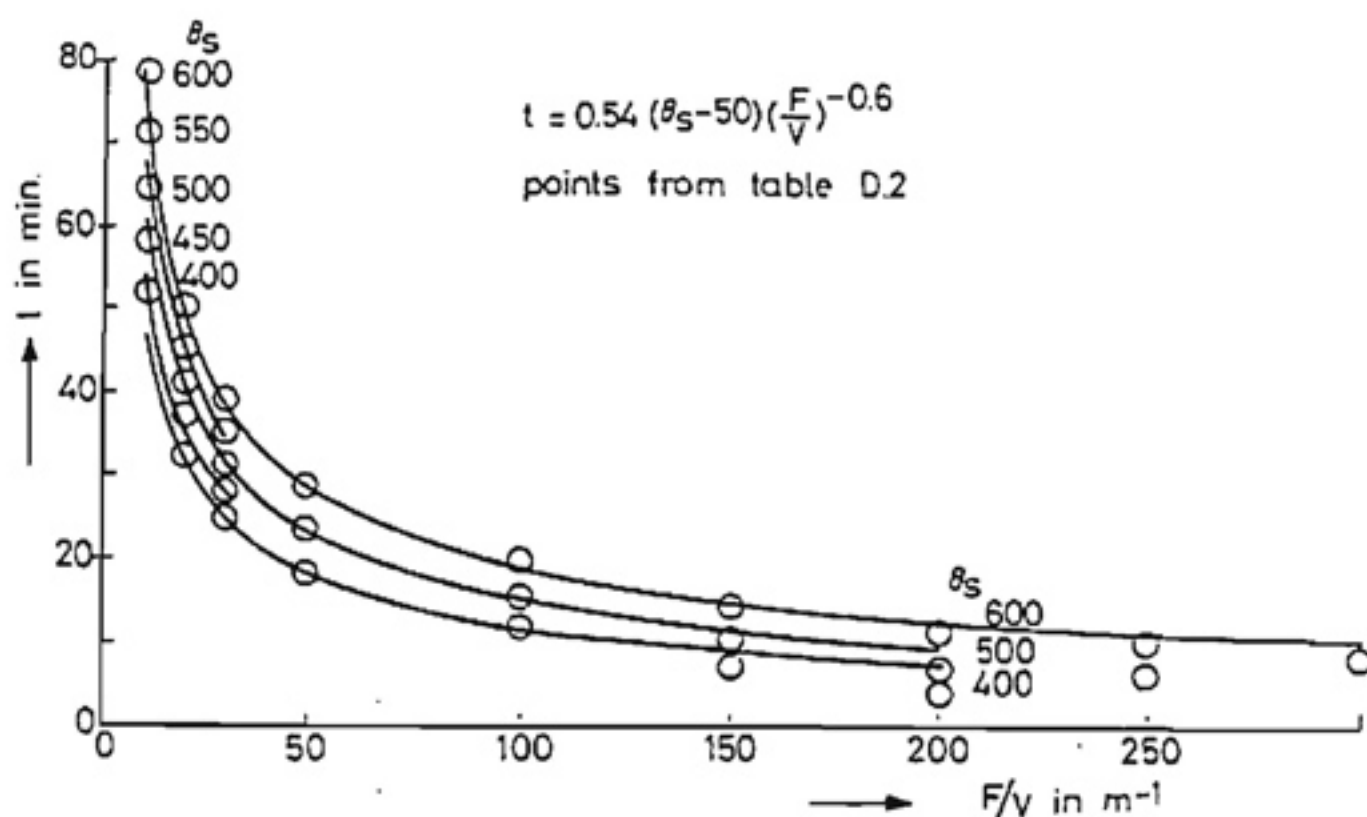
III.5 Practical applications

Charts and calculation methods for practical use have been developed by a number of workers in the past [7,8,9,10, 11]. Similar methods are described below.

III.5.1 Unprotected sections

In practice, the critical steel temperature will usually fall within the range 400-600°C. Fig. III.12 shows how the time to reach a given θ_s within this range varies with section factor. This has been derived, by interpolation, from Table D-2. Also shown is a series of curves given by the following equation:

$$t = 0.54 (\theta_s - 50) (F/V)^{-0.6} \quad (\text{III.12})$$



Calculated temperature of members without insulation.

Fig. III.12

This equation is a good approximation to the calculated values of t exceeding 10 minutes. Some test data for members without insulation [4,5,6] are given in Fig. III.13 for a steel temperature of 550°C together with the curve given by equation III.12 for $\theta_s = 550^\circ\text{C}$.

This illustrates that the curves give reasonable agreement with test data. Equation III.12 may be expressed in three ways as shown below:

$$t = 0.54 (\theta_s - 50) (F/V)^{-0.6}$$

$$\text{or } \theta_s = 1.85t (F/V)^{0.6} + 50$$

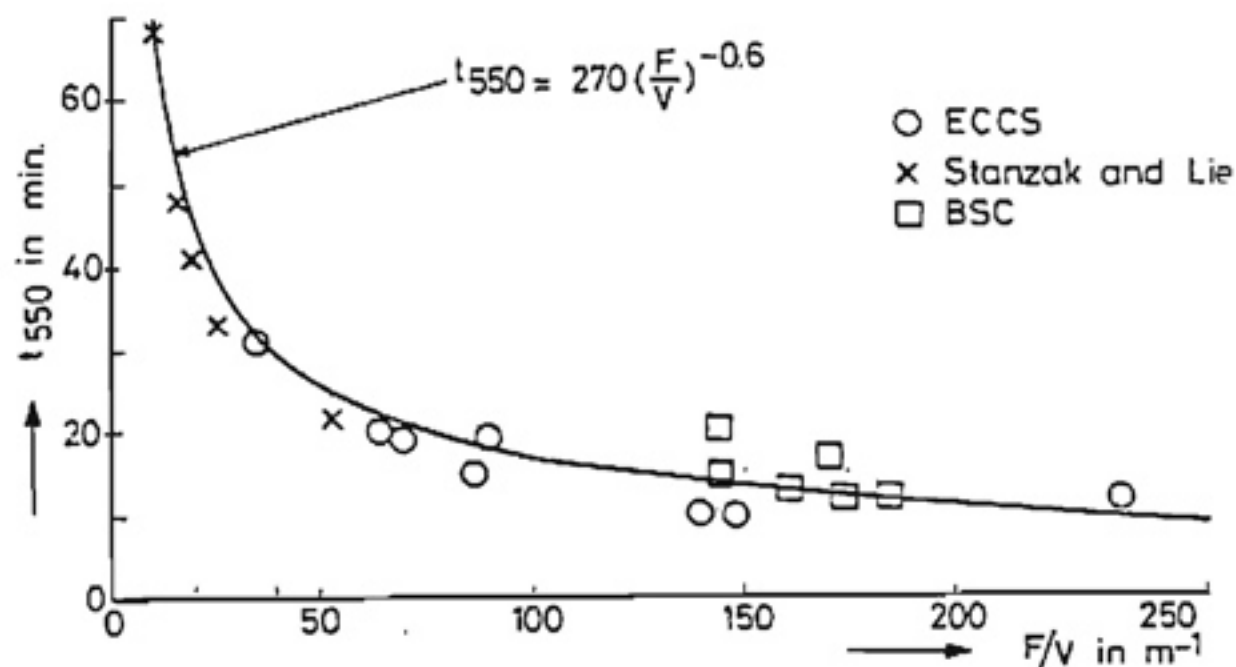
$$\text{or } F/V = 0.36 \left(\frac{\theta_s - 50}{t} \right)^{1.67}$$

Valid for

$$t = 10-80 \text{ min.}$$

$$\theta_s = 400-600^\circ\text{C}$$

$$F/V = 10-300 \text{ m}^{-1}$$



Measured temperatures of members without insulation.
[4, 5, 6]

Fig. III.13

III.5.2 Protected sections

In practice it is often possible to use an average value of λ_1 , derived for a specific temperature range, in calculations. Typical values are listed in Table III.1. These may be used when no other information is available. The values given for λ_1 do not include any allowance for the effect of thermal capacity of the material.

Material	Density ρ_{13} kg/m ³	Thermal conductivity λ_1 W/m ¹ .C
Sprayed mineral fibre	250-350	0.10
Perlite on 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

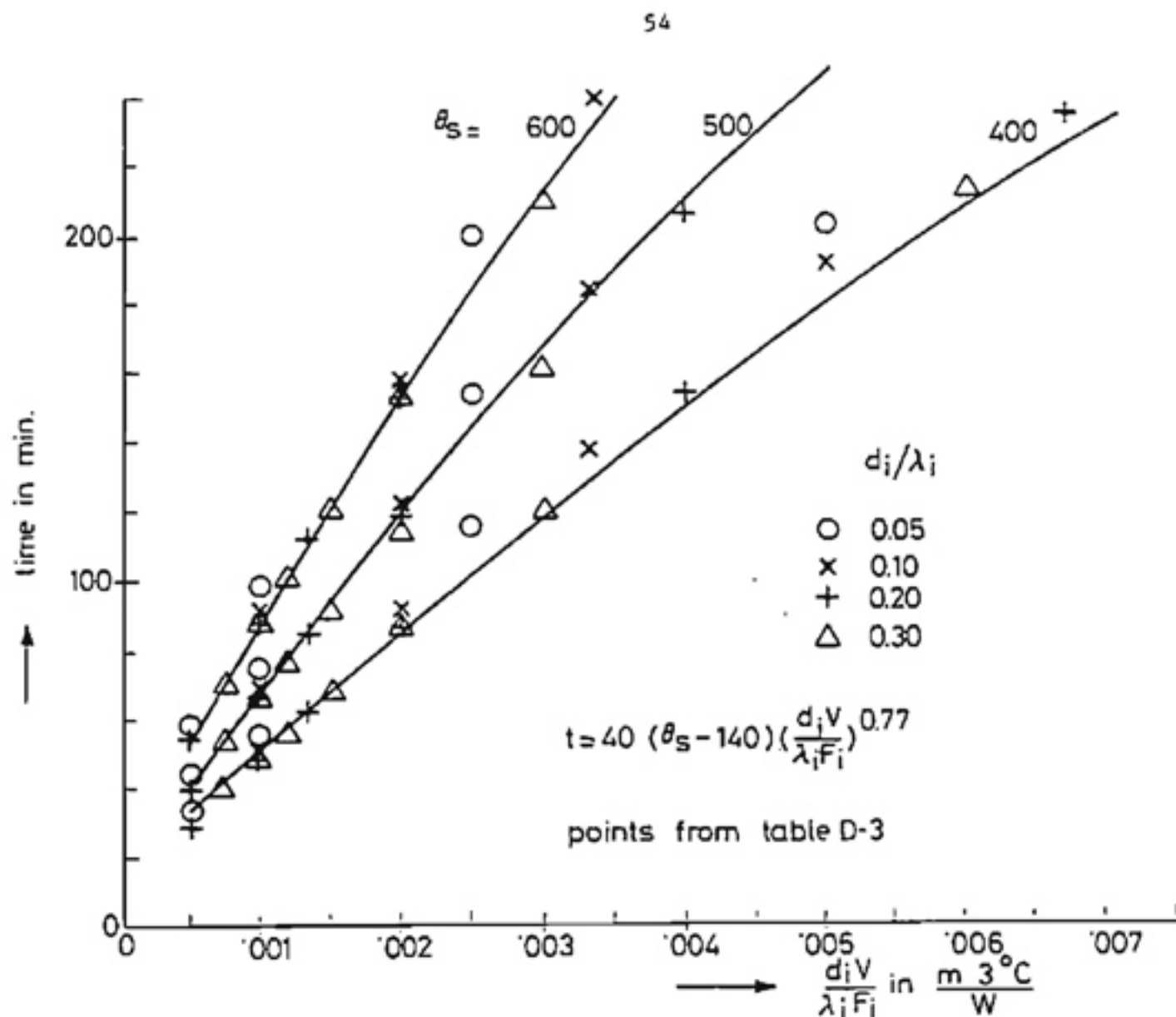
Table III.1: Approximate values for the thermal conductivity of various insulation materials under fire conditions.

For a practical range of critical steel temperatures, 400 to 600°C and a constant λ_1 , the time to attain a given value of θ_s depends mainly on the product $(d_1/\lambda_1) \times (V/F_1)^*$. This is illustrated in Fig. III.14 where the calculated values have been derived by interpolation from Table D.3.

The curves drawn have the equation

$$t = 40 (\theta_s - 140) \left(\frac{d_1 V}{\lambda_1 F_1} \right)^{0.77} \quad (\text{III.13})$$

*) Using equation III.9 for K. If equation III.10 is used for K, the time depends only on the product $(d_1/\lambda_1) \times (V/F_1)$.



Calculated temperature of members with insulation.

Fig. III.14

Equation III.13 can be expressed in three ways:

$$t = 40 (\theta_s - 140) \left(\frac{d_1 V}{\lambda_1 F_1} \right)^{0.77}$$

$$\theta_s = 0.025 t \left(\frac{d_1 V}{\lambda_1 F_1} \right)^{-0.77} + 140$$

$$\frac{d_1 V}{\lambda_1 F_1} = 0.0083 \left(\frac{t}{\theta_s - 140} \right)^{1.3}$$

and is valid for:

$$30 < t < 240 \text{ min}$$

$$400 < T_s < 600^\circ\text{C}$$

$$10 < V/F_1 < 300 \text{ m}^{-1}$$

$$0.10 < d_1 / \lambda_1 < 0.3 \text{ m}^2 \text{ } ^\circ\text{C/W}$$

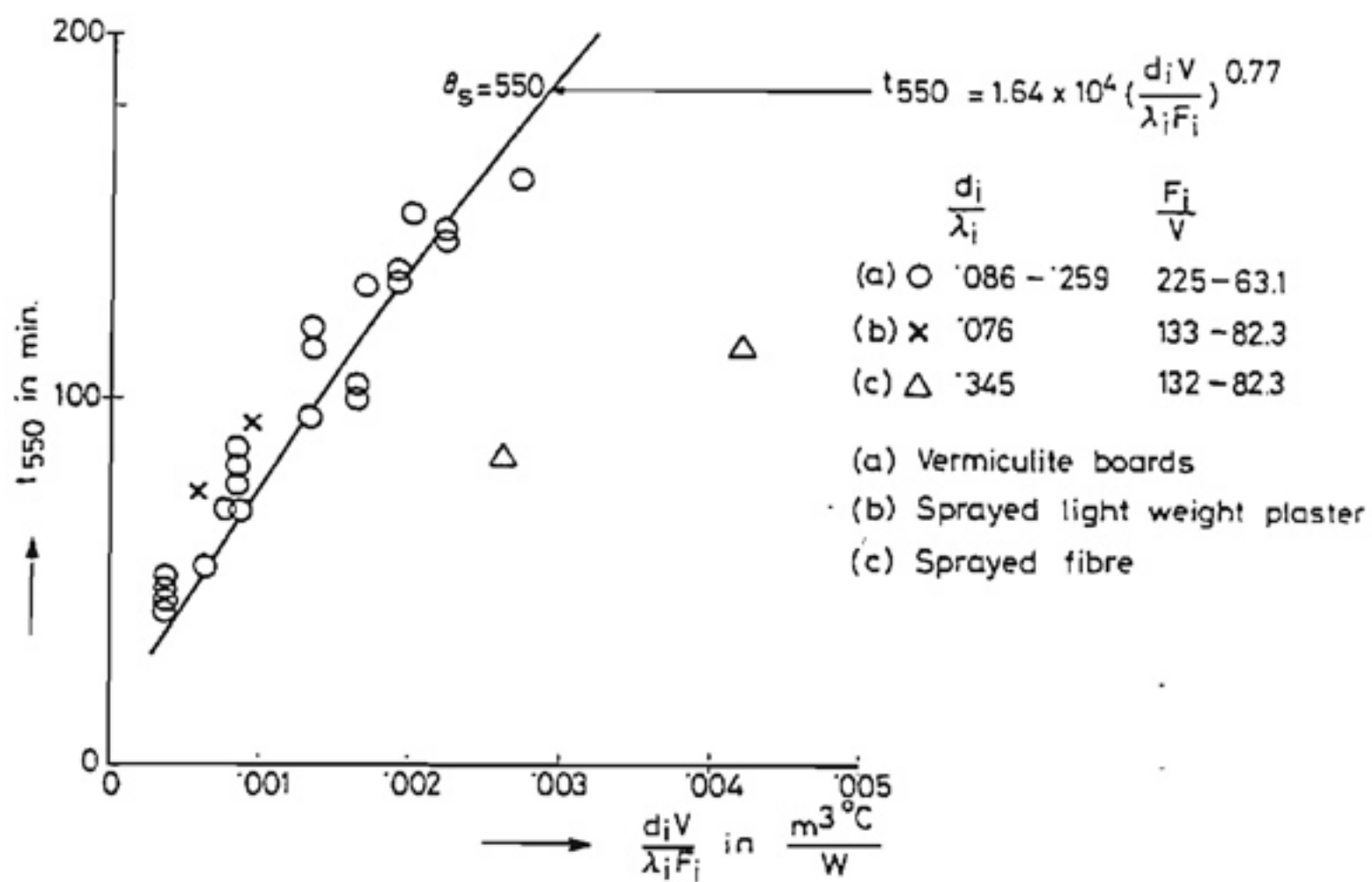
III.5.3 Test results

In practice the value of λ_1 will depend on the temperature rise and other physical changes which take place in the insulating material. However it is usually possible to define a nominal value of λ_1 which may be used in equation III.13. Results for three insulating materials are plotted in Fig. III.15 for a steel temperature of 550°C. The three materials were stated to have the following nominal values of λ_1 :

Material	λ_1 -W/m °C
Vermiculite boards	0.116
Sprayed light weight plaster	0.197
Sprayed fibre	0.058

The calculated curve is in reasonable agreement with the experimental values for the first two materials but the nominal value of λ_1 for the sprayed fibre appears to be incorrect.

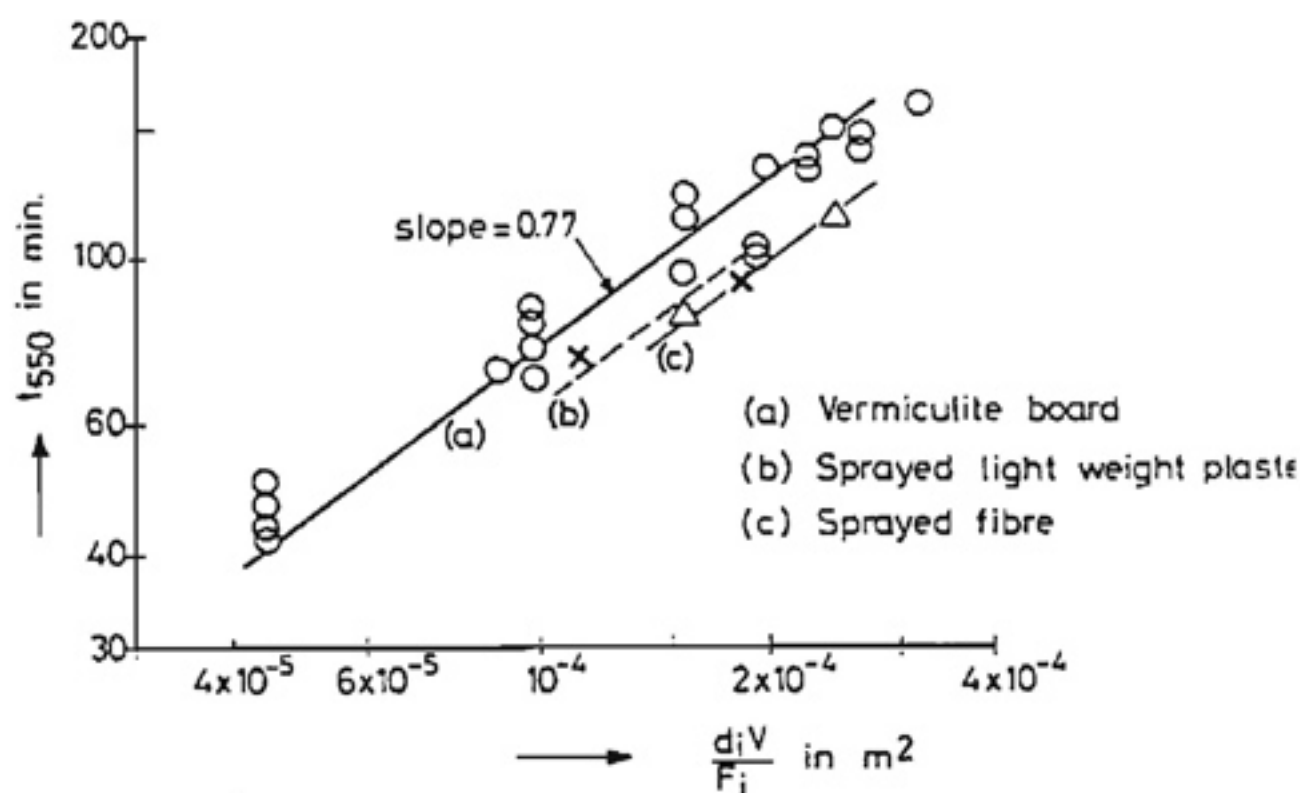
When test data are available, it is therefore better to use these directly.



Measured times for temperature of members with insulation to reach 550°C. Nominal values of λ_i assumed. [4]

Fig. III.15

When two or more test results are available for a given material they can be plotted on log-log paper as illustrated in Fig. III.16. If equation III.13 is applicable, then straight lines with a slope of 0.77 can be drawn through the points. A nominal value of λ_i can then be deduced, or the line can be used directly. These methods can be used for different values of θ_s , according to the defined critical temperature.



Measured times for temperature of members with insulation to reach 550°C. Values of λ_i not assumed.

Fig. III.16

It is also clear that scaling methods can be applied. For example for one material (with a single value of λ_i) the time to attain a given value of θ_s is proportional to $(d_i V / F_i)^{0.77}$

$$\text{i.e. } \frac{\tau_2}{\tau_1} = \left[\frac{(d_i V / F_i)_2}{(d_i V / F_i)_1} \right]^{0.77}$$

where τ_1 is the test result for $(d_i V / F_i)_1$ and τ_2 is the predicted result for $(d_i V / F_i)_2$.

Alternatively

$$\frac{(d_i V / F_i)_2}{(d_i V / F_i)_1} = \left[\frac{\tau_2}{\tau_1} \right]^{1.3}$$

A scaling method should be used with caution when extrapolating to values of τ_2 significantly greater than τ_1 (the test result). It may be noted that the methods in

this section do not explicitly take into account the effect, if any, of the thermal capacity of the insulation. This is discussed next.

III.5.4 Heavy insulation

The data shown in Fig. III.15 are for lightweight materials, that is the thermal capacity of the insulating material is small compared with that of the steel. For unit length of section, the thermal capacity of the steel is $V\rho_s c_s$ and the insulating material $d_i F_i \rho_i c_i$, where ρ_i and c_i are the density and specific heat respectively of the material. The material can be considered lightweight if its thermal capacity is not more than half that for the steel:

$$d_i F_i \rho_i c_i < \frac{c_s \rho_s V}{2}$$

In Table III.2 approximate values for the specific heat c_i and the density ρ_i of insulation materials under fire conditions are given.

Material	Density ρ_i kg/m ³	Specific heat c_i J/kg ¹ ·C
Insulation materials with 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 III.2: Approximate values for the specific heat c_i and the density ρ_i of insulation materials under fire conditions.

Since, as shown in Table III.2, for most insulating materials:

$$c_i \approx 2 c_s$$

we obtain

$$d_i \rho_i \frac{F_i}{V} < \frac{\rho_s}{4} = \frac{7850}{4}$$

and

$$\therefore d_i \rho_i \frac{F_i}{V} < 1960 \text{ kg/m}^3$$

If the material is not lightweight, equation III.13 can be used by substituting

$$\left(\frac{d_i V}{F_i} \right)_{\text{mod}}$$

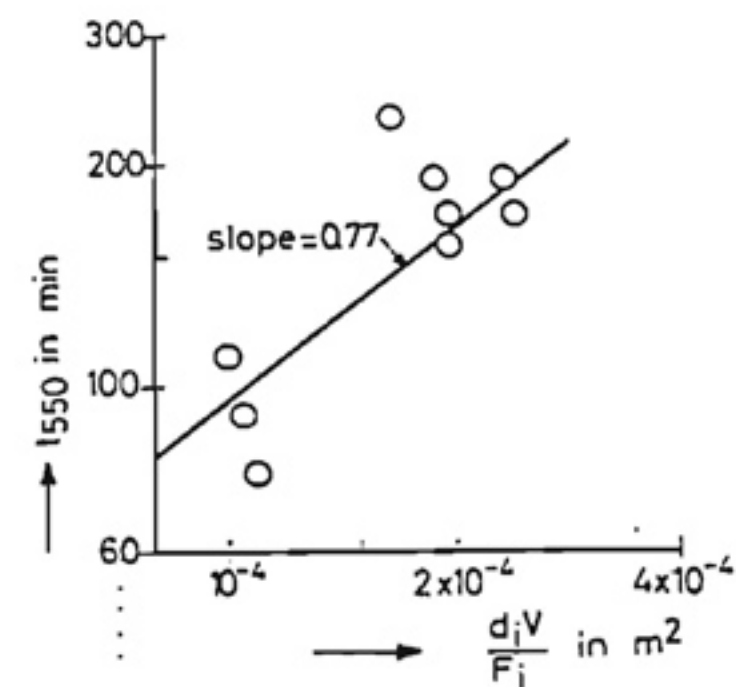
where

$$\left(\frac{d_i V}{F_i} \right)_{\text{mod}} = \frac{d_i V}{F_i} + \frac{c_i \rho_i d_i^2}{2 c_s \rho_s} \quad (\text{III.14})$$

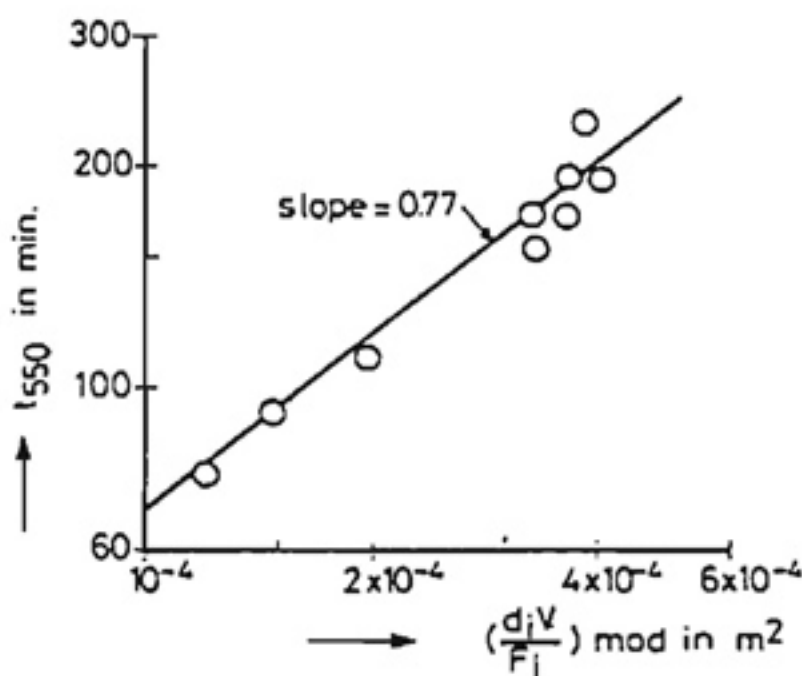
This modification in effect increases the thermal capacity of the steel by half the thermal capacity of the insulating material.

Fig. III.17 shows results of tests with a sprayed material with density (ρ_i) of 750 kg/m^3 . Without modification

- Fig. III.17a - the relationship is poor. When t_{550} is plotted against $(d_i V/F_i)_{\text{mod}}$ - Fig. III.17b -, there is a good relationship.



(a) $\frac{d_1 V}{F_1}$ unmodified



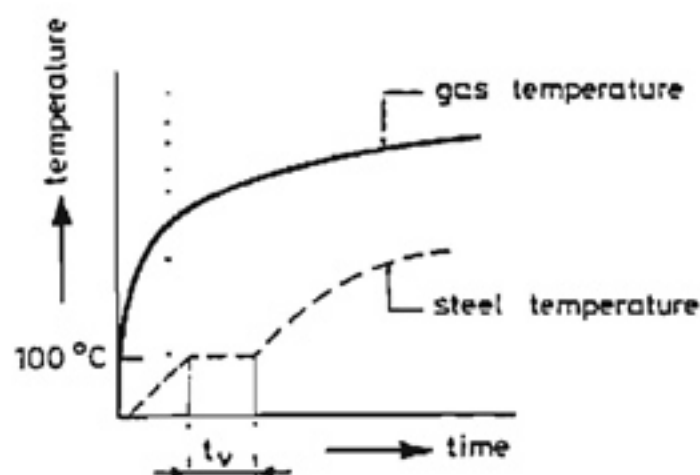
(b) $\frac{d_1 V}{F_1}$ modified for effect of weight of insulating material

Measured times for temperature of members with a heavy weight insulating material to reach 550°C .

Fig. III.17

III.5.5 Moisture content

The temperature rise of the steel can be delayed by evaporation of moisture. When a average value of λ_i is derived from test results this effect of moisture may be included. However, it is possible to allow for the moisture effect separately, by calculating a delay time t_v (min), when the steel reaches a temperature of 100°C, as illustrated in Fig. III.18.



Delay time t_v in the temperature increase of a steel member insulated by a moist insulation material.

Fig. III.18

The delay time depends on the moisture content p (% by weight), density, thermal conductivity and thickness of the insulating material and may be approximated by the following equation:

$$t_v = \frac{p \cdot \rho_i \cdot d_i^2}{5 \lambda_i} \quad (\text{III.15})$$

t_v	= delay time	min
p	= moisture content insulation material	% by weight
ρ_i	= density insulation material	kg/m ³
d_i	= thickness insulation material	m
λ_i	= thermal conductivity insulation material	W/m°C

Equation III.15 may be used for moisture contents corresponding to the hygroscopic equilibrium moisture at a temperature of 20°C and a relative humidity of 40%. Typical values for p are given in Table III.3.

Material	Density ρ (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 III.3: Approximate values for the hygroscopic equilibrium moisture content of various insulation materials.

Table III.4 gives some test results using a moist insulating material - vermiculite/gypsum/mineral fibre of density 540 kg/m^3 - showing the time t_{500} for the average steel temperature to reach 500°C and the delay time t_v .

Measured				Calculated		
$\frac{F_1}{V}$	d_1	t_v	t_{500}	$p = 20; \lambda_1 = 0.15$		$p = 0; \lambda_1 = 0.12$
m^{-1}	m	min	min	t_v min	t_{500} min	t_{500} min
116	.022	5	97	4	100	114
116	.032	12	146	13	147	159
216	.032	13	104	13	107	112
216	.047	43	186	30	171	168
359	.035	14	96	17	99	97
359	.062	55	234	54	215	191

Table III.4: measured values of time t_{500} for θ_s to reach 500°C and delay time t_v for a moist insulation material compared with calculated values.

By plotting values of $(t_{500} - t_v)$ against $(\frac{d_1 V}{F_1})_{\text{mod}}$, an effective value for λ_1 of $0.15 \text{ W/m}^\circ\text{C}$ can be deduced. Adopting a value for moisture content, p , of 20%, t_v can be calculated using equation III.15. These values and the resultant calculated values of t_{500} are shown in Table III.4 and are in good agreement with the measured ones. If the t_v effect is not considered separately, plotting values of t_{500} against $(\frac{d_1 V}{F_1})_{\text{mod}}$ gives an effective value for λ_1 of $0.12 \text{ W/m}^\circ\text{C}$. The calculated values of t_{500} which result are not in such good agreement as before, as is also illustrated in Table III.4.

IV MECHANICAL BEHAVIOUR OF FIRE EXPOSED STEEL MEMBERS

IV.1 Introduction

In order to predict the behaviour of a steel structure exposed to fire it is necessary to know the properties of steel at high temperatures. In this respect the following properties are of particular importance:

- strength and deformation,
- thermal expansion.

The variation of these properties with steel temperature is covered in Sections IV.2 and IV.3.

If the material properties are known with sufficient accuracy, it is possible in principle to calculate the mechanical behaviour of a steel structure at elevated temperature and the fundamentals of such a calculation are given in Section IV.4. However, the particular interest is to ascertain the temperature at which the structure will collapse. The factors that play a part in this are discussed in general terms in Section IV.5. The specific discussion of structural behaviour under fire conditions is confined to steel elements dealt with in the European Recommendations for the Fire Safety of Steel Structures [1], viz.:

- statically determinate and indeterminate beams (Section IV.6)
- centrically loaded columns (Section IV.7)

In Appendix C consideration is given to the specific effects which may occur if the above components form a part of a structural system. Finally, with the object of providing practical information, section IV.8 gives design rules for determining the failure temperatures of both beams and columns.

IV.2 Strength and deformation properties

The strength and deformation properties of structural steel are fairly rapidly impaired with rising temperature of the steel. A complicating feature is that steel also displays creep phenomena at temperatures above about 450°C. This means that under these circumstances, the deformation of a steel member under constant load will increase with time, even if the temperature and stress remain unchanged. As a result, the member will eventually fail. The process proceeds more rapidly as the stress and/or temperature in the member is increased. The occurrence of creep means that the deformation and collapse behaviour of a steel structure in a fire depends strictly on the heating-up program to which it is subjected.

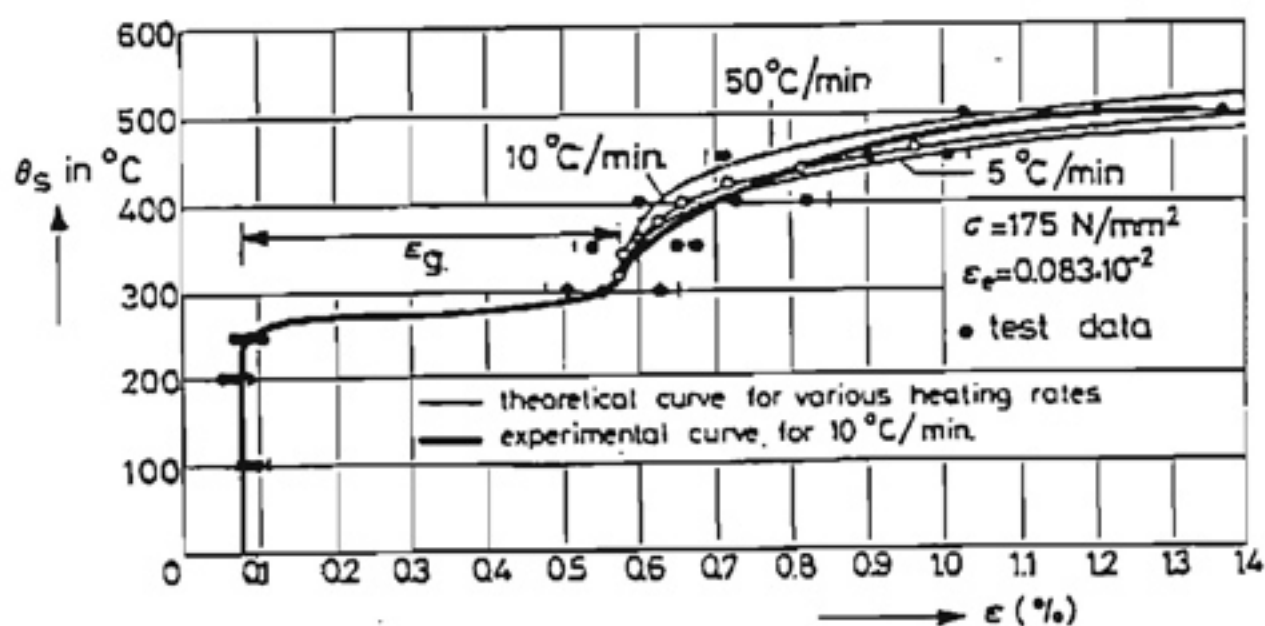
On the basis of information obtained from creep tests at high temperatures it is possible to analyse the creep behaviour of structures subjected to a time-temperature program, but this analysis is a rather complex procedure, and with regard to fire behaviour it has hitherto only proved possible to find solutions for simple cases such as beams [2], [3] and columns [4]. Extension of the method to structures of greater complexity, such as framed structures, runs into practical difficulties. Besides, it is difficult to incorporate this method into the existing time independent design methods to which the designer is accustomed. For this reason research has been undertaken into the effect of the heating rate upon the behaviour of a steel structure under fire conditions [5], [6]. In this context it is considered that a fire will in general not be of more than 2 hours' duration. Depending on the type and thickness of the insulation, the rate of heating - assuming it to be linear - will range from approximately 5 °C/minute (very well insulated) to 50° C/minute (not insulated).

The results of the research described in [5] and [6] lead to the conclusion that the deformation behaviour and thus the collapse temperature of beams and columns is not signifi-

cantly influenced by the heating-up program, subject to the proviso that the rate of heating varies between the above mentioned values and the final steel temperature does not exceed 600°C.

Based on the above conclusions and with due regard to the restrictions indicated, it is evident that there must be stress-strain relations for steel at elevated temperatures in which the creep behaviour is implicitly incorporated.

How these relations have been determined is described in [5], [6]. Essentially, the method was to carry out creep tests on standard test specimens (bars) under constant load and subjected to a linear rise in temperature. The advantage of this procedure is that the rate of loading is thus eliminated. Some results are represented in Fig. IV.1.

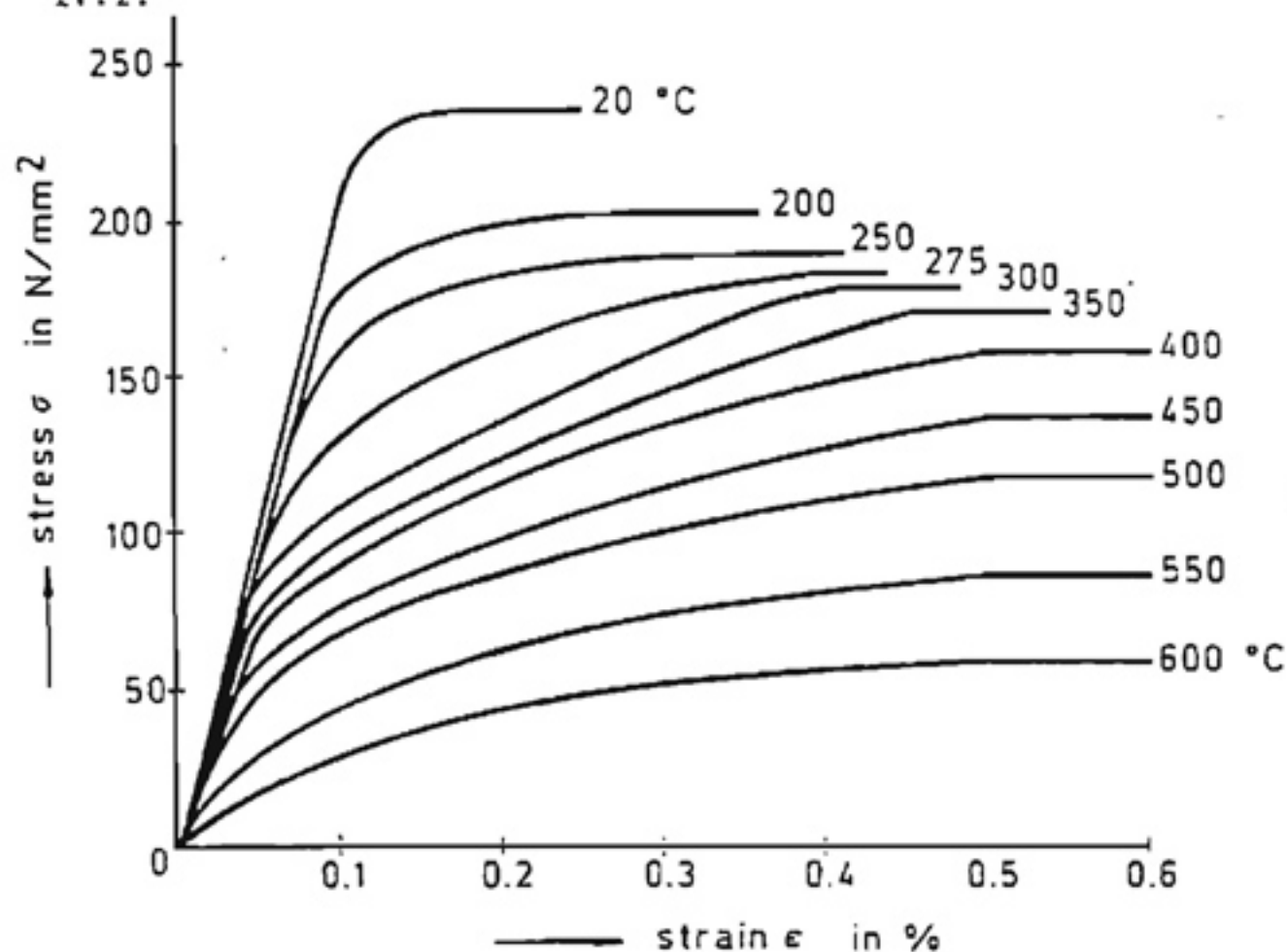


Temperature-strain relations at different rates of heating.

Fig. IV.1

At a temperature of about 300° C and at stresses above about 100 N/mm², thermally activated flow and strain hardening were observed. These phenomena are not manifested in creep tests in which the temperature and the load are kept constant.

Based on results illustrated in Fig. IV.1, stress-strain relations at elevated temperatures have been established, such as those for Fe 360 ($\sigma_{y20} = 235 \text{ N/mm}^2$) shown in Fig. IV.2.



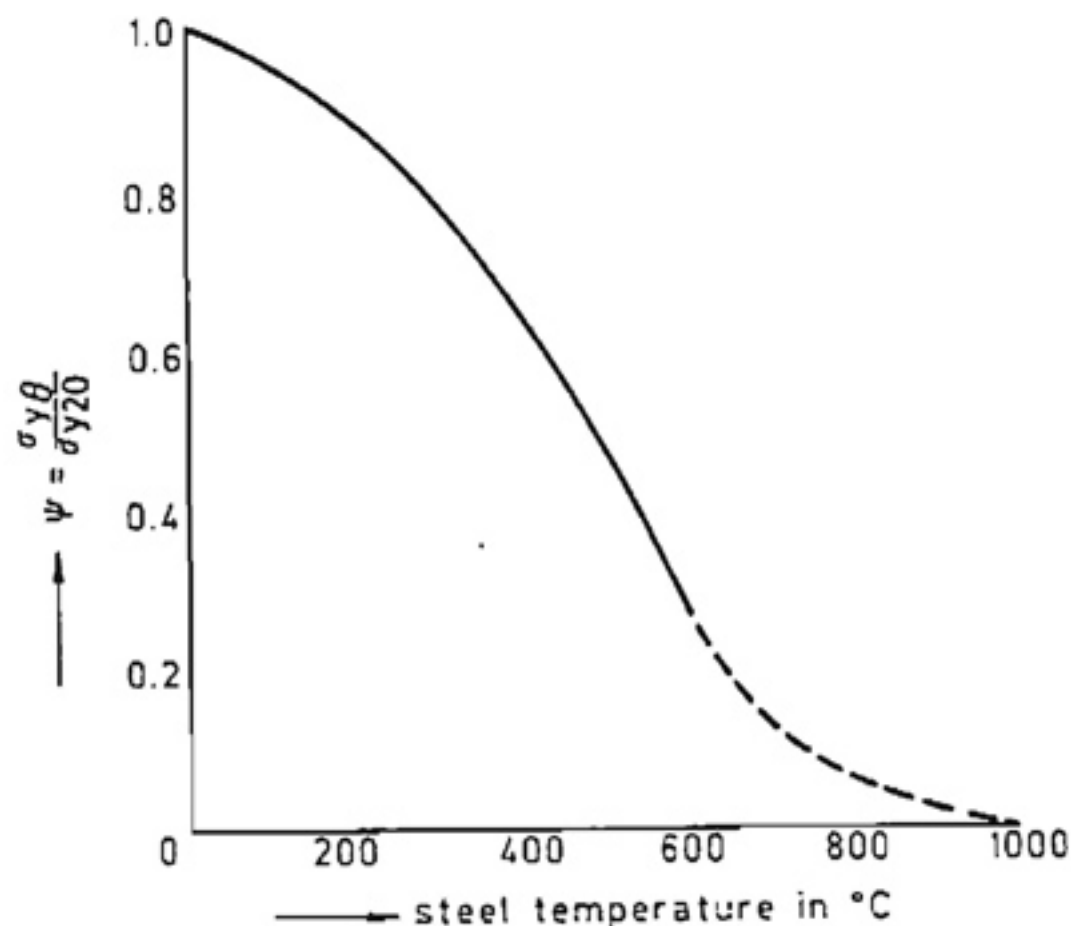
Stress-strain diagrams for Fe 360 at elevated temperature.

Fig. IV.2

The thermally activated flow is shown in this diagram by the discontinuity between the curves for 200°C and 300°C. The discontinuous transition of the curved to the straight horizontal portion in the stress-strain diagrams should also to be noted. In reality there is no such discontinuity:

instead, the stress-strain relation displays a continuous rising curve up to the occurrence of contraction (reduction of area). However in order to apply elementary plastic theory (see Section IV.4 et seq.), the stress-strain relations have been truncated. The corresponding level of stress is denoted by $\sigma_{y\theta}$ (= effective yield stress). Since this truncation is applied at large strain values, at which the gradient of the actual stress-strain curve is relatively small, the error due to the simplification is negligible.

In those cases where the collapse behaviour can be analysed with the aid of elementary plastic theory it is not necessary to know the whole stress-strain diagram at various temperatures. All that is needed is the relation between the effective yield stress and the steel temperature. This relation is shown in Fig. IV.3 and can be assumed to be independent of the grade of steel.



Effective yield stress of structural steel as a function of temperature.

Fig. IV.3

For temperatures below 600°C the diagram is based on effective yield stresses as described earlier. The range for temperatures above 600°C has been obtained from the literature [7], but since much less research has been done in this range, the available knowledge is limited. In addition, at these temperatures the effect of creep cannot be dealt with implicitly. On the other hand, it is known that steel retains some strength up to a temperature of about 1000°C. Hence, as an approximate estimate, the curve for the effective yield stress between 600° and 1000°C has been indicated as a broken curve in Fig. IV.3.

The curve showing the relation between effective yield stress ($\sigma_{y\theta}$) and steel temperature (θ_s) can be expressed by:

$$0 \leq \theta_s \leq 600^\circ\text{C}:$$

$$\sigma_{y\theta} = \left[1 + \frac{\theta_s}{767 \ln \frac{\theta_s}{1750}} \right] \cdot \sigma_{y20} \quad (\text{IV.1}^a)$$

$$600 < \theta_s \leq 1000^\circ\text{C}:$$

$$\sigma_{y\theta} = \left[\frac{108 \left(1 - \frac{\theta_s}{1000} \right)}{\theta_s - 440} \right] \cdot \sigma_{y20} \quad (\text{IV.1}^b)$$

As already stated, these relations are independent of the grade of steel.

Stress-strain relations for steels other than Fe 360 have also been derived [8], [9]. For a survey relating to the steel grades as envisaged in European Standard No. 25, see Appendix B. Since creep plays an important part at temperatures above 600°C, as already explained, no stress-strain relations for the range between 600° and 1000°C are given in that Appendix.

For a tabulated review of yield stress as function of steel temperature, it is referred to Table D-1^{a,b}.

After cooling, structural steel (hot rolled) will regain its original properties. Structural members which have become unacceptably distorted will, on cooling, thus have regain their original strength. This can be of importance in connection with demolition work. Rolled steel members which have undergone little or no deformation can be re-used.

IV.3 Thermal expansion of steel

When the upper and lower flanges of a heated beam are at different temperatures, there will be additional deflections if the beam is unrestrained, i.e., if it can expand freely or there will be additional stresses if it is restrained from expanding. According to the theory of elasticity these stresses can theoretically attain very high values. However, the stresses that actually occur will usually be lower. Measurements performed during experimental fires have shown that the temperatures occurring within one and the same enclosed space are liable to vary a great deal and that the maximum values are attained only in a few places in that space. Also, elastic or plastic movement is possible to some extent, so that the thermal stresses are levelled down. If thermal expansion can take place in consequence of "give" in the connecting elements or the presence of expansion joints, the structure can undergo a rise in temperature without coming to harm. If elongation is prevented by adjacent structural components, stresses will develop which cause cracks in materials such as concrete or masonry. Thus, the expansion of floors may cause the collapse of loadbearing walls.

However, as will be shown in Section IV.6 et seq., the effect of thermal expansion upon the loadbearing capacity of the steel structure itself may, in practical cases, often be neglected.

The coefficient of thermal expansion depends on the steel temperature [7], but for many practical applications it is sufficient, as an approximation, to consider the coefficient to be independent of temperature. The following equation is then obtained:

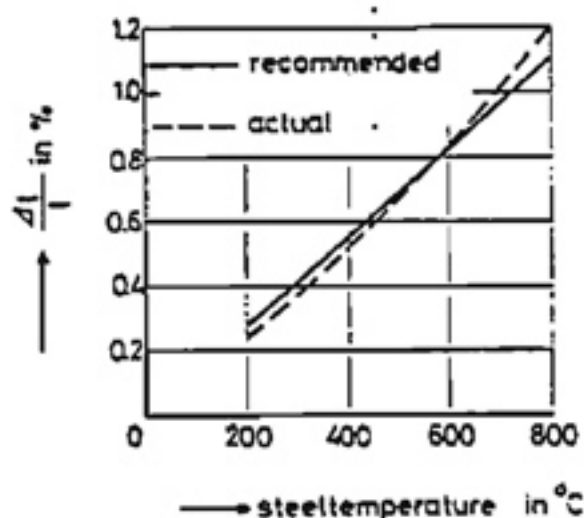
$$\frac{\Delta l}{l} = 1.4 \times 10^{-5} \theta_s \quad (\text{IV.2})$$

Where

Δl = thermal elongation

l = original length over which Δl is measured

θ_s = steel temperature



Thermal expansion of steel as a function of the steel temperature.

Fig. IV.4

IV.4 Principles of analysis

By knowing the properties of steel at elevated temperature, as discussed in the preceding Sections, it is possible to calculate the mechanical behaviour of a steel structure in a fire. In principle, such calculations are similar to those for structural analysis at room temperature, provided plastic analysis is used. Since the stress-strain relations at elevated temperature are curved from the beginning the linear elastic theory cannot be used for fire design. An elasto-plastic approximation must be adopted.

In most cases only the failure under fire conditions will be of interest. If this state is not affected by the deformations that occur, the elementary plastic theory can be used. On the assumption of a known temperature distribution in the steel, the associated collapse load can be calculated with the aid of the temperature-dependent effective yield stress as indicated in Fig. IV.3. A situation of this kind occurs, for example, in beams.

However, in those cases where stability plays a part, collapse will be determined partly by the deformations that occur and the elementary plastic theory cannot provide a solution. Instead, for a given temperature distribution, the load must be increased incrementally. After each loading stage the deformation and the associated pattern of forces must be determined. The collapse load at elevated temperatures is now dependent not only on the effective yield point, but on the whole course of the stress-strain diagram. See Fig. IV.2. This method of analysis is necessary for determining the collapse behaviour of columns, for example. When buckling curves are calculated using this incremental method and the stress-strain relationships in Section IV.2, the solutions are conservative. For design purposes, therefore, alternative buckling curves have been prepared, which, though derived in a more approximate way, show good agreement with the results of full scale fire tests on columns. See Section IV.7.

During a fire the load on the structure will remain practically constant, while the temperature rises. In fire calculations it is therefore usual to analyse the structural behaviour under constant load and rising temperature. Such a calculation procedure is entirely similar to that in which the load is increased and the temperature is kept constant. In the incremental method, the temperature instead of the load is increased in small steps.

The load acting on the structure in a fire will obviously have a great influence upon the deformation and collapse behaviour. The "degree of loading" can be expressed as the quotient ($=\gamma$) of the design load in a fire ($=P$) and the maximum load that the structure can support at room temperature ($=P_p$). In formula:

$$\gamma = P/P_p \quad (IV.3)$$

Since the analysis of a structure under fire conditions has to be based on the plastic theory, in this manual P_p will always be taken to denote the collapse load at room temperature likewise calculated on the basis of the plastic theory. If the design of the structure at room temperature has also been based on the plastic theory, the load P_p will be known and therefore constitute a given value in the fire safety assessment. Alternatively, however, the design at room temperature may be based on the elastic theory, in which case the value of P_p will have to be separately calculated. This situation is more likely to occur with beams (see Section IV.8). The design of columns at room-temperature is always based on the limit state of collapse, so that for these members the complication does not arise. The design load P for fire conditions can be estimated using the rules in Section IV.8.

For a more general discussion on the principles of analysis, it is referred to, e.g. [10]-[15].

IV.5 The concept "critical temperature"

In the analysis of the behaviour of steel structures under fire conditions it is usual to assume a uniform temperature distribution in the steel, both over the depth of the structural section and along the length of the member. See also chapter III. For this case the concept of a "critical temperature" can be adopted, this being defined as the steel temperature at which the structure is on the point of collapsing.

For a steel structure the critical temperature is in practice found to be between 400° and 600°C. That a steel structure does not collapse until reaching such fairly high temperatures is because a certain margin of safety against overloading is embodied in its design. When this margin is narrower, the critical temperature will be lower. Thus the critical temperature of a structure which has been loaded almost to collapse at room temperature will be little higher than 20°C. In the case where the structural member has a low degree of loading, however, the critical temperature is normally not reached in real fires.

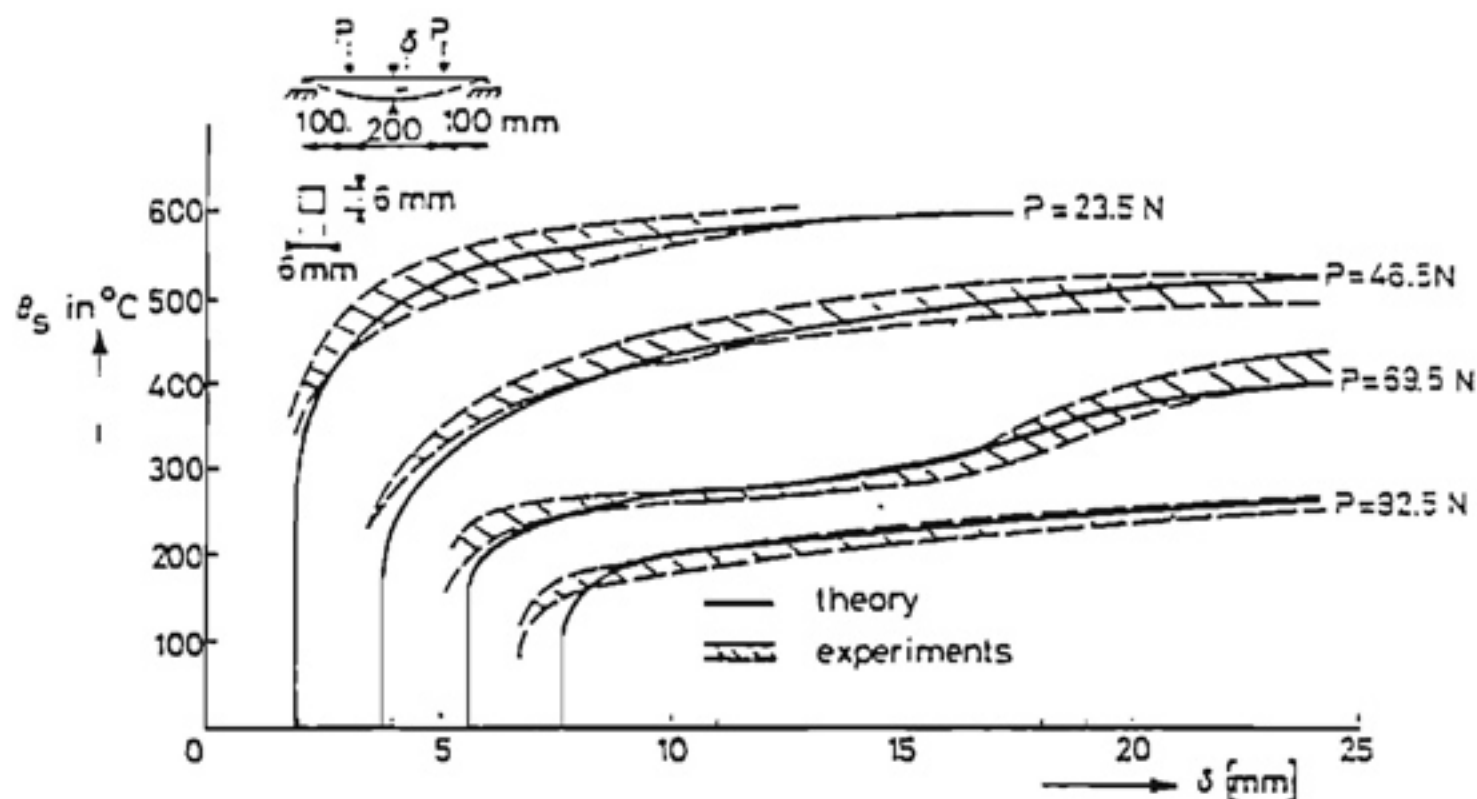
In practice the critical temperature will be between these extremes and tend to be within the range of 400° and 600°C as already stated. The effect in question is taken into account in the analysis by the degree of loading $\gamma = P/P_p$, defined in Section IV.4.

Besides being dependent on γ , the critical temperature will depend on the type of structural member under consideration: beam or column. The structural system - statically determinate or indeterminate - can also be of importance.

IV.6 Behaviour of beams under fire conditions

An obvious way to characterize the behaviour of steel beams in fire is to give the relation between the temperature and the maximum deflection at a certain load. For statically determinate beams, the thermal expansion will have no effect on this relation, if the temperature is assumed to be uniformly distributed through the depth and along the length of the beam, and deflection is caused solely by the diminishing stiffness. In other words: if the stress-strain relations at elevated temperature are known, the relation between steel temperature and deflection - for a given load - can be calculated. In view of the curved shape of the stress-strain diagrams (see Section IV.2) the calculation is rather laborious and for practical purposes a computer must therefore be used.

The mathematical model has been verified experimentally. For this purpose model tests were performed on simply supported small steel beams (400 mm span, 6 x 6 mm² cross-section) under concentrated loads P at 1/4 and 3/4 of the span. An advantage of such small-scale tests is that an approximately uniform temperature distribution in the steel can be readily achieved, which facilitates comparison with the theory. The relationship between steel temperature and mid span deflection was determined for various values of the load P . In Fig. IV.5 some results are given [6]. The dashed curves refer to test results; the full-line curves are calculated from the stress strain relationship given in Fig IV.2. The temperature-deformation diagram for $P = 69.5$ N is found to produce large deflections for relatively small temperature rises for temperatures between 250 and 300 °C. For $P = 46.5$ and 23.5 N this flattening-off of the curves does not occur. This behaviour is linked with the phenomenon of thermally activated flow, discussed earlier. In the case of the low load ($P = 46.5$ and 23.5 N) the stresses are not high enough to cause thermally activated flow to manifest itself. See also Section IV.2 and Fig. IV.1. The agreement between theory and experiment is satisfactory.

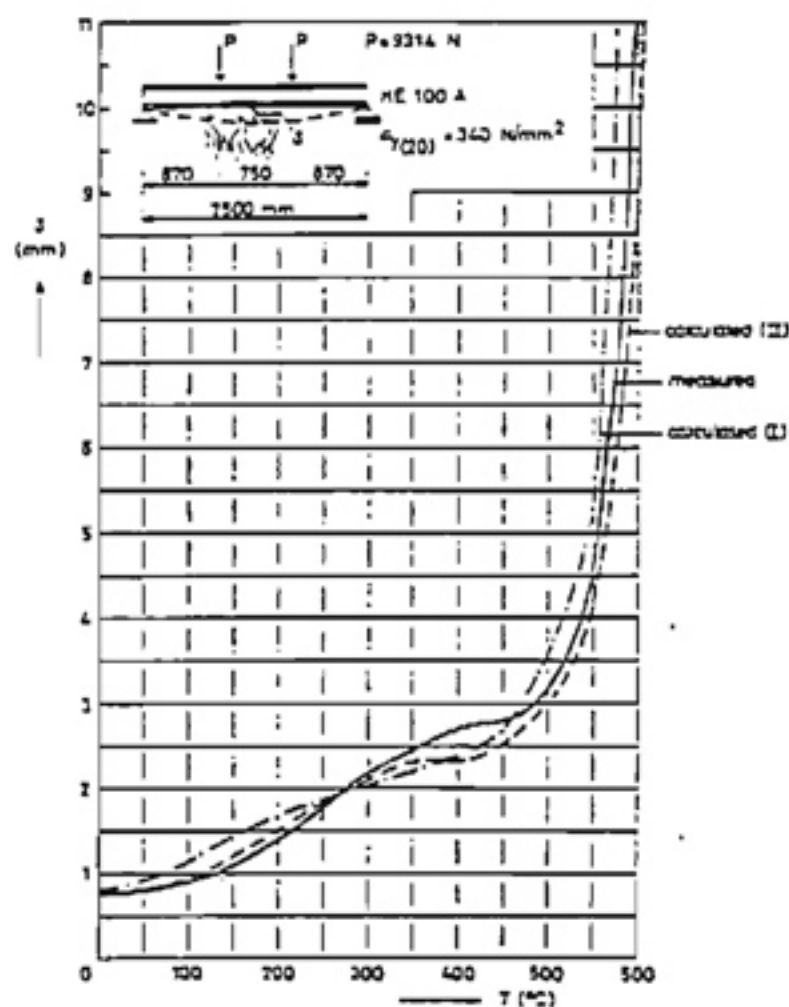


Experimental verification of the mathematical model for determining the behaviour of steel beams under a uniform rise of temperature [6].

Fig. IV.5

In practice, steel beams generally have I-shaped instead of rectangular cross-sections. The top flange will often carry a floor slab. Since the heat can flow relatively easily out of the steel into the slab, a difference in temperature between top and bottom flange must be expected to occur. Furthermore, a temperature variation in the longitudinal direction of the beam must be expected.

Provided that the steel temperature distribution is known, the deformation of the beam can be calculated as a function of some characteristic temperature of the beam. This is illustrated in Fig. IV.6 in which the results of a fire test on a full scale beam are compared with theoretical predictions.



Comparison between the theoretically and experimentally determined deformation behaviour of a loaded beam with a non-uniform temperature distribution along and across the member.

Fig. IV.6

The beam (profile HE 100 A, span 2500 mm) was exposed to natural fire conditions [2]. During the test a significant temperature variation was measured both over the height and the cross section of the beam. The full-line curve gives the measured relationship between the mid span deflection and the average steel temperature in the cross section at mid span.

Using the measured steel temperatures, the mid span deflection was calculated, taking into account the mechanical properties and expansion of steel as specified in Section IV.2 and IV.3 respectively [16]. These calculated

results are shown by the upper dashed curve (curve I). The other dashed curve (curve II) refers to results of a calculation model in which creep is taken into account explicitly [2].

The differences between the experimental and the two calculated curves are only marginal. The analysis illustrates that the approximate way in which the creep phenomenon is dealt with in the stress strain relationship of Fig. IV.2 leads to fully acceptable results, which are on the conservative side. It is moreover seen that also under less idealized circumstances, a good agreement between experiment and theory can be achieved. It is noted however that, in a practical design, the temperature distribution in a fire exposed steel member will not be known in detail. The following discussion will therefore be based on the assumption of a uniform steel temperature. In Section IV.8 the consequences of this simplification regarding the calculation rules for the critical temperature will be evaluated.

In the foregoing treatment of the subject, the behaviour of steel beams in fire is characterized by the relation between temperature and deflection. In practice, however, interest will be confined almost entirely to the collapse behaviour. Collapse occurs when the deflection increases excessively without appreciable further rise in temperature. In the case of uniform temperature distribution the instant of collapse is determined by the so-called critical temperature. See also Section IV.5. The critical temperature can be read from the diagrams already presented. Alternatively, it can be determined much more simply by making use of the plastic theory. This will be illustrated in the next example.

Worked example IV.1

Consider a simply supported steel beam (IPE 240 rolled steel section) carrying a uniformly distributed load.

The relevant data are:

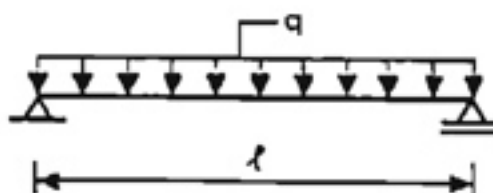
span	: l	= 4.80 m
load	: q	= 19.22 kN/m
plastic section modulus	: Z	= $362 \times 10^3 \text{ mm}^3$
yield stress at room temp.	: σ_{y20}	= 240 N/mm^2
effective yield stress at temperature of θ_s °C.	: $\sigma_{y\theta}$	

It is assumed that the load remains unchanged during the fire and that the temperature in the beam is uniformly distributed, both through the depth of the section and along the length of the beam.

Determine the critical temperature.

Solution: Under the influence of the rise in the temperature of the steel, the effective yield stress will decrease. The beam collapses when yield occurs at the mid-span section. The term "plastic hinge" is used in this context. For the collapse mechanism see Fig. IV.7, where the stress distribution at the plastic hinge is also shown.

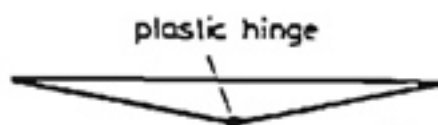
structural system



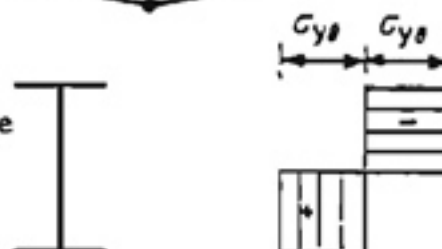
bending moment distribution



collapse mechanism



stress distribution at plastic hinge



Collapse behaviour of a simply supported beam.

Fig. IV.7

At a critical temperature of $\theta^{\circ}\text{C}$, the notation $\sigma_{y\theta}$ indicates the effective yield stress. The bending moment that the plastic hinge can transmit - the so-called full plastic moment $M_{p\theta}$ - is expressed by the formula:

$$M_{p\theta} = Z \cdot \sigma_{y\theta}$$

which is entirely similar to the calculation of this moment at room temperature. In view of the behaviour of $\sigma_{y\theta}$ as a function of temperature (Fig. IV.3), the full plastic moment decreases with rising steel temperature. If $M_{p\theta}$ thus decreases to a value corresponding to the applied moment ($\frac{1}{8} ql^2$), the beam will be about to collapse.

Then:

$$Z \sigma_{y\theta} = \frac{1}{8} ql^2$$

or:

$$\sigma_{y\theta} = \frac{ql^2}{8Z}$$

On dividing by σ_{y20} we obtain:

$$\frac{\sigma_{y\theta}}{\sigma_{y20}} = \frac{ql^2}{8Z \sigma_{y20}}$$

In a given situation the quotient $\frac{\sigma_{y\theta}}{\sigma_{y20}}$ can be calculated. In the present case:

$$\frac{\sigma_{y\theta}}{\sigma_{y20}} = \frac{19.22 \times (4.8)^2 \times 10^6}{8 \times 362 \times 10^3 \times 240} = 0.64$$

The corresponding critical temperature can now quite simply be read from Fig. IV.3. We thus find: 400°C .

The result of the calculation in example IV.1 can be presented in a more generalized form, bearing in mind that the collapse load at room temperature is: $q_p = 8 \frac{\sigma_{y20} \cdot Z}{l^2}$. On substituting this into the expression which has been obtained for $\frac{\sigma_{y\theta}}{\sigma_{y20}}$ we thus find:

$$\frac{\sigma_{y\theta}}{\sigma_{y20}} = \frac{q}{q_p} (= \gamma) \quad (\text{IV.4})$$

If the design calculation at room temperature is based on the plastic theory, q_p is a known quantity. If the design load under fire conditions ($= q$) is known, the quotient $\frac{\sigma_{y\theta}}{\sigma_{y20}}$ and therefore also the critical temperature can be determined.

The preceding discussion deals with statically determinate beams. The calculation rule derived for the critical temperature can, however, be applied to statically indeterminate beams as well. At first sight this may be surprising since the pattern of forces in the beam will - as contrasted with what happens in a statically determinate structure - undergo a change during fire exposure. The way in which the collapse condition is formulated is, however, identical.

This because so many plastic hinges have developed at the collapse stage that the pattern of forces can be determined solely from considerations of equilibrium. For this reason, too, the thermal expansion will, in the event of non-uniform distribution of temperature, not affect the collapse load of a statically indeterminate beam, as will be illustrated by the following example.

Worked example IV.2

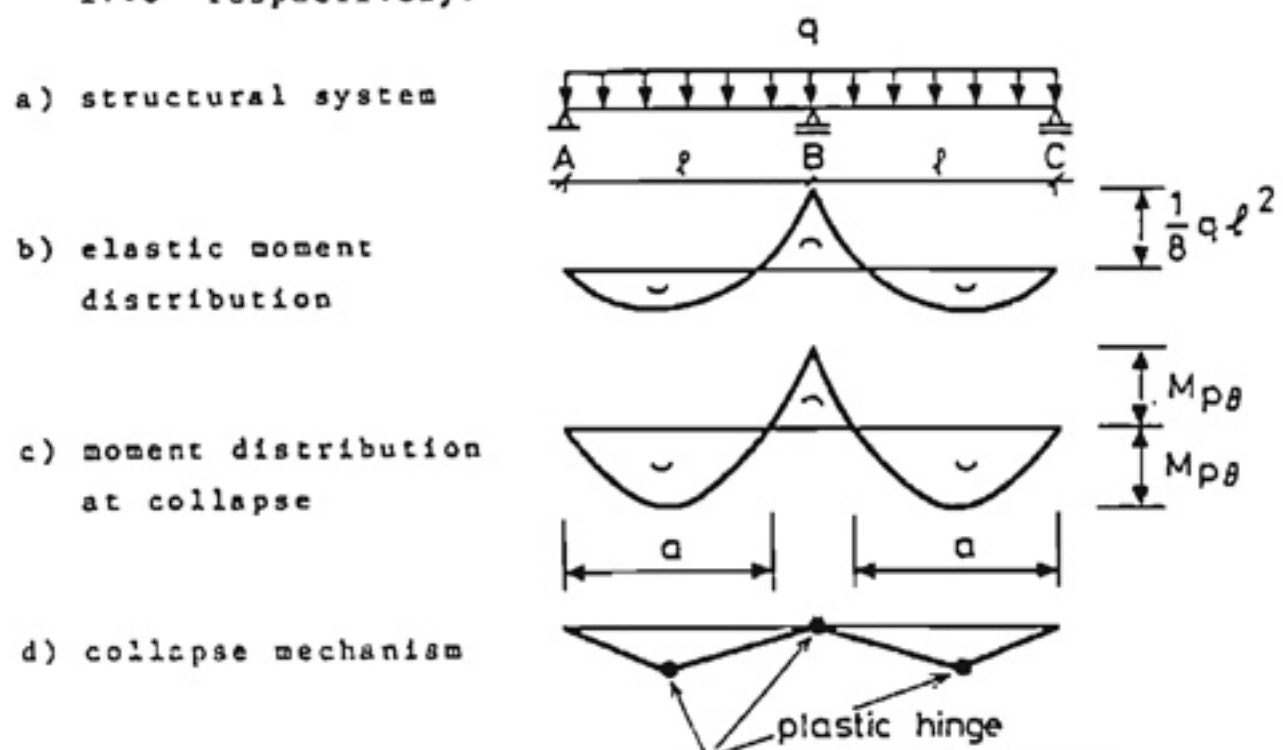
Consider a steel beam (IPE 450 rolled section) with statically indeterminate support conditions and subjected to uniformly distributed load as shown in Fig. IV.8. The relevant data for this beam are:

span	: $l = 12 \text{ m}$
load	: $q = 21 \text{ kN/m}$
plastic section modulus	: $Z = 1702 \times 10^3 \text{ mm}^3$
yield stress at room temperature	: σ_{y20}
effective yield stress at steel temperature of θ °C	: $\sigma_{y\theta}$

As in example IV.1, it is here again assumed that the steel temperature is uniformly distributed both through the depth and along the length of the beam.

Determine the critical temperature.

Solution: for the collapse behaviour of the beam see Fig. IV.8. With rising steel temperature a plastic hinge will first develop over the centre support, because here the bending moment at room temperature is highest (see Fig. IV.8^b). However, the temperature can continue to rise until plastic hinges are formed also in the spans of the beam. The associated distribution of the bending moment and the collapse mechanism are shown in Fig. IV.8^c and IV.8^d respectively.



Collapse behaviour of a continuous beam on three supports.

Fig. IV.8

When the plastic hinge has developed over the centre support, the portions AB and BC of the beam behave as though statically determinate. The load corresponding to the bending moment distribution as shown in Fig. IV.8^c is therefore simple to determine. If this load is just equal to the load actually present ($= q$), the beam fails. The collapse condition is therefore:

$$M_{p\theta} = \frac{1}{2} q l (l - a) = \frac{1}{8} q a^2$$

Therefore:

$$a = 2l (\sqrt{2} - 1) = 0.828 l$$

With:

$$M_{p\theta} = \sigma_{y\theta} \cdot Z$$

$$\text{it follows that } \sigma_{y\theta} = \frac{1/8 q (0.828l)^2}{Z} = \frac{q l^2}{11.7 Z}$$

Finally, on dividing by σ_{y20} , we obtain:

$$\frac{\sigma_{y\theta}}{\sigma_{y20}} = \frac{q l^2}{11.7 Z \sigma_{y20}}$$

The quotient $\frac{\sigma_{y\theta}}{\sigma_{y20}}$ can again be calculated in the given situation.

$$\text{We thus obtain: } \frac{\sigma_{y\theta}}{\sigma_{y20}} = \frac{21.0 \times (12)^2 \times 10^6}{11.7 \times 1702 \times 10^3 \times 240} = 0.63$$

From Fig. IV.3 the value for the critical temperature is found: 410°C.

The analysis can now again be generalized, bearing in mind that for the collapse load at room temperature we have:

$$q_p = \frac{11.7 Z \sigma_{y20}}{l^2}$$

On combining this with the expression found for $\frac{\sigma_{y\theta}}{\sigma_{y20}}$ we obtain:

$$\frac{\sigma_{y\theta}}{\sigma_{y20}} = \frac{q}{q_p} (= \gamma) \quad (\text{IV.4})$$

This equation is identical to the one derived earlier for the statical determinate beam. It is noted that considerably higher critical temperatures are possible than those given by this equation, if transmission of forces through membrane action can be taken into account. For a discussion on this subject see Appendix C.2.

IV.7 : Behaviour of columns under fire conditions

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 centrically loaded. The applied load is kept constant during the test. Hence, a similar model can provide the basis for the calculation of the critical temperature. This is shown in Fig. IV.9. It is assumed that the load P is constant and that the temperature is uniformly distributed.



assumptions: - centrically loaded
- uniform temperature distribution.

Model for the calculation of the critical temperature of columns.

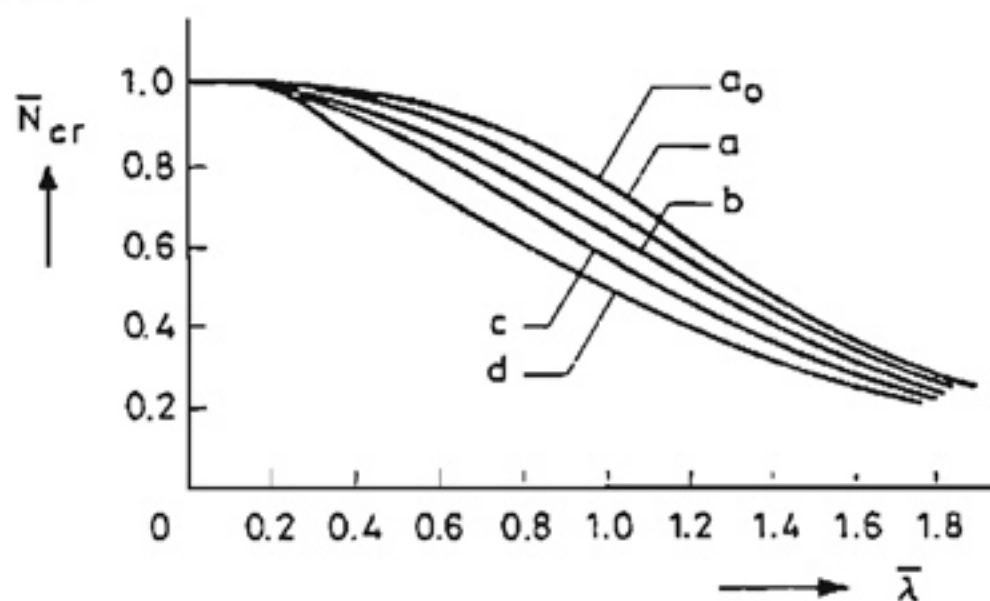
Fig. IV.9

The analysis of the collapse behaviour of centrically loaded struts is complicated, even in the relatively simple case envisaged in Fig. IV.9. This is true not only for the structure under fire conditions, but also at room temperature. The reason is that there is a number of basically stochastic factors involved which, in combination, influence the collapse behaviour.

The following may be mentioned:

- residual stresses;
- unintended eccentricities of the load;
- unintended initial lateral deflections of the column

Because of these complicating factors, the design rules for steel columns at room temperature are based on statistically interpreted experimental research. For this purpose a large number of tests have been performed on columns with various cross-sectional shapes and slenderness ratios, more particularly in experimental work carried out internationally within the European Convention for Constructional Steelwork (ECCS). For each slenderness ratio, the mean buckling stress and the standard deviation were calculated. The results of the tests, covering a wide range of application, were interpreted statistically and based on this analysis buckling curves were established (so called ECCS-buckling curves). These curves are given in Fig. IV-10 in non-dimensional form.



ECCS-buckling curves at room temperature.

Fig. IV.10

In Fig. IV.10 the dimensionless buckling load $\bar{N}_{cr}$ 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 $\bar{\lambda}$ which is related to the slenderness ratio λ as follows:

$$\bar{\lambda} = \frac{\lambda}{\pi \sqrt{\frac{E}{\sigma_{y20}}}} \quad (\text{IV.5})$$

with

E = Youngs modulus at room temperature

σ_{y20} = yield stress at room temperature

The curves can be described by the following general expression:

$$\bar{N}_{cr} = \frac{\sigma_{cr20}}{\sigma_{y20}} = \frac{1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2}{2\bar{\lambda}^2} - \frac{1}{2\bar{\lambda}^2} \sqrt{[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]^2 - 4\bar{\lambda}^2} \quad (\text{VI.6}^a)$$

for $\bar{\lambda} > 0.2$

and

$$\bar{N}_{cr} = 0.2$$

for $\bar{\lambda} < 0.2$ (IV.6^b)

The coefficient α (dimensionless) varies with the selected curve and values are given in Table IV.1.

ECCS buckling curve	α_0	a	b	c	d
a	0.125	0.206	0.339	0.489	0.756

Table IV.1: Buckling coefficient α for various types of cross section.

The choice of curve a to d depends on the type of cross section and the manufacturing process. Hot rolled and cold formed hollow steel sections, for example apply to the "a" curve. Due to the unfavourable effect of residual stresses, for hot rolled I sections the a, b or even c curve holds, depending on buckling axis and the cross sectional dimensions.

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 σ_{y20} , the Youngs modulus E and the slenderness ratio λ .

It is possible to tackle the buckling problem at room temperature by an analytical approach, e.g., with the aid of the ECCS curves. In that case an initial deflection of such magnitude is chosen that the calculated failure stress corresponds to that which conforms to the relevant ECCS-curve. All the uncertain factors - such as residual stresses, incidental eccentricities etc. - are, as it were, taken into account in the so called fictitious initial deflection.

With the aid of the strain-stress relationships discussed in Section IV.2 (see Fig. IV.2) it is possible, in principle, to carry out such analyses also for elevated temperatures [15]. The results are, however, conservative compared to the outcomes of full scale fire tests on columns. The reason is that there is insufficient understanding of the various factors which influence the stability of columns at elevated temperatures, such as the unloading characteristics of steel and the effect of residual stresses [18], [19].

Therefore, when deriving buckling curves at elevated temperatures, a more pragmatic approach has been chosen [20]. In this approach it has been assumed that the buckling curves at elevated temperatures can be described by an equation similar to the one above for room temperature, viz.:

$$\frac{\sigma_{cr\theta}}{\sigma_{y\theta}} = \frac{1+\alpha (\bar{\lambda}_{\theta}-0.2)+\bar{\lambda}_{\theta}^2}{2\bar{\lambda}_{\theta}^2} - \frac{1}{2\bar{\lambda}_{\theta}^2} \sqrt{[1+\alpha (\bar{\lambda}_{\theta}-0.2)+\bar{\lambda}_{\theta}^2]^2 - 4\bar{\lambda}_{\theta}^2} \quad (\text{IV.7})$$

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 $\bar{\lambda}_{\theta}$ can be quantified by substituting temperature dependent values for the yield stress ($\sigma_{y\theta}$) and the modulus of elasticity ($E_{t\theta}$) in the equation for the modified slenderness ratio $\bar{\lambda}$, viz.:

$$\begin{aligned} \bar{\lambda}_{\theta} &= \frac{\lambda}{\pi \sqrt{\frac{E_{t\theta}}{\sigma_{y\theta}}}} \\ \text{or} \quad \bar{\lambda}_{\theta} &= \bar{\lambda} \sqrt{\frac{\frac{\sigma_{y\theta}}{\sigma_{y20}}}{\frac{E_{t\theta}}{E}}} \end{aligned} \quad (\text{IV.8})$$

Since both $\frac{\sigma_{y\theta}}{\sigma_{y20}}$ and $\frac{E_{t\theta}}{E}$ vary in a similar way with the steel temperature (cf. Fig. IV.3 and Appendix B respectively), it follows, by approximation:

$$\bar{\lambda}_{\theta} = \bar{\lambda}$$

Consequently:

$$\frac{\sigma_{cr\theta}}{\sigma_{y\theta}} = \frac{\sigma_{cr20}}{\sigma_{y20}} = \bar{N}_{cr} \quad (\text{IV.9})$$

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

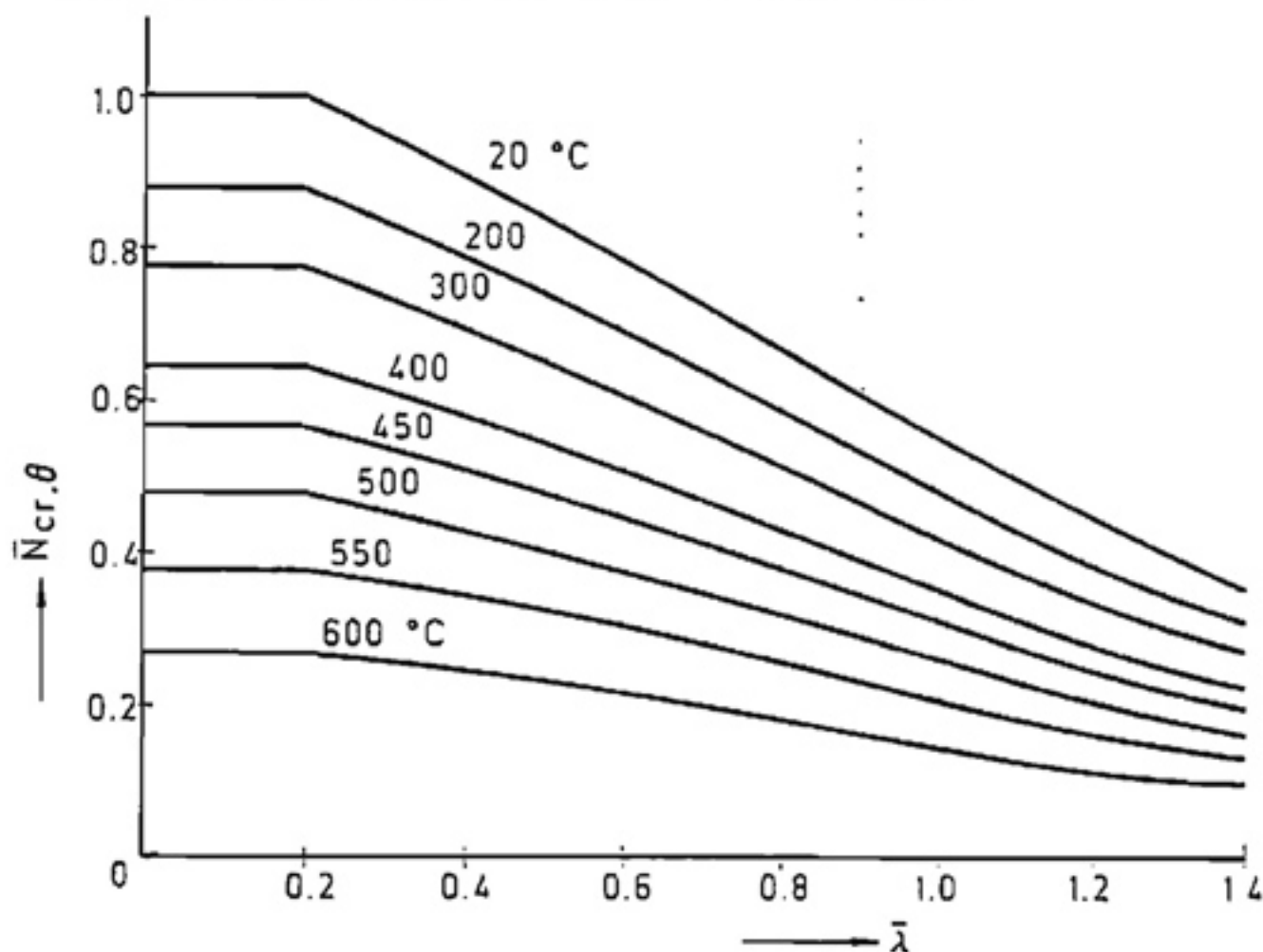
$$\bar{N}_{cr\theta} = \frac{\sigma_{y\theta}}{\sigma_{y20}} \bar{N}_{cr} \quad (\text{IV.10a})$$

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

$$\bar{N}_{cr\theta} = \frac{\sigma_{y\theta}}{\sigma_{y20}} \left\{ \frac{1+\alpha (\bar{\lambda}-0.2)+\bar{\lambda}^2}{2\bar{\lambda}^2} - \frac{1}{2\bar{\lambda}^2} \sqrt{[1+\alpha (\bar{\lambda}-0.2)+\bar{\lambda}^2]^2 - 4\bar{\lambda}^2} \right\} \quad (\text{IV.10b})$$

It is emphasized that the buckling curves derived in this way are for central loading and bifurcation buckling only. They do not take into account other types of instability and axial restraint. Therefore, the room temperature 'c' curve of Fig. IV.10 has been used as the basis for the elevated temperature buckling curves. This corresponds to a value of 0.489 for α .

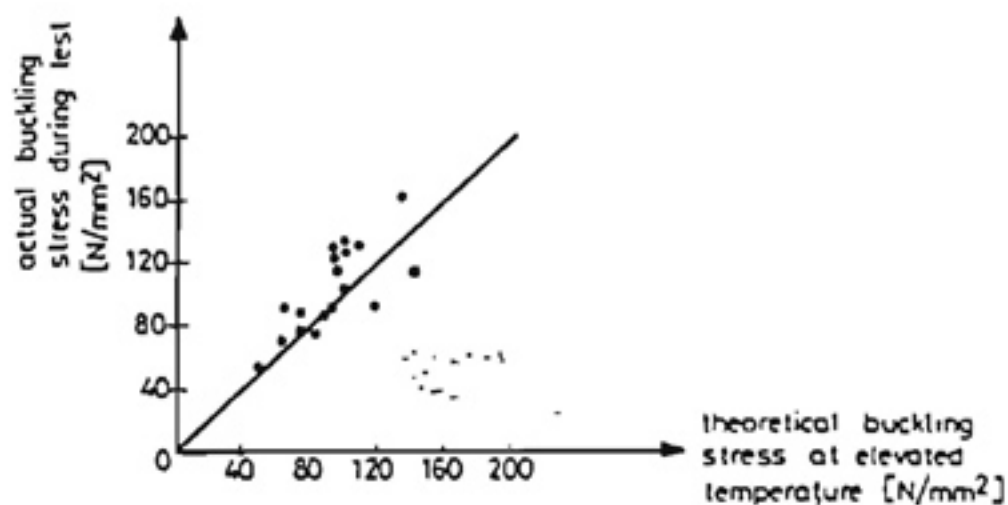
These buckling curves are shown in Fig. IV.11.



Dimensionless buckling curves at elevated temperatures.

Fig. IV.11

The recommended elevated temperature buckling curves are in good agreement with the results of recent full scale tests carried out in Belgium and Denmark [20], [21]. This is, for the Belgian test results, illustrated in Fig. IV.12 in which the theoretical buckling stress at elevated temperature is compared with the actual stress during the test.



Comparison of the results of column tests at elevated temperatures and theoretical predictions [18].

Fig. IV.12

Using these buckling curves, the critical temperature of centrically loaded columns can easily be determined, as illustrated by the following example:

Worked example IV.3.

Consider a centrically loaded column with the following specifications:

grade of steel	: Fe 360
profile	: HE 200 A
slenderness ratio	: 100
load	: 340 kN

It is assumed that the load remains unchanged during the fire and that the temperature in the column is uniformly distributed.

Determine the critical temperature.

Solution: Note that $\sigma_{y20} = 235 \text{ N/mm}^2$ (grade of steel: Fe 360) and $A = 5.38 \times 10^3 \text{ mm}^2$ (HE 200A). The dimensionless buckling load at elevated temperature, now follows directly from:

$$\bar{N}_{cr\theta} = \frac{340 \times 10^3}{5.38 \times 10^3 \times 235} = 0.269$$

The modified slenderness ratio $\bar{\lambda}$ is calculated from: .

$$\bar{\lambda} = \sqrt{\frac{100}{\frac{2.1 \times 10^5}{235}}} = 1.07$$

For the combination of $\bar{N}_{cr,\theta} = 0.269$ and $\bar{\lambda} = 1.07$, in Fig. IV.11 a critical temperature of 460°C is read.

The limit state condition for centrically loaded columns can also be formulated in a way which is similar to the one obtained for beams. The starting point is the expression derived above for the buckling curves at elevated temperatures.

$\bar{N}_{cr}$ and $\bar{N}_{cr\theta}$ can respectively be denoted as:

$$\bar{N}_{cr} = \frac{N_{cr}}{A \sigma_{y20}}$$

$$\bar{N}_{cr\theta} = \frac{N_{cr\theta}}{A \sigma_{y20}}$$

It follows with (IV.10^a) for the limit state of failure:

$$\frac{\sigma_{y\theta}}{\sigma_{y20}} = \frac{N_{cr\theta}}{N_{cr}} \quad (= \gamma) \quad (\text{IV.11})$$

Identifying the quotient $\frac{N_{cr\theta}}{N_{cr}}$ as the "degree of loading" γ , it is noted that the equation derived this way for the limit state of failure for columns is identical to that derived for beams. (cf equation VI.4) If γ is known, the critical temperature of columns is defined with the aid of the quotient $\sigma_{y\theta}/\sigma_{y20}$, in combination with the material properties as envisaged in Fig. IV.3 or by the equations for the effective yield stress as a function of the steel temperature given in section IV.2. (cf equation IV.1)

Note that the equation derived for the limit state of failure is only valid for centric buckling under constant load. For a discussion on the effect of axial restraint and eccentric loading, see Appendix C.2 and C.3 respectively.

IV.8 Practical applications

IV.8.1 Calculation of the critical temperature of beams and columns

The discussion in Section IV.6 and IV.7 proves it possible to give a generally-valid expression for the "critical" temperature of beams and columns. For this purpose the margin between the design load under fire conditions ($= P$) and the collapse load at room temperature ($= P_p$) is represented by the "degree of loading": γ . Therefore: $\gamma = P/P_p$. The critical temperature of beams and columns is defined with the aid of the quotient $\sigma_{y\theta}/\sigma_{y20}$ in combination with the material properties as envisaged in figure IV.3 and follows from:

$$\sigma_{y\theta}/\sigma_{y20} = P/P_p (= \gamma) \quad (IV.3)$$

Where:

- σ_{y20} = yield stress at room temperature
 $\sigma_{y\theta}$ = effective yield stress at elevated temperature, as discussed in Section IV.2
 P = the design load on the beam or column under fire conditions
 P_p = for beams : the collapse load at room temperature following from elementary plastic theory
 or
 for columns : the ECCS buckling load corresponding to the so called c-curve.

On the basis of the above expression, the critical temperature for a number of practical values of γ is given in Table IV.2.

γ	0.3	0.4	0.5	0.6	0.7
$\theta_{crit} [^{\circ}\text{C}]$	585	540	490	430	360

Table IV.2: Values for the critical temperature of steel beams and columns as a function of the degree of loading ($\gamma = P/P_p$).

The values presented for the critical temperature can be obtained directly from Fig. IV.3 or can be calculated using the equations for the effective yield stress as a function of the steel temperature, given by equation IV.1.

Using Table IV.2 for beams can give rise to complications, if the analysis at room-temperature has been based on the elastic theory since the maximum load P_e that the beam can support as an elastic behaving structure, and not the collapse load P_p , will be the known quantity. Under other-

wise equal conditions a lower value will be found for P_e than for P_p , the reason being that in an elastic analysis the plastic reserve of the beam is not taken into account. In a more general way this phenomenon can be represented as follows:

$$P_e = \frac{1}{\alpha} \cdot \frac{1}{\theta} \cdot P_p \quad (\text{IV.12})$$

where:

α = factor (> 1) representing the plastic reserve of the beam section;

θ = factor (≥ 1) representing the plastic reserve of the beam due to a possible redistribution of bending moments.

The factor α can be expressed in the quotient of the plastic section modulus Z and elastic section modulus W , i.e.:

$$\alpha = \frac{Z}{W}$$

The value of α thus determined by the cross-sectional shape of the beam, is therefore sometimes called the shape factor. For the usual I-sections α is approximately equal to 1.15.

The factor θ is determined by the redistribution of moments when plastic hinges are formed and is therefore system-dependent. For statically determinate beams no redistribution of moments is possible, and therefore: $\theta = 1$. For statically indeterminate beams: $\theta > 1$.

If P_e is known from the design calculation at room temperature, it is more practical to express the degree of loading likewise in P_e . The degree of loading to be defined in this way is denoted by γ^* .

Thus:

$$\gamma^* = \frac{P}{P_e} \quad (\text{IV.13})$$

By expressing γ^* in γ ($\gamma^* = \alpha \cdot \theta \cdot \gamma$) we can now also determine the values of the critical temperature corresponding to γ^* . This is summarized in Table IV.3 adopting a shape factor $\alpha = 1.15$.

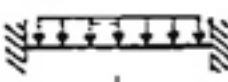
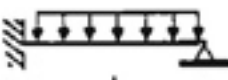
		factor $\gamma^* = \frac{P}{P_e}$				
		0.3	0.4	0.5	0.6	0.7
statically determined $\theta = 1.00$		605	565	525	475	425
statically indetermined						
$\theta = 1.33$		640	605	575	545	510
$\theta = 1.0$		605	565	525	475	425
$\theta = 1.47$		650	615	590	560	535
$\theta = 1.12$		615	580	545	505	465
$\theta = 1.47$		650	615	590	560	535

Table IV.3: Values of the critical temperature of steel beams as a function of the degree of loading when the latter is expressed terms of . the maximum elastically supportable load P_e ($\gamma^* = P/P_e$).

The effect of the load is clearly apparent from Table IV.2 and IV.3. It further appears that with elastic design, the critical temperature may be considerably influenced by the structural system.

IV.8.2 Design load P under fire conditions

Comparison of the calculated critical temperatures with the results of standard fire tests on beams and columns shows that in the former case critical temperatures are found which are 50° - 100° C lower than in the latter. Hence the calculation provides "safe" solutions in relation to the standard fire tests. The difference between the two sets of results is attributable to the necessary idealizations on which the mathematical model is based. Important factors in this context are:

- In the mathematical model the design value of the yield stress for the grade of steel employed is adopted. The design value is equal to the minimum guaranteed value of the yield stress, whereas in fire tests it is the actual value of the yield stress that matters.
- In the calculation the temperature is assumed to be uniformly distributed throughout the structural member. In fire tests - as in a real fire - considerable variation in temperature both over the height and along the length of the member will occur. The maximum measured temperature at the instant of collapse is often regarded as representing the critical temperature.
- Structural behaviour, especially of columns, is influenced by geometrical imperfections and residual stresses which in tests can be considered as random samples from a wide variety. In calculations, conservative design values are adopted.

Experiments as well as exploratory calculations show that the differences between the calculated results and those obtained in a standard fire test can be almost completely explained by the above-mentioned factors [22], [23]. All the same, the phenomenon in question does constitute a

complication. Standard fire tests have in fact been established for a great many years, and their results are generally accepted as valid. It is therefore somewhat unrealistic to offer a method of analysis which yields systematically safer results than the standard tests. For practical reasons it is, accordingly, proposed that the design assumptions be so modified that critical temperatures are found which are at the same level as those obtained from standard fire tests. This can be done in alternative ways.

One way is to multiply the calculated load bearing capacity - corresponding to the calculated uniformly distributed steel temperature θ_s - by a factor of magnification f which includes corrections for representative deviations from the assumptions listed above. Such a magnification factor then can be written as

$$f = f_m f_1 f_{\theta_c} f_{\theta_a} \quad (\text{IV.14})$$

where f_m = a correction factor related to the mechanical properties of the structural material at temperature θ_s , f_1 = a correction factor related to the imperfections of the structural member, f_{θ_c} = a correction factor in respect of non-uniformity in the temperature distribution over the cross section, and f_{θ_a} = a correction factor in respect of non-uniformity in the temperature distribution along the structural member.

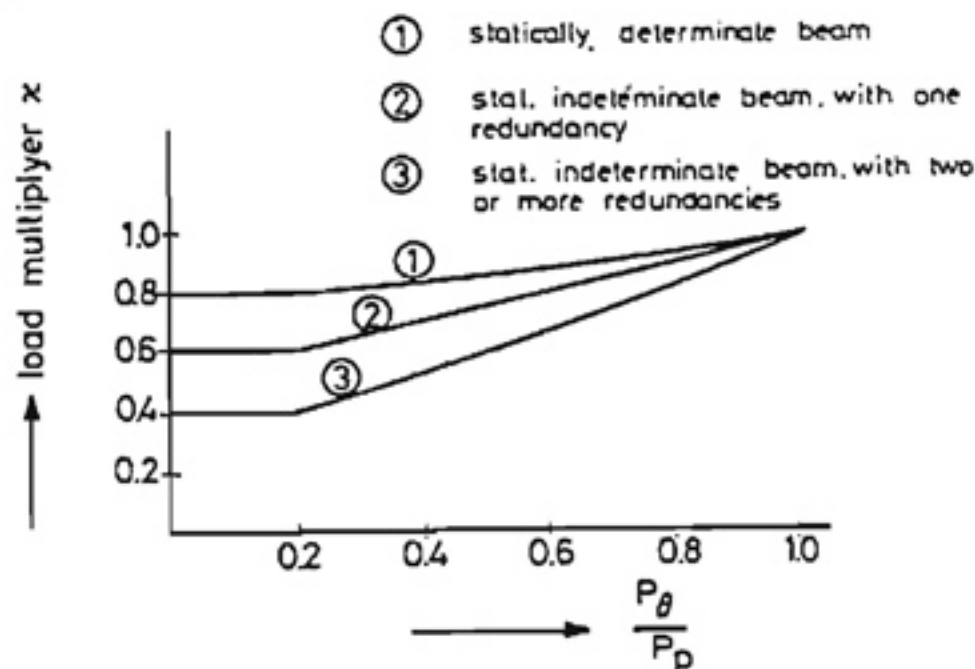
Representative values are derived in [23] for the various correction factors as well as the total magnification factor f . Practically, f then can be given as a function of the uniformly distributed, calculated steel temperature θ_s with a differentiation with respect to columns, statically determinate beams and statically indeterminate beams with one or more redundancies.

The same results can be achieved by reducing the design load in the calculation, P_{cal} , by a load multiplier κ , i.e.

$$P_{cal} = \kappa P_{\theta} \quad (IV.15)$$

where P_{θ} = the load to be applied on structural element in the fire resistance test.

Values for κ derived for beams are presented in Fig. IV.13 [23].



Design values for the load multiplier κ .

Fig. IV.13

The following equations hold:

statically determinate beam:

$$\kappa = 0.25 \left(\frac{P_{\theta}}{P_p} \right) + 0.75 \geq 0.8 \quad (IV.16^a)$$

statically indeterminate beam with one redundancy:

$$\kappa = 0.50 \left(\frac{P_{\theta}}{P_p} \right) + 0.50 \geq 0.6 \quad (IV.16^b)$$

statically indeterminate beam with two or more redundancies:

$$\kappa = 0.75 \left(\frac{P_{\theta}}{P_p} \right) + 0.25 > 0.4 \quad (\text{IV.16}^c)$$

It is apparent that the value of κ for beams depends both on the design load and on the type of structural system. The value of κ for statically determinate beams will be higher than for indeterminate ones, the reason being that, in the former, one plastic hinge will suffice to bring about collapse, whereas two or more such hinges will be needed for collapse of a statically indeterminate beam. The variations in temperature along the length of the beam will therefore in principle only be of importance for statically indeterminate beams. In the case of beams with a higher degree of statical indeterminacy this effect is greater than in those with first-degree statical indeterminacy.

Accordingly, a load multiplier κ should also be introduced for columns. In view of the considerable approximations necessary when deriving the calculation rule for the critical temperature, it is not realistic to differentiate κ in the case of columns. Therefore a single (conservative) value has been chosen: $\kappa = 0.85$.

V. FIRE SAFE USE OF STEEL STRUCTURES

V.1 Introduction

The fire resistance of a structure is the time during which the structure, when submitted to the standard fire, continues to fulfil its load-bearing function.

Structural behaviour depends on many factors, as has been explained previously (chapters I to IV) :

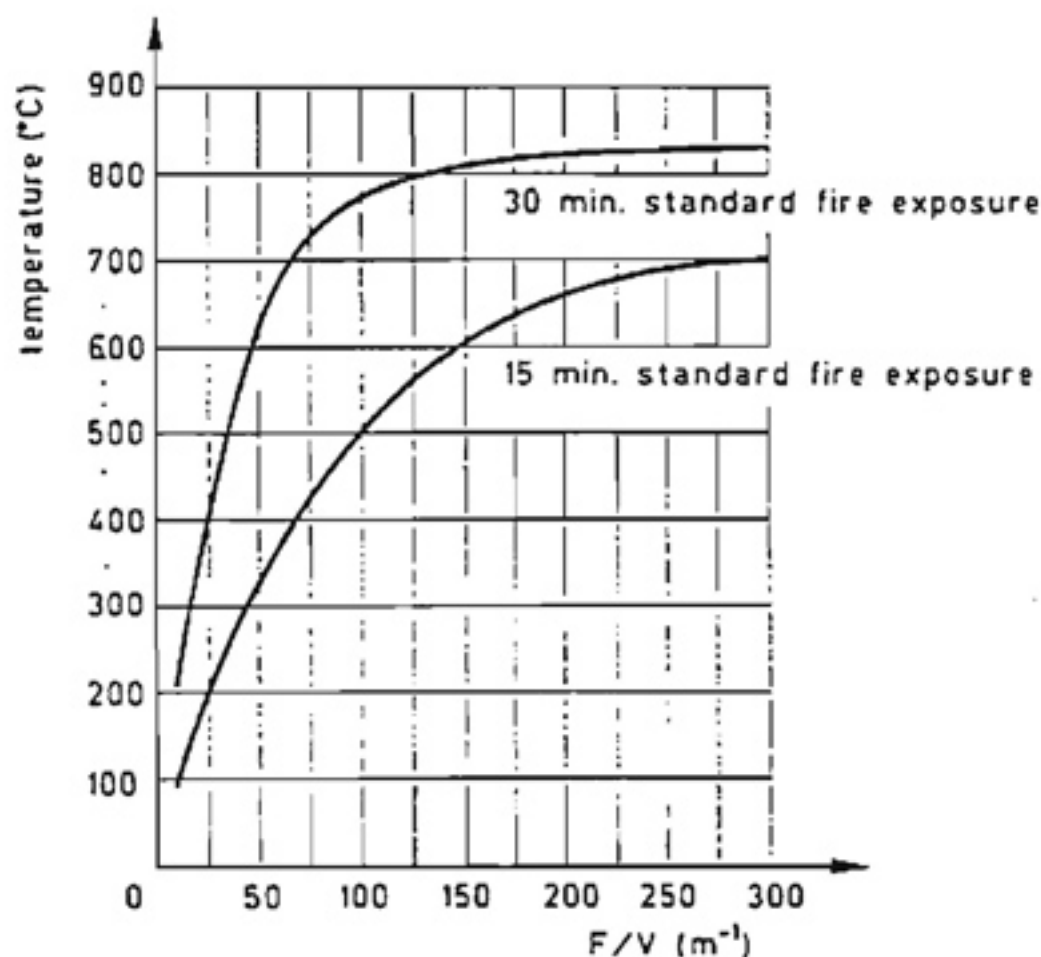
- the intensity of the fire which is a function of the amount, nature and location of the combustibles, the conditions of ventilation of the compartment and the thermal insulating characteristics of the surrounding walls of the compartment,
- the type of structure,
- the nature and properties of the materials making up the structure,
- the loading level,
- the heating up conditions.

Since the intensity of the fire has been, and still remains to some extent, a difficult parameter to quantify, many countries have adopted the "standard fire". Although this approach provides a comparison of the fire behaviour of many different structures, it does not enable a real structural fire safety analysis of a building to be carried out.

The heating of an unprotected steel structure subjected to this "standard fire" is often very rapid and collapse may occur very quickly, within some ten to thirty minutes of the standard exposure.

By assuming a uniform distribution of temperature in the steel, the calculation of the time for the critical temperature to be reached is simple (see chapter III). At this temperature, the structure would collapse as it would be unable to bear the loads applied.

From tables or graphs which give the temperature reached by the steel as a function of the time and the section factor F/V , it is possible, knowing the critical temperature, to determine the actual fire stability. An example is given in Fig. V.1, which is based on Appendix D.2.



Temperature of unprotected steel exposed to the standard fire as a function of time and section factor (F/V), according to the ECCS-TC3 Recommendations [1].

Fig. V.1

According to these graphs, most unprotected steel structures will have a low fire stability. A hasty and unfounded reaction would lead to the conclusion that the safety of people cannot be assured in unprotected steel buildings. Such erroneous thinking is largely based on the deep seated idea that adequate fire safety can only be obtained when the structure has a large fire resistance.

Furthermore, these low values of fire resistance are related to the standard fire which, in the opinion of many specialists, is not representative of actual fires in buildings.

Before investigating the current means of thermal protection aimed at increasing the fire resistance of steel structures (V.4) it is therefore of prime importance to set down the circumstances in which unprotected structures can meet the requirements of the regulations in force at the present time (V.3) and above all to analyse the concept of structural fire resistance in relation to the safety of both human lives and property (V.2).

V.2 The role of structural fire stability

Substantial fire stability is obviously essential for high-rise buildings, since a fire taking place in the lower storeys which lead to the collapse of a tall building would have catastrophic consequences. On the other hand, for low-rise buildings, a thorough analysis should be carried out in order to establish the precise role played by the structural fire stability in relation to the general safety of the building.

In the event of a fire, the safety of the occupants should be ensured by escape routes of adequate width which are appropriately signposted, easily accessible, protected from smoke and which provide protection from falling debris. The evacuation of the occupants should be completed before a dangerous concentration of toxic gases exists and before ambient temperatures rise above 80 - 100°C.

With regard to the safety of the contents, the combustibility of which is quite variable, there is little that can be done for the contents of the space where the fire occurs, if automatic extinction measures have not been installed. On the

other hand, firestop walls can prevent extensive fire spread and, as a consequence, the destruction of part or most, if not all, of the contents can be avoided.

Fire stability of structures in low-rise buildings plays a limited role in the protection of either human safety or contents safety.

As an example, consider a single-storey building. The great difference between the limit temperature (100°C) for occupants' survival and that likely to cause the collapse of the building (600°C .- 700°C) indicates that the collapse of the structure is very unlikely to be the reason for loss of life. In this case it is not the duration of fire stability duration which is important, but the ambient temperature rise. It does not matter whether the critical temperature of the structure is reached in 10 or 60 minutes, because well before the stage at which such a temperature would be reached, the ambient temperature in the enclosed space would have become unbearable and all the occupants would clearly have needed to have made their escape. In any case, structural steel members do not collapse suddenly in a fire; depending on how the temperature rises, they deform and so provide an additional element of warning to occupants protected from the fire but near the fire zone.

The fire stability of a structure is especially important for occupants above, but much less for those beneath the part of the structure exposed to the fire. However, in the latter case it is important that sudden collapse does not occur. The failure should be gradual, involving large plastic type deformations and this collapse should be confined only to the fire zone. The parts of the building away from the fire should remain intact so as to allow for the safe evacuation of the occupants and to permit the firemen to operate.

For the safety of contents in single-storey buildings, it is only necessary that structural parts be independent on both sides of firestop (compartment) walls. In fact, steel structural members reach their critical temperature only if large parts of the contents have caught fire. At this stage, it would be difficult to intervene to save the contents, but collapse of the structure over this area should not damage the compartment wall and thus endanger the contents of the adjacent spaces.

In this type of building, the only point of providing thermal protection to the steel members is to preserve the structure. As a result, the problem is one of economic feasibility rather than of public safety. Very often, the building value represents only 25 % of the value of the total building plus contents ; of this percentage, the structure itself represents some 25 %, i.e. 6 % of the total cost. Hence it is a question of striking an economic balance before deciding to increase the fire resistance of low-rise buildings [2].

Before going more deeply into the analysis of structural fire stability, it should be recalled that in the regulations requirements have been determined in a most empirical way ; many compensating clauses have been introduced in order to correct the initial approximation caused by using the same standard fire for every type of building and activity.

Furthermore the values of fire grading are expressed as submultiples or multiples of a time unit : the hour. This may be of no consequence for protected steel structures because an increase from 1 h 20 min to 1 h 30 min of fire stability requires only some few millimetres of additional protection at a relatively low cost. However, unprotected structures which provide a 20 min fire stability must be protected to obtain a 30 min fire duration, so the relative cost involved in providing this small increase in fire resistance is generally high.

It is thus of prime importance that regulating requirements are involved, which reflect the actual needs of both the occupants and the fire services. A correct analysis would avoid the error made by certain national regulations which require fire stability for structures with low calorific potential. For example in sports halls, meeting rooms, there are very low risks of flash-over or of any fire which could endanger the stability of unprotected steel structures. At the worst, if there is a concentration of combustible in one area, there could be a localized fire but without flashover occurring over the whole enclosed space. For fire loads per unit floor area less than 15 kg/m^2 (wood equivalent), it would be generally unnecessary and futile to improve the structural fire resistance. Instead, the real dangers posed by smoke and toxic gases should be tackled, using automatic detectors and smoke control.

Some of the national regulations have begun to recognise the true role of the structure in the fire safety of a building and alternative measures have already been taken into account, to give more flexible requirements for low-rise constructions. A few others take a more thorough approach to the real dangers by aiming to adopt the most appropriate safety measures.

V.3 Unprotected steel complying with regulation requirements

When fire stability is necessary, the minimum requirement in most classification systems is $1/4 \text{ h}$. This fire stability is easily reached by unprotected steel structures.

However, $1/2 \text{ h}$ fire stability under the standard fire conditions is certainly a most unfavourable requirement where steel construction is concerned. While steel very often needs thermal insulation to meet this requirement, other building materials generally achieve it unaided.

The only structural elements needing to be checked from the fire resistance point of view are those which actually contribute to the general stability. Purlins and ties whose sole function is to fix the cladding elements need not be dimensioned to resist fire effects, whereas wind bracings may need to comply with the regulation requirements for the fire resistance of load carrying structural members. However, in big warehouses, a reduced fire resistance for bracings may be adequate, provided that these elements are plentiful and well distributed [3]. In any case purlins and the sheeting are generally capable of providing sufficient structural stability for 20 minutes or even half an-hour.

If the steel members must comply with the fire resistance requirements, a fire resistance of as much as 60 min may sometimes be obtained without any protection either by correctly applying present day knowledge of the heating of structural steel members and their collapse temperature, or by using construction methods which enable composite action of steel and concrete. Alternatively, by placing the load carrying structure outside the building the fire exposure may be reduced.

V.3.1 Heating process and critical temperature

Table D.2 shows that after a 1/2 hour exposure to the standard fire, steel elements with low values of section factor (30 m^{-1}) will have a temperature lower than the critical temperature of 500°C - 550°C generally quoted for steel structures. Under these conditions, the stability of the elements concerned can be ensured without providing fire protection. These types of massive elements are, however, rare in steel construction and elements with section factor values of 200 m^{-1} are more commonly found.

Furthermore too great an increase in the mass of steel of such elements would generally be too expensive in comparison with the cost of a normal protective coating.

It is therefore necessary to consider raising the critical temperature of the steel elements by decreasing the load level; see also chapter IV.

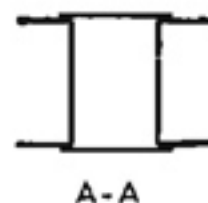
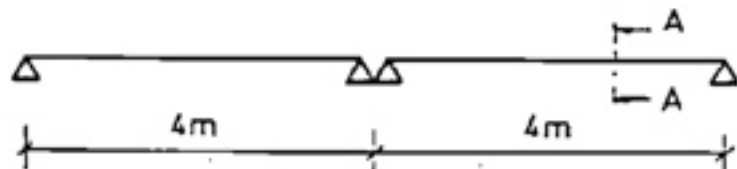
This can be done in several ways :

- reduce the applied loads in each member by increasing the number of load carrying structural members,
- increase the mechanical resistance of the load carrying elements,
- use statically indeterminate elements or, in a more general way, take proper account of the real end conditions at the connections between the structural elements.

Accordingly, a combination of a low section factor and a high critical temperature can provide 1/2 hour fire resistance without the need for any protection ; of course, this procedure should be embarked on at the design stage of the construction project.

The following example shows that in order to provide a beam to support a given uniform load (32 kN/m^1) over two adjacent spans of equal length (4 m), different solutions can be obtained.

The first solution consists in using two simply supported beams, each built up of 2 UNP 300 standard beam section connected by steel flats (Fig. V.2). The maximum stress is 60 N/mm^2 , the load level: $P_\theta/P_p = 0.22$, the load multiplier $\kappa : 0.81$ and so $P/P_p = 0.18$.

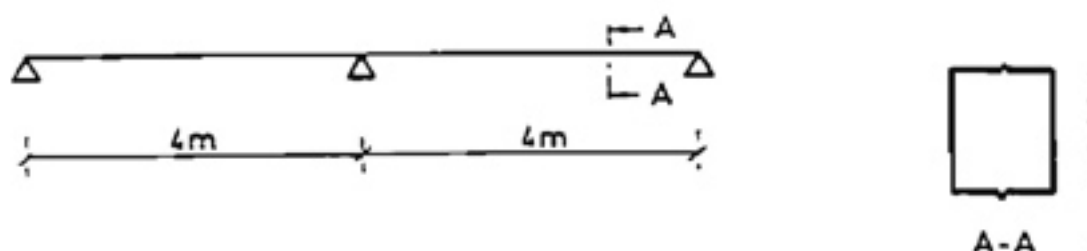


Solution 1: two simply supported beams

Fig. V.2

In this case, a critical temperature of 658°C is obtained and with a section factor of 162 m^{-1} , provides 16 min fire stability under the standard fire conditions (see Fig. V.1).

The second solution consists in adopting the same standard beam sections but closed to form a box section and providing a beam continuous over the central support (Fig. V.3). The maximum stress based on elasticity is still 62 N/mm^2 , but the load level is now only 0.15, the load multiplier 0.57 and so we obtain $P/P_p = 0.085$.



Solution 2: one continuous beam.

Fig. V.3

The critical temperature is 753°C and with a section factor of 85 m^{-1} , it has a fire stability of 30 min.

It may alternatively be possible to demonstrate that a fire stability less than 30 min is amply sufficient, so that an unprotected steel member could then be used.

V.3.2 Composite (steel-concrete) constructions

By combining steel and concrete, it is often possible to obtain a 1/2 hour fire stability or even more, without protecting the steel. The most important constructions of this type are : concrete filled hollow tubular sections with or without steel reinforcement or steel core, rolled steel columns with concrete encasing, and concrete slabs with profiled steel sheet. The design of calculation methods for these elements is the subject of additional technical notes to the European Recommendations for the Fire Safety of Steel Structures [4,5].

V.3.3 External structural elements

By placing the columns outside the building they are thus moved away from the potential fire zone. In this way fire stabilities much greater than $\frac{1}{2}$ hour can be achieved. However, on account of the complex process of heating and the associated mechanical behaviour of such loaded elements, the accurate calculation of the structural stability is difficult. Of course, in this case, exposure to a natural fire and not to the standard fire must be assumed [6,7].

Nevertheless, existing approximate calculation methods and experimental results enable us to make the following observations:

- the best position for the column is as far as possible from openings, and shielded by a wall between the windows which has an adequate firestop rating;
- if there is a risk of the column being submitted to severe thermal loads, the role of the mechanical behaviour of the beam to column connection becomes very important; the best results are obtained by using rigid connections, for a given stress level in the column;
- in order to restrict exposure to heating, the column should either be moved as far away as possible from the facade or be protected by a screen;
- generally, the load carrying floor beams need no protection over their external parts.

Structures with external components can therefore offer an interesting solution to the problems of fire stability of steel structures.

V.4 Protected structures

If bare steel structures cannot provide the required fire stability, there is always a possibility of reducing the heating rate of the steel element by using thermal protective materials. Depending on the nature and thickness of the material, fire resistances of $\frac{1}{2}$ to 4 h can be obtained.

The various protection methods and procedures available on the market [8,9] can be grouped as follows:

- protection enclosing the element: intumescent paint, sprayed materials or protective casing;
- protection by screens;
- water cooling of the steel.

V.4.1 Enclosure protection

- a. Contour protection using an intumescent paint or a sprayed product (figure V.4.a).

Though similar in appearance to ordinary paints, intumescent paints contain special agents which cause expansion at about 150 to 300°C to form a "meringue" whose thickness may reach several centimetres: it is this coating of "meringue" which acts as the heat shield.

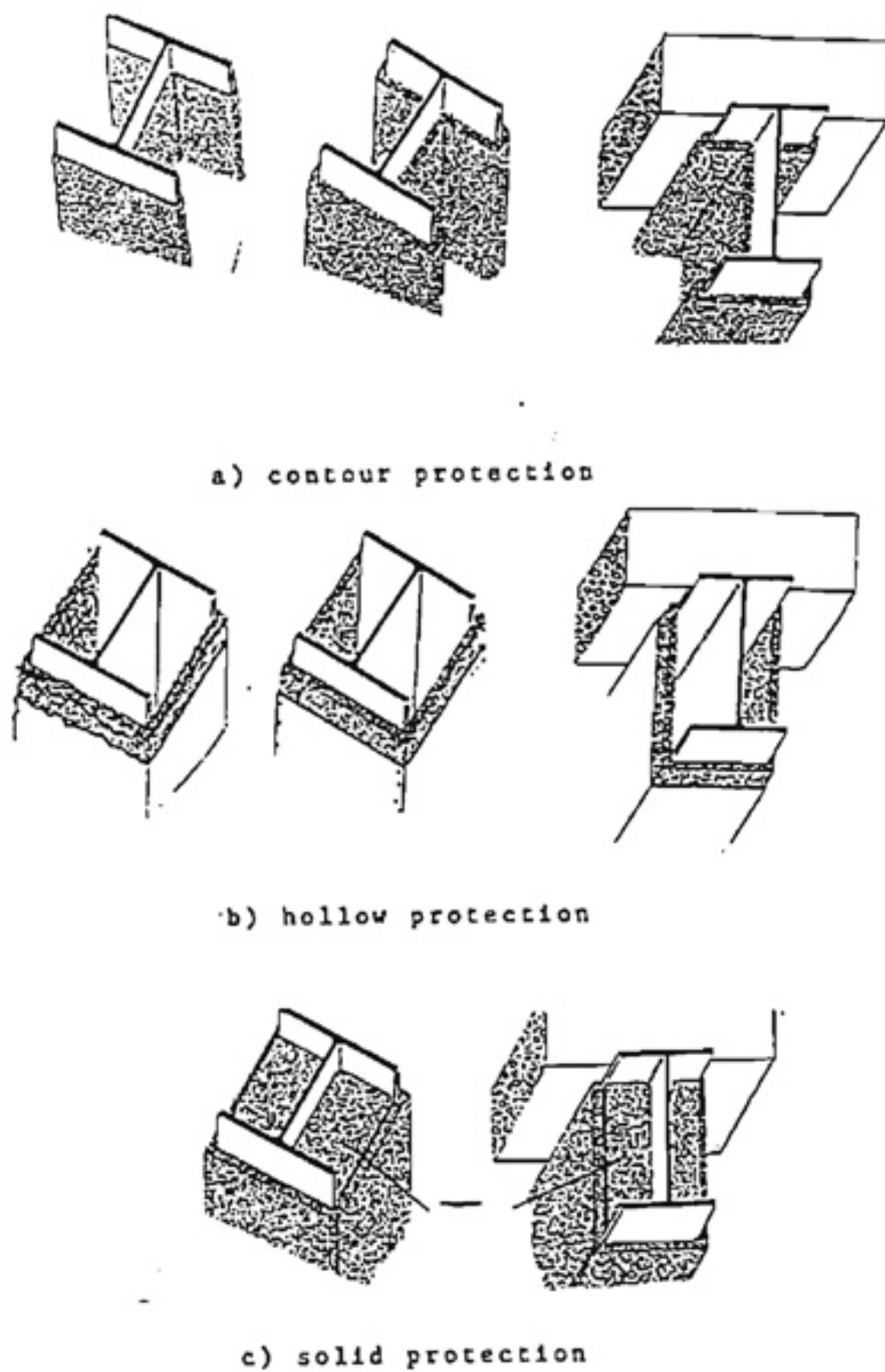
The sprayed products are composed of materials such as gypsum (which includes 20% crystallized water), expanded vermiculite, expanded perlite, mineral fibres, asbestos fibres and binder. However, asbestos fibre is now prohibited in many countries because of the danger to workers and occupants. Composite materials are often employed, such as gypsum plaster + vermiculite, cement + vermiculite, gypsum plaster + fibres,etc.

When these materials are used inside a building, the steel surface does not generally require an anti-corrosion treatment, unless the relative humidity of the atmosphere is high. The main difficulty encountered in permanently damp conditions is not the risk of steel corrosion, but the potential loss of protective properties of the coating.

- b. Hollow protection carried out by spraying on steel mesh or by boards (figure V.4.b). The composition of the boards can be similar to that of the sprayed product mentioned above, but in addition other products can be used, e.g. timber. As a matter of fact, although timber is a combustible material, it provides good thermal insulation and can be used to protect steel which, while incombustible, provides poor thermal insulation.

When using box protection, the shrinkage and the deformation of the panels due to rising temperature should be taken into account to prevent leakage of hot gases through the joints. A different solution consists of simple steel sheet which can provide a fire stability of $\frac{1}{2}$ h.

- c. Solid protection of the steel either in cementitious-type product or loose fill such as vermiculite or expanded clay in steel casing (Fig. V.4.c).



Methods of fire protection

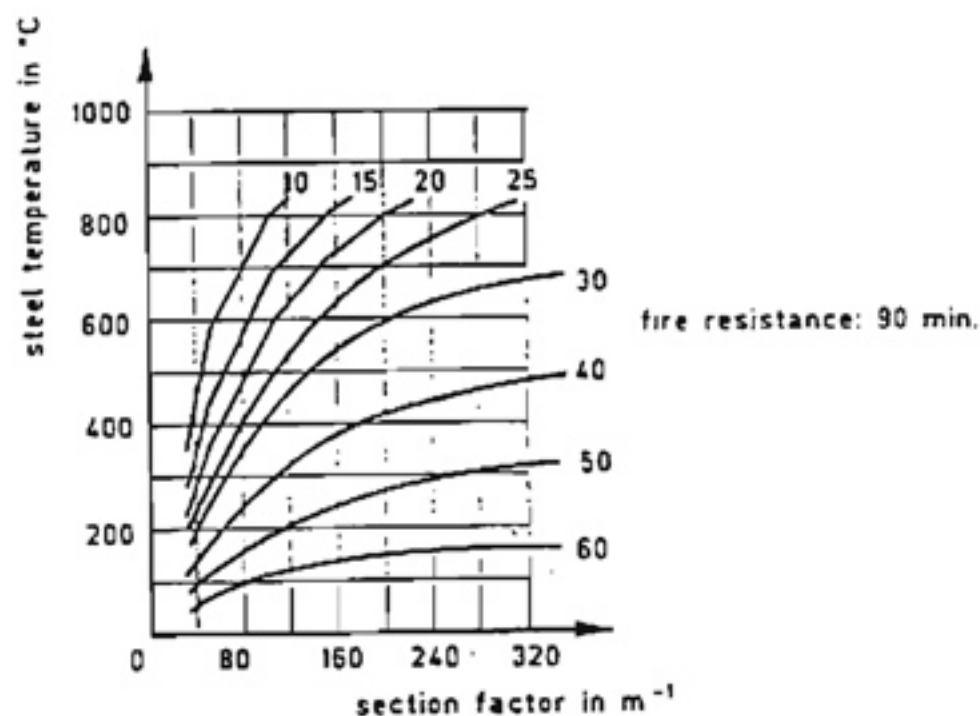
Fig. V.4

The desirable properties of these materials are as follows:

- good thermal insulation i.e. low thermal conductivity and/or high thermal capacity;
- satisfactory mechanical resistance to shock and impact at both normal temperature and fire temperature and good attachment or adhesion to the element so that separation or spalling of the protection material is not caused by rising temperature and/or deformation of the supporting element.

The condition of the steel surface, the application technique and the jointing methods for both sheet to sheet and sheet to steel, particularly in the region of structural connections, should be given careful attention since the protection plays a role of primary importance in achieving the fire stability. Consequently, these materials should be subjected to tests aimed at checking their thermal and mechanical characteristics at steel temperatures up to 700 - 800°C.

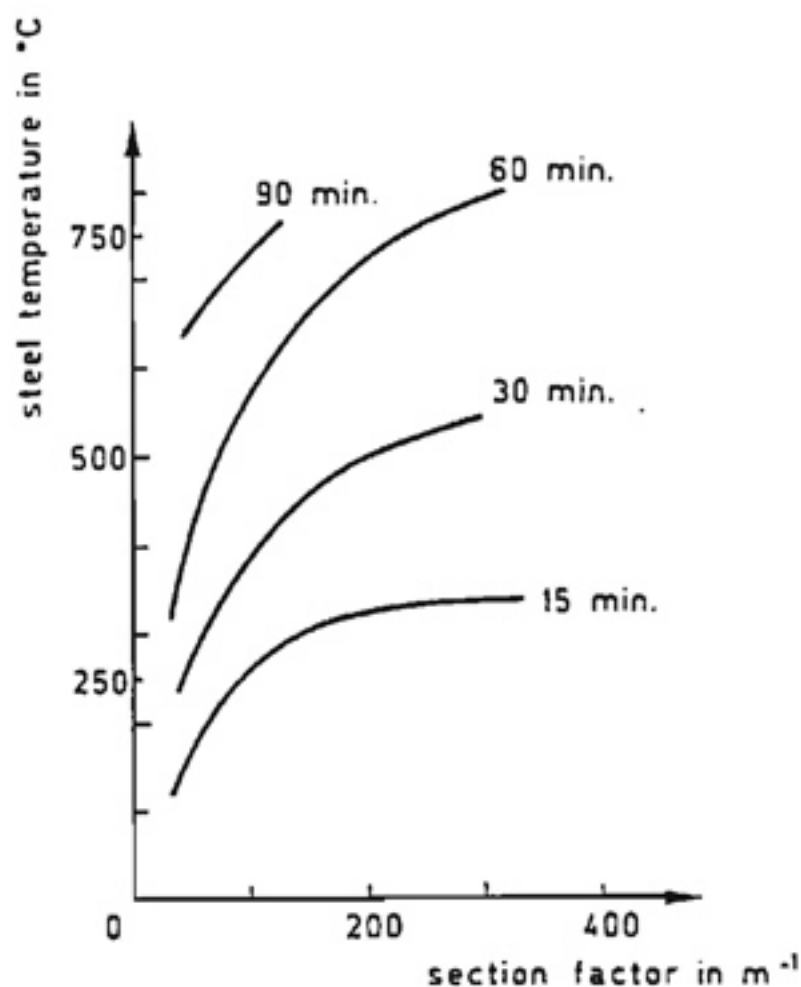
To facilitate the use of sprayed materials and sheet materials, special graphs have been prepared by authorized fire testing laboratories. These graphs give the thickness of a specific material as a function of the section factor, the critical temperature of the structural member, and the required fire resistance period (Fig. V.5) [10,11]. Values given in Appendix D.3 can also be used.



Example of a graph from which the required insulation thickness can be derived for a particular fire resistance, given the section factor and the critical temperature.

Fig. V.5

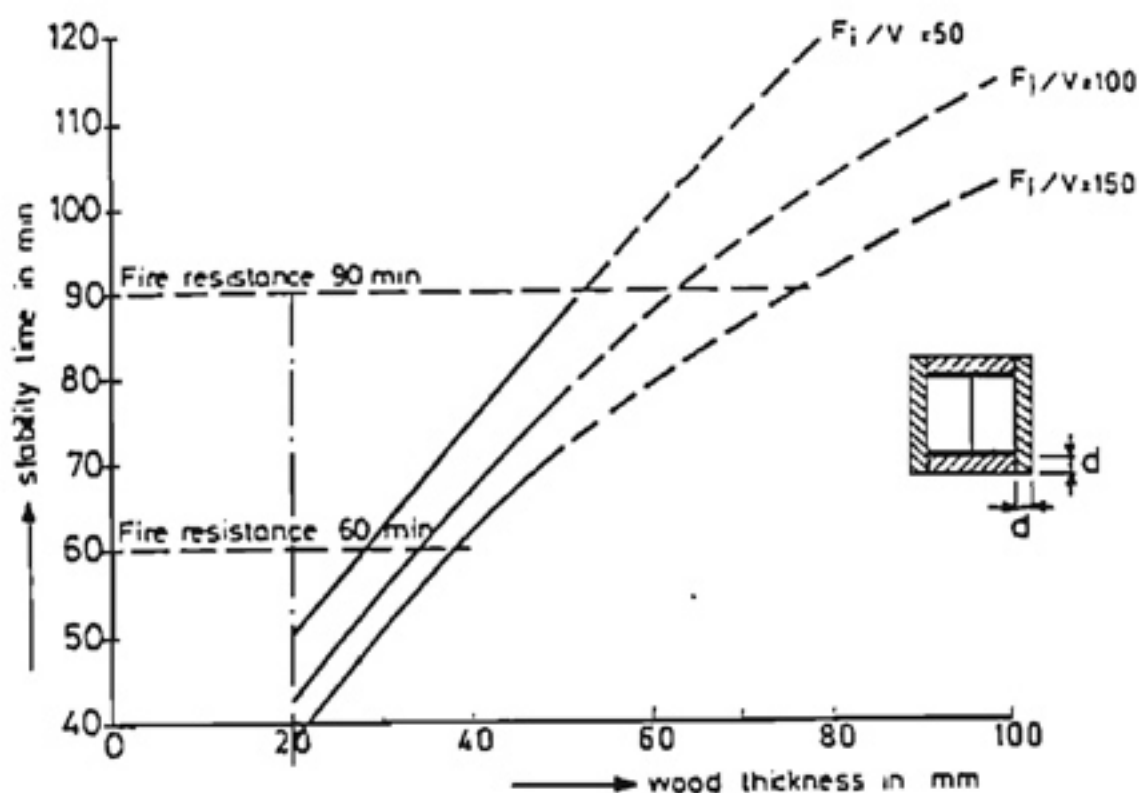
As regards intumescent paints, a similar although slightly simplified graph can be obtained since only one or two coats of a specified thickness of paint can usually be used (Fig. V.6). The graph shows that fire resistance periods vary from about 30 min, for structural members with large F_1/V values and low critical temperatures, up to 90 min for members with small F_1/V values and high critical temperatures.



Fire protection provided by a single coat of specified thickness of a particular intumescent paint.

Fig. V.6

When using timber boards as fire protection, Fig. V.7, which incorporates a sizeable safety margin, can be used to determine the thickness as a function of the required fire resistance. It is not advisable to use timber boards less than 20 mm thick [12].



Fire protection by timber boards.

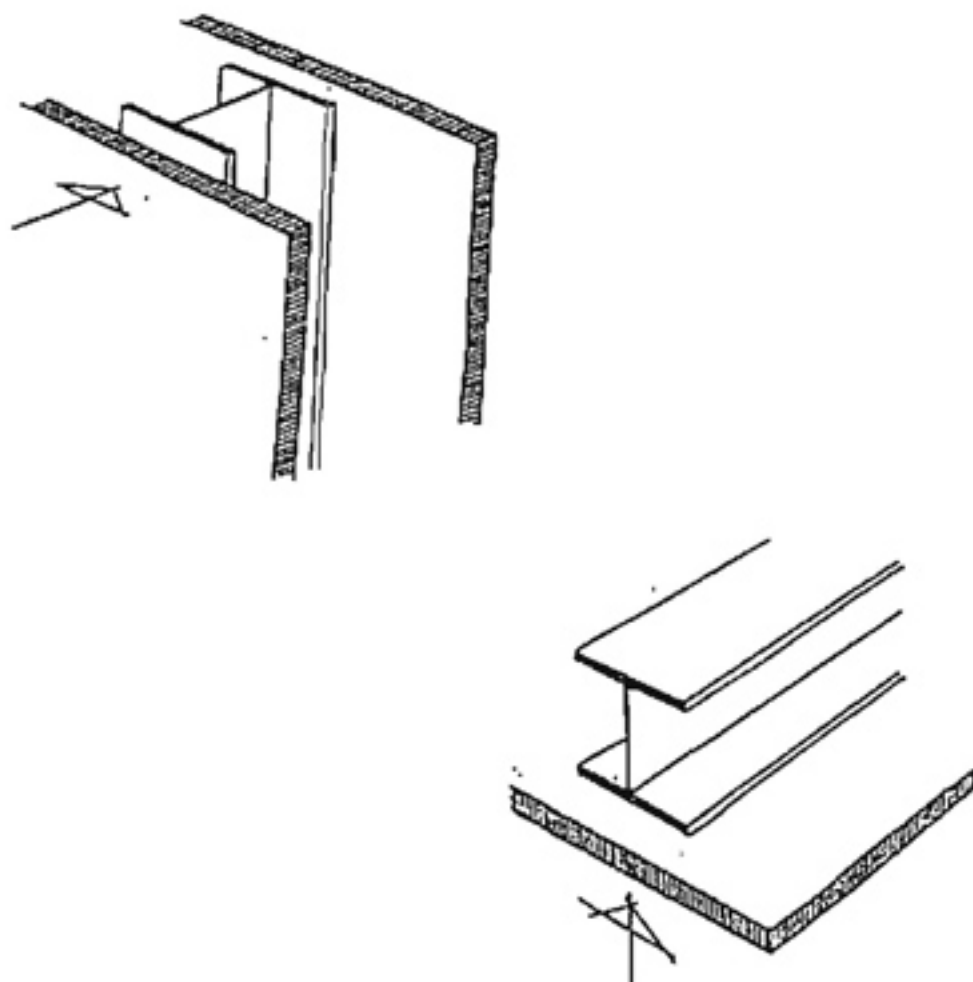
Fig. V.7

V.4.2 Fire protection by screens

Fire protection by screens can consist of suspended ceilings or of partition wall panels (Fig. V.8).

Protective screens offer the advantage of combining the function of providing safety against fire with other functions such as thermal insulation, sound proofing and aesthetics, which results in appreciable reductions in the cost of the fire protection. Nevertheless, because the behaviour of the whole depends on that of the weakest part of the screen assembly, special attention should be paid to the method of assembly and in particular to the joints and connections. It is also necessary to have regular cavity barriers within the wall panels or above the suspended ceiling, in order to restrict the movement of combustion gases which could emanate from the fire and enter the void at some local perforation of the screen. Moreover, the voids are often used for installing electric cables and other service facilities. Such use of the

voids can often result in partial damage of the screens, hence the need for making the parties responsible for the installation and maintenance of these services aware of the fire protective role of these elements.



Fire protection by screens.

Fig. V.8

We do not yet have at our disposal methods which are generally applicable for the calculation of the heating up of steel members protected by screens or slabs (in the case of floor with suspended ceiling). Estimates can nevertheless be obtained by interpolation and extrapolation of test results.

Thus, having obtained a fire test result of a particular steel section, with a section factor $(F/V)_{\text{test}}$, protected by two screens, it is possible, to estimate the temperature (and so the fire behaviour) of steel sections with different values of F/V protected by the same screens, by using one of the following:

- the temperature recorded in the steel if $F/V > (F/V)_{\text{test}}$;
- the temperature recorded in the void between the two screens, if $F/V < (F/V)_{\text{test}}$;
- the temperature recorded on the interior side of the fire exposed screen, if the steel profile is to be nearer to this screen than was the case in the test.

Some modifications to the unexposed screen or slab (in the case of a floor with a suspended ceiling) can be made, but only if the modified screen or slab does not reduce the heat loss experienced during the reference test.

V.4.3 Water cooling

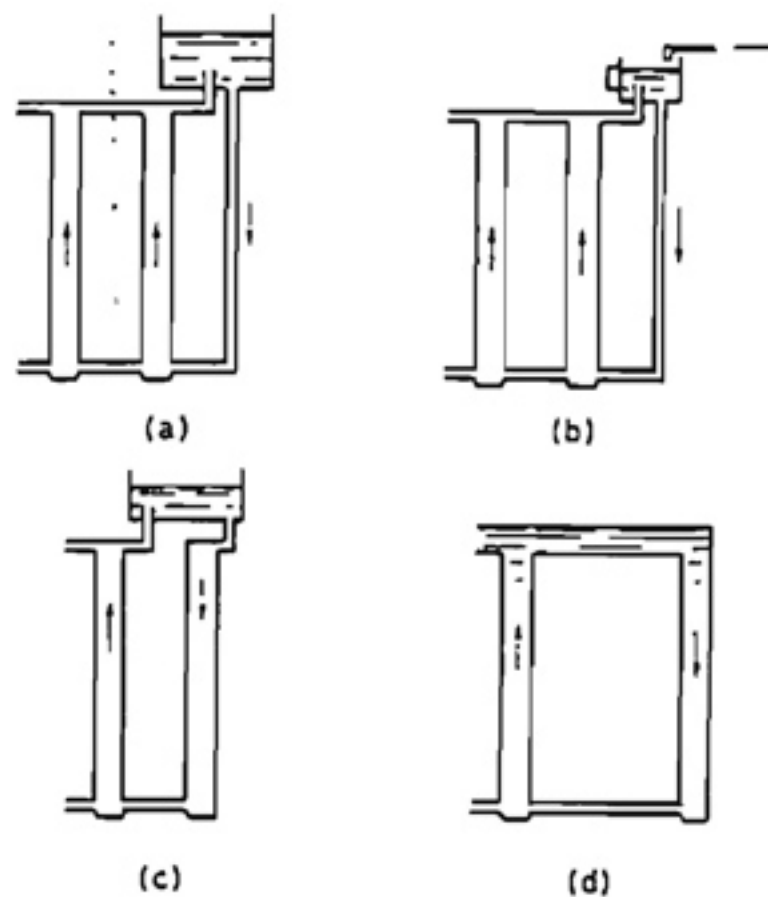
Structural hollow sections permanently filled with either circulating or non circulating water (Fig. V.9) are the main features of this method.

Circulating water procedures rely on the heating of the water at the fire level to create a natural thermosiphon action but this circulation can be accelerated by a pump. At the top of the building, an expansion tank is installed to avoid over-pressure.

Particular attention is required to ensure that the water levels in the members are maintained at all times, and that the inside surfaces of the hollow section are such as to avoid the formation of air bubbles which may cause localized over heating of the section.

When using non circulating water, the members - essentially columns - are filled with water and the water level must be maintained in each element separately. For isolated structural elements, it is possible to promote circulation of water by placing a second tube of small diameter inside the section.

With any system, although the water temperature can exceed 100°C , depending on the pressure, the steel temperature will always be well below its critical temperature [13].



Various possibilities of water cooling

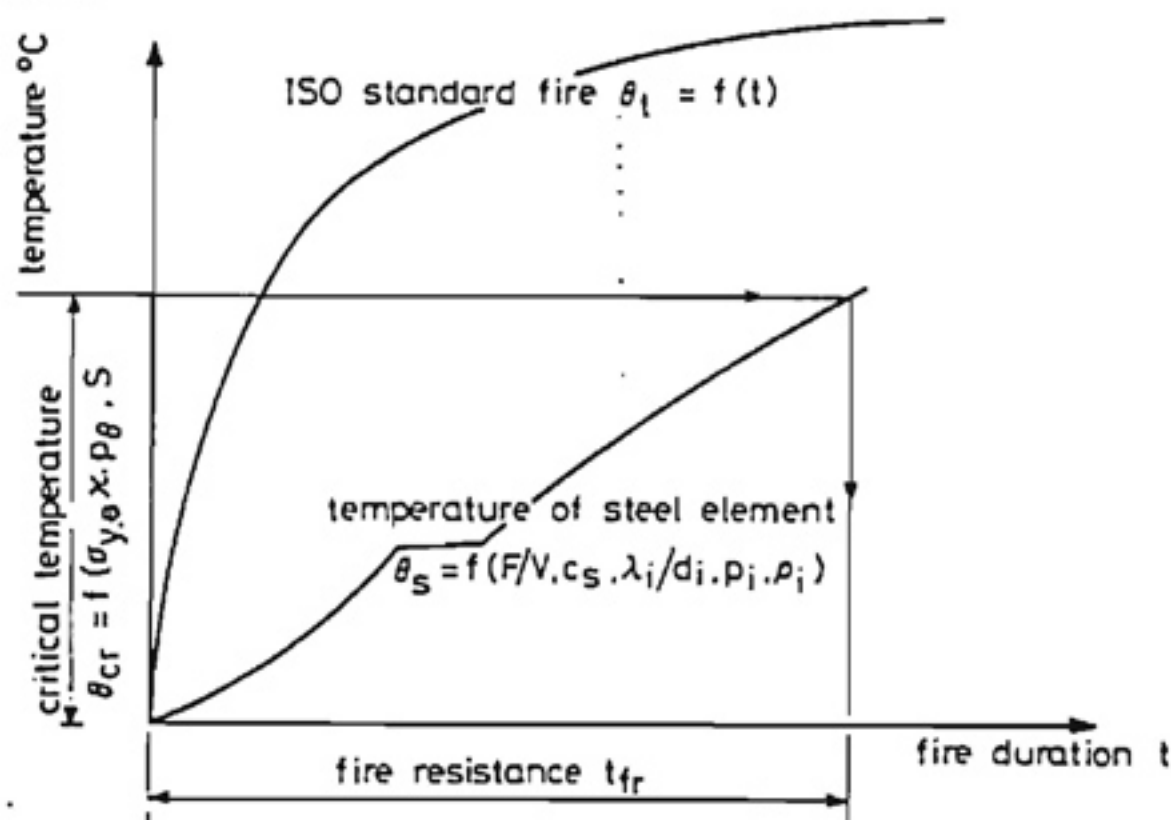
Fig. V.9

Standard rolled beam and column sections (I or H) may be cooled by water spray. The procedure consists in directing towards these elements two or three sprinkler sprays which come into operation as soon as the ambient temperature exceeds, say, 100°C. The water flow must be provided in such a way that there is a continuous film of flowing water over the entire length of the member [14], [15].

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VI CALCULATION EXAMPLESVI.1 Introduction

The calculation of fire resistance consists of the computation of the temperature development in the fire exposed steel element and of the computation of the critical temperature of this element. The fire resistance is the time necessary for the steel element to reach its critical temperature. See Fig. VI.1.



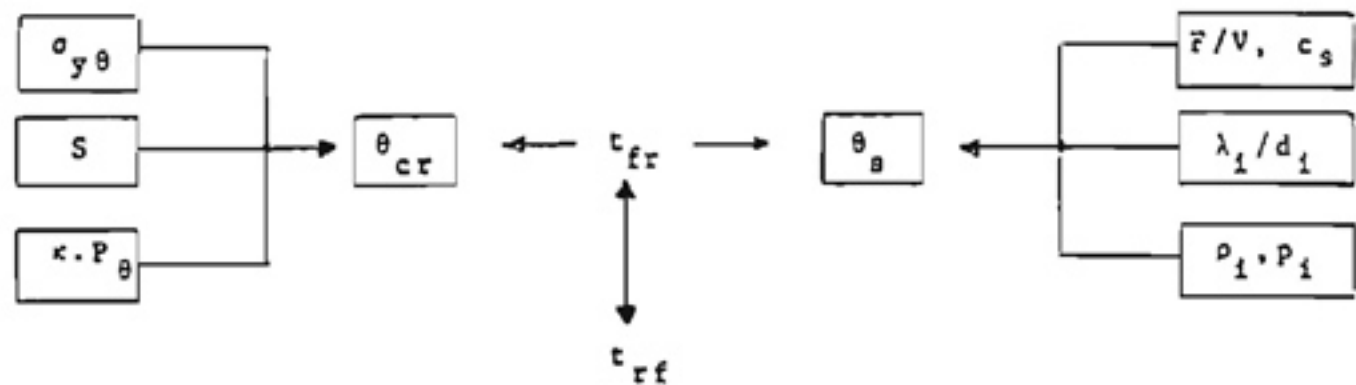
Basis of analysis

Fig. VI.1

The following data are necessary for such a calculation:

- properties of steel at elevated temperatures, viz.: the yield stress $\sigma_{y\theta}$ and specific heat c_s
- structural behaviour, S , of the steel element; (mostly beam or column)
- design load in fire safety calculation κP_θ
- the section factor of the steel element F/V
- thickness d_i and the thermal conductivity λ_i of the insulation
- density ρ_i and the moisture content p_i of the insulation

The procedure for calculating the fire resistance is, schematically, presented in Fig. VI.2.



Procedure for calculating the fire resistance.

Fig. VI.2

According to the initial data given, the calculation can be conducted in three different ways, to prove that:

1. the time necessary to heat the steel to the critical temperature is longer than the required fire resistance

$$t(\theta_s = \theta_{cr}) > t_{rf}$$

2. the temperature θ_s reached after a fire duration which is equal to the required fire resistance, is lower than the critical temperature

$$\theta_s < \theta_{cr}$$

3. the loading of the element during the fire is lower than the loading which would cause, at the temperature reached at the required fire duration, the failure of the structural element

$$\kappa.P_\theta < P$$

The various options of the calculation procedure will be illustrated in the following Sections, using the practical design tables as presented in Appendix D.

Finally it is emphasized that in the following calculations it is implicitly assumed that stickability of the applied insulation, if any, is satisfactory. In practical situations this should be demonstrated explicitly by experimental evidence.

VI.2 Check on actual fire resistance

Course of calculation:

1. Determine the load level $\kappa \cdot P_{\theta} / P_p (= \gamma)$.
2. Determine the critical temperature θ_{cr} .
3. Determine the time t_{fr} necessary for the steel member to reach a temperature $\theta_s = \theta_{cr}$.

Example 1:

Steel column protected with light weight, dry insulation
material

Assumptions:

Centrically loaded column HEB 300 ($A_s = 14.9 \times 10^3 \text{ mm}^2$; $I_y = 75.8 \text{ mm}^4$), with height $h = 3.50 \text{ m}$.

Steel grade $F_e 360$, hence $\sigma_{y20} = 235 \text{ N/mm}^2$.

Loading during fire: $P_{\theta} = 1700 \text{ kN}$.

The column is protected by sprayed mineral wool in the form of a contour protection of uniform thickness. Insulation thickness $d_1 = 20 \text{ mm}$, thermal conductivity $\lambda_1 = 0.10 \text{ W/m}^\circ\text{C}$; hence: $d_1 / \lambda_1 = 0.02 / 0.10 = 0.2 \text{ m}^2\text{C/W}$.

Step 1

In accordance with common practice in standard fire resistance testing on columns, the buckling length of the column is taken as equal to $0.7 \times$ the structural height. It follows for the slenderness ratio:

$$\lambda = \frac{0.7 h}{i_y} = \frac{0.7 \times 3500}{75.8} = 32.3$$

Hence: buckling stress $\sigma_{cr} = 217 \text{ N/mm}^2$ (ref. Table D-4)
 and buckling load $P_p = 14.9 \times 217 = 3233 \text{ kN}$
 Load multiplier for columns $\kappa = 0.85$

The load level then follows from:

$$\frac{\kappa P_{\theta}}{P_p} = \frac{0.85 \times 1700}{3233} = 0.447 \quad (= \gamma)$$

Step 2

For $\gamma = \frac{\sigma_{y\theta}}{\sigma_{y20}} = 0.447$, Table D-1 gives a critical steel temperature: $\theta_{cr}^{y20} = 517^\circ\text{C}$.

Step 3

For a HEB 300 profile, with contour protection and exposed to fire on four sides, Table D-5 gives a section factor: $F_1/V = 116 \text{ m}^{-1}$.

The steel temperature θ_s can, for $d_1/\lambda_1 = 0.2 \text{ m}^2\text{C/W}$ and $F_1/V = 116 \text{ m}^{-1}$, be interpolated from Table D-3:

$$\text{— after 105 min.: } \theta_s = \frac{577 - 461}{50} \cdot 16 + 461 = 498^\circ\text{C}$$

$$\text{— after 120 min.: } \theta_s = \frac{627 - 508}{50} \cdot 16 + 508 = 546^\circ\text{C}$$

The critical temperature $\theta_{cr} = 517^\circ\text{C}$ will be reached after:

$$t_{fr} = 105 + \frac{517 - 498}{546 - 498} \cdot 15 = 111 \text{ min.}$$

Instead of using the Table D-3, the time t_{fr} can, for values of $d_i/\lambda_i > 0.1$, be approximately calculated with the following equation:

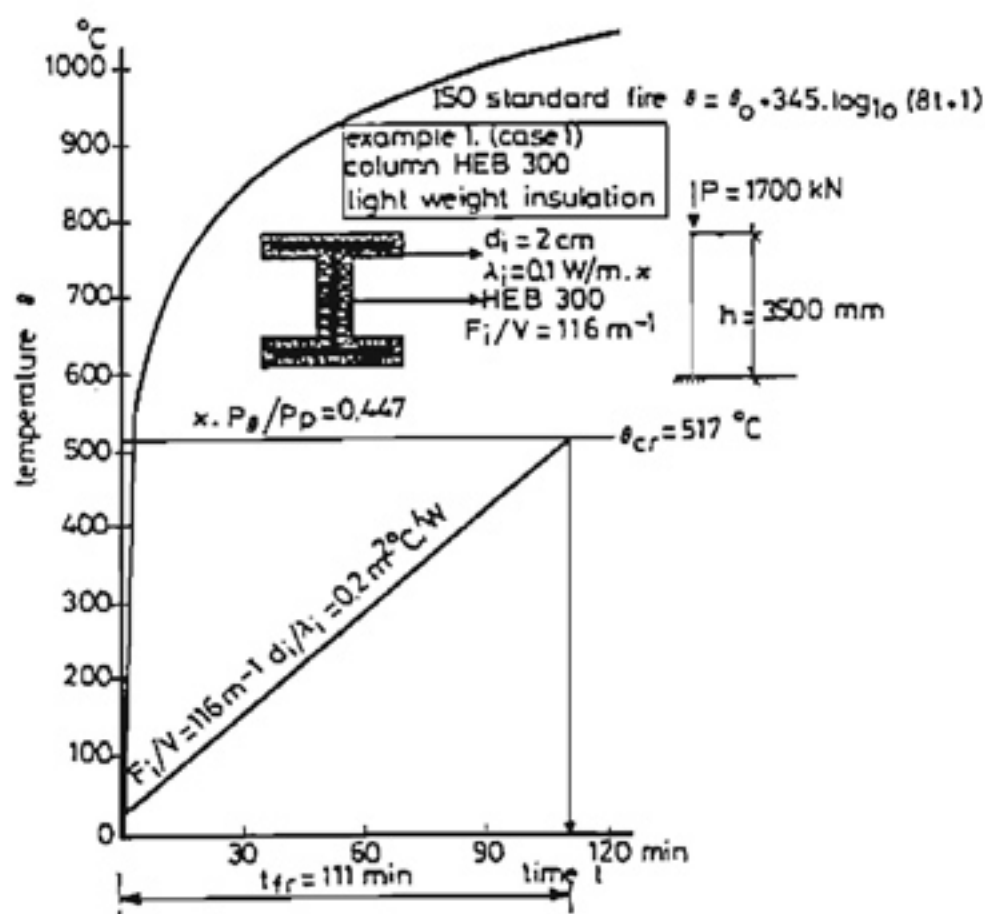
$$t_{fr} = 40 \cdot (\theta_s - 140) \cdot (d_i V / \lambda_i F_i)^{0.77}$$

In the case of Example 1:

$$t_{fr} = 40 \cdot (517 - 140) \cdot (0.2/116)^{0.77} = 112 \text{ min.}$$

Conclusion: The fire resistance of the column amounts to 111 minutes.

Example I is reviewed in Fig. VI.3



Review of Example 1

Fig. VI.3

Example 2:

Steel column protected with heavy weight, moist insulation material

Assumptions:

The same column as in Example 1 is now protected by a box encasement with the following properties:

thickness	: $d_i = 0.02 \text{ m}$
thermal conductivity	: $\lambda_i = 0,2 \text{ W/m}^\circ\text{C}$
specific density	: $\rho_i = 800 \text{ kg/m}^3$
specific heat	: $c_i = 1.7 \text{ kJ/kg}^\circ\text{C}$
moisture content	: $p = 20 \%$ (by weight)

Step 1

$\gamma = 0.447$ (as in Example 1)

Step 2

$\theta_{cr} = 517^\circ\text{C}$ (as in Example 1)

Step 3

HEB 300, protected with a box encasement and exposed to fire on four sides, gives a section factor $F_i/V = 80.5 \text{ m}^{-1}$ (ref.: Table D-5). Heat capacity of the insulation:

$$c_i \cdot \rho_i \cdot F_i \cdot d_i = 1.7 \times 800 \times 1.2 \times 0.02 = 32.65 \text{ kJ/}^\circ\text{C m}$$

Heat capacity of the steel section: (ref. Section III)

$$c_s \cdot \rho_s \cdot V = 0.52 \times 7850 \times 0.0149 = 60.8 \text{ kJ/}^\circ\text{C m}$$

Determining the steel temperature development, the heat capacity of the insulation should be taken into account since:

$$c_s \cdot \rho_s \cdot V = 60.8 < 2 \cdot c_i \cdot \rho_i \cdot F_i \cdot d_i = 65.3$$

Consequently, the section factor F_i/V ($= 80.5 \text{ m}^{-1}$) is to be transferred to $(F_i/V)_{\text{mod}}$ using the following equation:

$$\begin{aligned} F_i/V_{\text{mod}} &= F_i/V \cdot \frac{c_s \cdot \rho_s}{c_s \cdot \rho_s + \frac{c_i \cdot \rho_i \cdot F_i \cdot d_i}{2 \cdot V}} \\ &= \frac{80.5 \times 0.52 \times 7850}{0.52 \times 7850 + \frac{32.65}{2 \times 0.0149}} = 64 \text{ m}^{-1} \end{aligned}$$

Interpolating from Table D-3 for $d_i/\lambda_i = \frac{0.02}{0.20} = 0.1 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$ gives:

$$\text{— after 90 min.: } \theta_s = \frac{489 - 446}{10} \cdot 4 + 446 = 463^\circ\text{C}$$

$$\text{— after 105 min.: } \theta_s = \frac{545 - 500}{10} \cdot 4 + 500 = 518^\circ\text{C}$$

If the effect of moisture in the insulation is not taken into account, the critical steel temperature θ_{cr} ($= 517^\circ\text{C}$) will be reached after:

$$90 + \frac{517 - 463}{518 - 463} \times 15 = 105 \text{ min.}$$

As in Example 1 this time can also be approximated with:

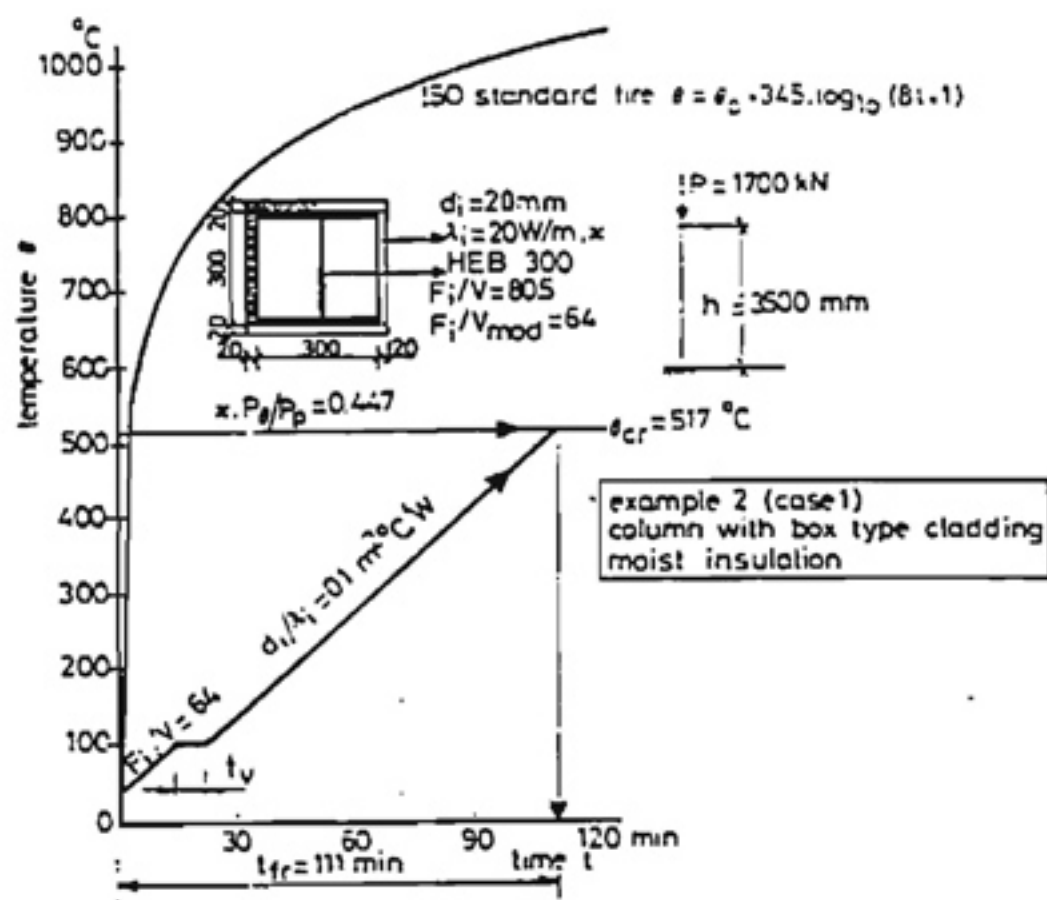
$$40 \cdot (\theta_s - 140) \cdot (d_i V / \lambda_i F_i)^{0.77} = 40 \cdot (517 - 140) \cdot (0.1/64)^{0.77} = 104 \text{ min.}$$

The evaporation of moisture will prolong the time, necessary for the steel column to reach θ_{cr} by about:

$$t_v = \frac{p \cdot \rho_i \cdot d_i^2}{5 \cdot \lambda_i} = \frac{20 \times 800 \times 0.02^2}{5 \times 0.20} = 6 \text{ min.}$$

Conclusion: the fire resistance of the column amounts to $105 + 6 = 111 \text{ min.}$

Example 2 is reviewed in Fig. VI.4



Review of Example 2

Fig. VI.4

VI.3 Design for minimum insulation thickness

Course of calculation:

1. Determine the load level $\kappa \cdot P_\theta/P_p$.
2. Determine the critical steel temperature θ_{cr} .
3. Determine the d_i/λ_i - value necessary for the steel member to reach a temperature $\theta_s = \theta_{cr}$.

Example 3:

Steel beam to be designed for a fire resistance of 90 minutes

Assumptions:

Uniformly loaded, continuous beam IPE 300 ($Z = 628 \times 10^3 \text{ mm}^3$) with a span of $l = 6.00 \text{ m}$. Steel grade $F_e 360$, hence $\sigma_{y20} = 235 \text{ N/mm}^2$. Loading during fire: $P_\theta = 36 \text{ kN/m}$. The heat conduction from the upper flange of the beam to the floor slab above may be neglected.

Required fire resistance: 90 minutes.

Step 1

Plastic moment of IPE 300 at room temperature:

$$M_p = Z \cdot \sigma_{y20} = 628 \times 235 \times 10^{-3} = 147 \text{ kNm}$$

The collapse load P_p at room temperature follows from plastic theory:

$$P_p = 16 \frac{M_p}{l^2} = 16 \frac{147}{6^2} = 65.3 \text{ kN/m}$$

Load multiplier for statically indeterminate beams with two redundancies:

$$\begin{aligned} \kappa &= 0.75 \left(\frac{P_\theta}{P_p} \right) + 0.25 \quad (> 0.4) \\ &= 0.75 \frac{36.0}{65.3} + 0.25 = 0.66 \quad (> 0.4) \end{aligned}$$

The load level then follows from:

$$\frac{\kappa \cdot P_\theta}{P_p} = \frac{0.66 \times 36.0}{65.3} = 0.363 (= \gamma)$$

Step 2

For $\gamma = \frac{\sigma_{y\theta}}{\sigma_{y20}} = 0.363$ Table D-1 gives a critical steel temperature $\theta_{cr} = 558^\circ\text{C}$.

Step 3

For a IPE 300 profile exposed to fire on three sides and protected by hollow encasement, Table D.5 gives a section factor $F_1/V = 139 \text{ m}^{-1}$.

After a standard fire exposure of 90 minutes the steel temperature is: (ref. Table D-3).

for

$$d_1/\lambda_1 = 0.2 \text{ m}^2\cdot\text{C/W} :$$

$$\theta_s = 410 + \frac{521 - 410}{50} \times 39 = 497^\circ\text{C}$$

and for

$$d_1/\lambda_1 = 0.1 \text{ m}^2\cdot\text{C/W} :$$

$$\theta_s = 591 + \frac{705 - 591}{50} \times 39 = 680^\circ\text{C}$$

The critical steel temperature $\theta_{cr} = 558^\circ\text{C}$ corresponds to:

$$d_1/\lambda_1 = 0.10 + \frac{680 - 558}{680 - 497} \times 0.1 = 0.167 \text{ m}^2\cdot\text{C/W}$$

In order to obtain a fire resistance of 90 minutes, the thickness of a mineral wool spray with, say, $\lambda_1 = 0.10 \text{ W/m}^\circ\text{C}$ should be at least:

$$(d_1)_{\min} = 0.167 \times 0.1 = 0.017 \text{ m} = 17 \text{ mm}.$$

Instead of using Table D-3, the necessary value of d_1/λ_1 can - for $0.10 < d_1/\lambda_1 < 0.30 \text{ m}^2\cdot\text{C/W}$ - be approximated with the following equation:

$$d_1/\lambda_1 = 0.0083 \cdot \frac{F_1}{V} \cdot (\tau/(\theta_s - 140))^{1.3}$$

In the present example:

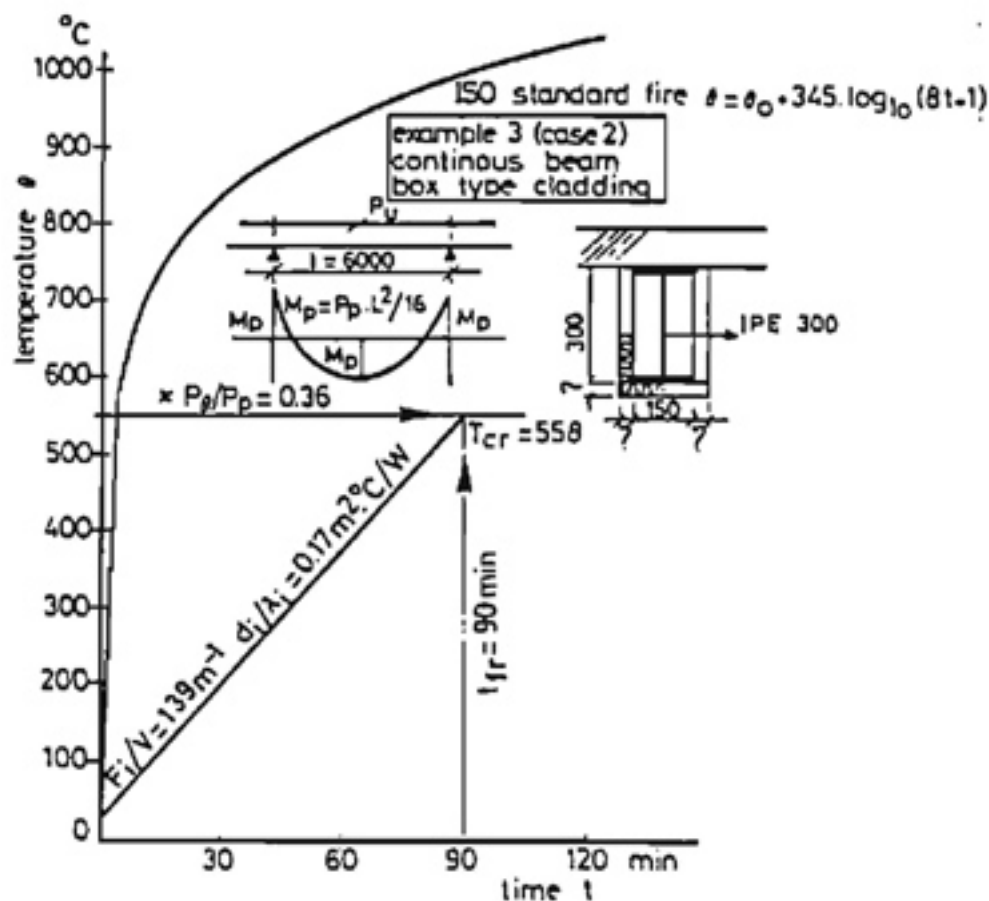
$$d_1/\lambda_1 = 0.0083 \cdot 139 \cdot (90/(558-140))^{1.3} = 0.16 \text{ m}^2\cdot\text{C/W}$$

Hence:

$$(d_1)_{\min} = 0.16 \cdot 0.1 = 0.016 \text{ m} = 16 \text{ mm}.$$

Conclusion: The requirement of 90 minutes fire resistance for the beam will be met if a hollow encasement of mineral wool ($\lambda_1 = 0.10 \text{ W/m}\cdot\text{C}$) with a thickness of at least 17 mm is applied.

Example 3 is reviewed in Fig. VI.5.



Review of example 3

Fig. VI.5

VI.4 Design for admissible load

Course of calculation:

1. Determine the steel temperature θ_s due to standard fire exposure during a period of time equal to the required fire resistance.
2. Determine the yield stress of steel at θ_s .
3. Determine the load, necessary to fail the structural member at a steel temperature of θ_s .

Example 4:

Unprotected solid steel column, with round cross section.

Assumptions

Centrically loaded, solid round steel column with diameter $D = 240$ mm and height $h = 4.00$ m. Steel grade $F_e 510$, hence $\sigma_{y20} = 355 \text{ N/mm}^2$.

Required fire resistance : 60 minutes.

Step 1

The section factor F/V follows from

$$F/V = \frac{\pi D}{4 \times D^2} = \frac{4}{D} = \frac{4}{0.24} = 16.7 \text{ m}^{-1}$$

Table D.2 gives for 60 minutes fire exposure:

$$\theta_s = 603 + \frac{700 - 603}{5} \times 1.7 = 636^\circ\text{C}$$

Step 2

Table D.1 gives for $\theta_s = 636^\circ\text{C}$: $\sigma_{y\theta}/\sigma_{y20} = 0.201$

$$\sigma_{y\theta} = 0.201 \times 355 = 72 \text{ N/mm}^2$$

Step 3

Cross section area of the steel member: $A_s = 45,2 \times 10^3 \text{ mm}^2$
 Radius of inertia : $i = 60 \text{ mm}.$

In accordance with common practice in standard fire resistance testing on columns, the buckling length of the column is taken as equal to $0.7 \times$ the structural height. It follows for the slenderness ratio:

$$\lambda = \frac{0.7 h}{i} = \frac{0.7 \times 4000}{60} = 47$$

Hence: buckling stress at room temperature: $\sigma_{cr20} = 276 \text{ N/mm}^2$.
 (cf. Table D.4)

The buckling stress $\sigma_{cr\theta}$ at $\theta_s = 636^\circ\text{C}$ follows from

$$\sigma_{cr\theta} = \sigma_{cr20} \times \frac{\sigma_{y\theta}}{\sigma_{y20}} = 276 \times \frac{72}{355} = 56 \text{ N/mm}^2$$

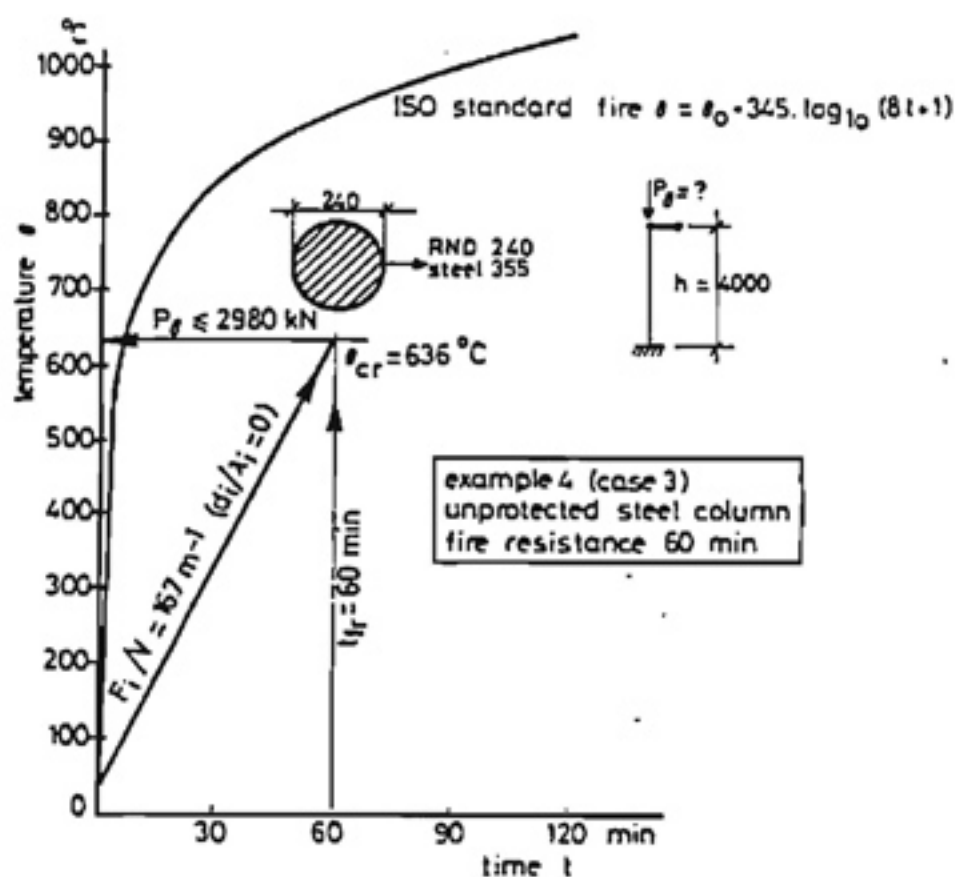
Load multiplier for columns: $\kappa = 0.85$

The collapse load at $\theta_s = 636^\circ\text{C}$ then follows from:

$$(P_P)_{\theta_s=636^\circ\text{C}} = \frac{56 \times 45.2}{0.85} = 2980 \text{ kN}$$

Conclusion: The requirement of 60 minutes fire resistance for the column will be met if the column load is not over 2980 kN.

Example 4 is reviewed in Fig. VI.6).



Review of Example 4

Fig. VI.6

Example 5:

Unprotected heavy section column with I-shaped cross section.

Assumptions:

Centrically loaded, heavy section steel column HD 400 x 400 x 744 with height $h = 4.00 \text{ m}$. Steel grade F_{510} , hence $\sigma_{y20} = 355 \text{ N/mm}^2$.

Required fire resistance: 30 minutes.

Step 1

The section factor F/V follows from:

$$F/V = \frac{2.59}{0.0948} = 27.3 \text{ m}^{-1}$$

Table D.2 gives for 30 minutes fire exposure:

$$\theta_s = 429 + \frac{484 - 429}{5} \times 2.3 = 454^\circ\text{C}$$

Step 2

Table D.1 gives for $\theta_s = 454^\circ\text{C}$: $\sigma_{y\theta}/\sigma_{y20} = 0.561$

$$\sigma_{y\theta} = 0.561 \times 355 = 199 \text{ N/mm}^2$$

Step 3

Cross section area of the steel member: $A_s = 94.8 \times 10^3 \text{ mm}^2$

Radius of inertia : $i_y = 112 \text{ mm}$.

In accordance with common practice in standard fire resistance testing on columns, the buckling length of the column is taken as equal to $0.7 \times$ the structural height. It follows that the slenderness ratio is given by:

$$\lambda = \frac{0.7 h}{i} = \frac{0.7 \times 4000}{112} = 25$$

Hence buckling stress at room temperature: $\sigma_{cr20} = 332 \text{ N/mm}^2$.
(cf. Table D.4).

The buckling stress $\sigma_{cr\theta}$ at $\theta_s = 454^\circ\text{C}$ follows from

$$\sigma_{cr\theta} = \sigma_{cr20} \times \frac{\sigma_{y\theta}}{\sigma_{y20}} = 332 \times \frac{199}{355} = 186 \text{ N/mm}^2$$

Load multiplier for columns: $\kappa = 0.85$

The collapse load at $\theta_s = 454^\circ\text{C}$ then follows from:

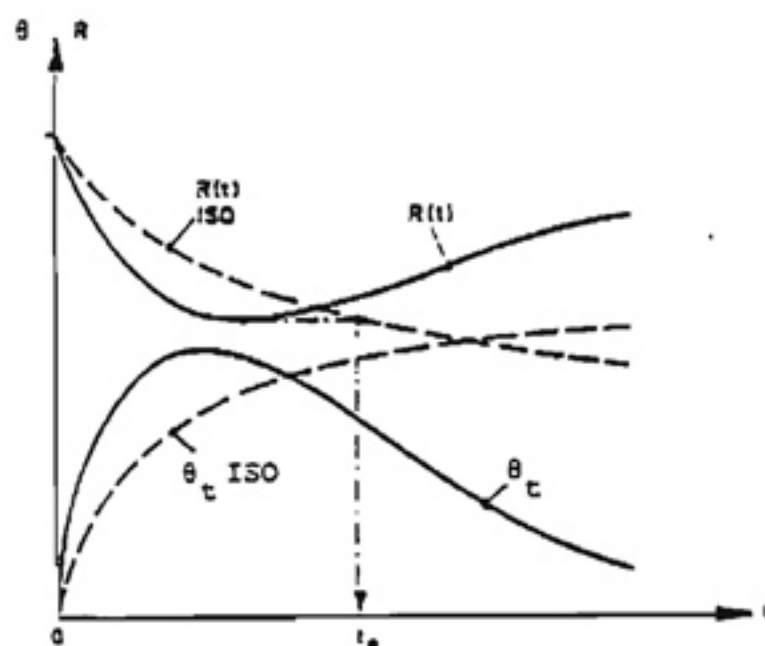
$$(P_p)_{\theta_s=454^\circ\text{C}} = \left(\frac{186 \times 94.8}{0.85} \right) = 20744 \text{ kN}$$

Conclusion: The requirement of 30 minutes fire resistance for the column will be met if the column load is not over 20744 kN. For a load factor of 1.7 and a buckling length of 0.7 h, this load is higher than the allowable design load at room temperature (= 18584 kN).

APPENDIX A Accurate Form of Equivalent Time of Fire Exposure

As stated in section II.3.3, the equivalent time of fire exposure t_e is defined in principal as that length of the heating period in a standard fire which gives the same effect on a structural element with respect to failure as the complete process of the compartment fire.

This concept is illustrated in figure A1. The full-line curves show the variation with time of the gas temperature θ and the load bearing capacity $R(t)$ of the structural element, for a compartment fire exposure, determined by the fire load density q_c , the opening factor $A_w \sqrt{h_w}/A_c$ and the thermal properties of the structures bounding the fire compartment. The dash-line curves give the standard fire temperature-time variation θ_t , ISO, according to equation II-5 and the cor-

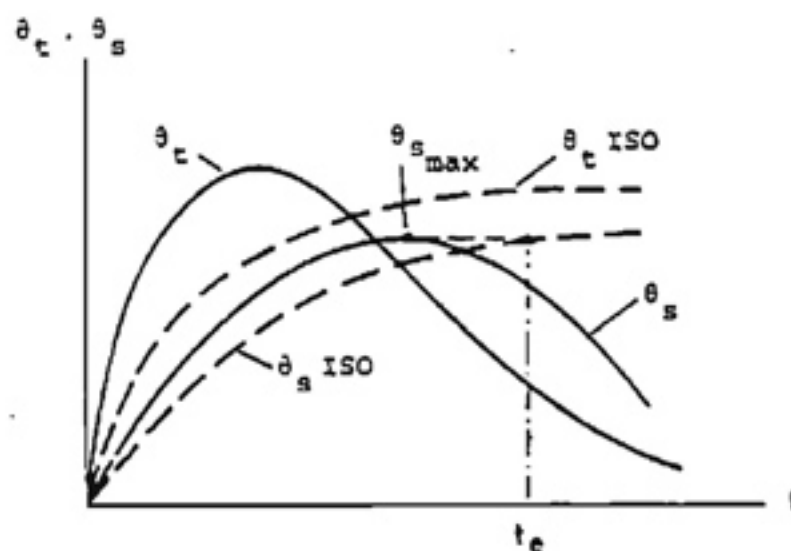


Definition of the equivalent time of fire exposure t_e . Full-line curves for a compartment fire exposure, dash-line curves for an exposure according to the standard fire ISO 834, equation II-5. θ_t is gas temperature and $R(t)$ load bearing capacity

Fig. A1.

responding time curve of the load bearing capacity $R(t)$, ISO. The minimum load bearing capacity of the structural element during the compartment fire is attained during the standard fire at the equivalent time of fire exposure t_e . The minimum load bearing capacity can be defined in three ways: as a critical value of a maximum deflection, a maximum rate of deflection or a maximum temperature.

For steel structures, it can normally be assumed that the minimum load bearing capacity is reached at the time for the maximum steel temperature $\theta_{s \max}$. The definition of the equivalent time of fire exposure t_e according to figure A1 can then be transferred to the definition shown in figure A2.



The equivalent time of fire exposure t_e as defined by the maximum steel temperature $\theta_{s \max}$ of a fire exposed protected structural steel element. Full-line curves for a compartment fire exposure, dash-line curves for an exposure according to the standard fire, ISO 834, equation II-5. θ_t is gas temperature and θ_s steel temperature

Fig. A2.

A direct application of the definition according to figure A1 or A2 means that the equivalent time of fire

exposure t_e depends on parameters influencing the fire process as well as on structural parameters. Figures A3 and A4 illustrate this in more detail for unprotected and protected steel structures, respectively [15], [16], [44].

Figure A3 applies to unprotected steel structures assuming a compartment fire exposure according to figure II.3. The diagrams give the equivalent time of fire exposure t_e as a function of the fire load density q_c , the opening factor of the fire compartment $A_w \sqrt{h_w}/A_c$ and the structural parameter F/V . F is the fire exposed surface and V the volume of the steel structure per unit length. The resultant emissivity ϵ_r has been chosen as 0.5 for the heating according to the standard fire, in conformity with the ECCS-Recommendations, and alternatively as 0.5 and 0.7 for the compartment fire. With respect to practical applications, it can be estimated roughly that $\epsilon_r = 0.7$ represents the conditions for a steel column within a fire compartment and $\epsilon_r = 0.5$ the conditions for a steel beam. For a given value of t_e , the corresponding steel temperature θ_s can be calculated from equation III-8 in chapter III.

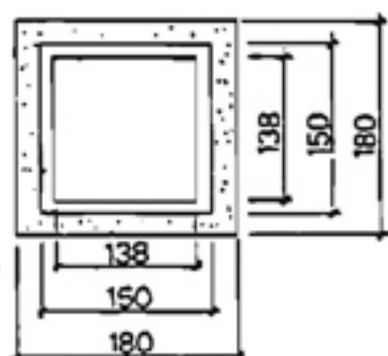
Figure A4 presents design curves giving the equivalent time of fire exposure t_e for protected steel structures as a function of the fire load density q_c , the opening factor of the fire compartment $A_w \sqrt{h_w}/A_c$ and the structural parameter $F_i \lambda_i / V d_i$. F_i is the inside surface area of the protection per unit length of the structure, λ_i the thermal conductivity of the protecting material and d_i the thickness of the protection. For a given value of t_e , the corresponding steel temperature θ_s can be calculated from equation III-11 in chapter III.

By using effective values of the fire load density q_c and the opening factor $A_w \sqrt{h_w}/A_c$ according to equation II-4 and table II.3, the influence of varying

thermal properties of the surrounding structures of the fire compartment can be taken into account in the determination of the equivalent time of fire exposure t_e from the diagrams in figures A3 and A4.

Example 1

A protected steel column has a cross section as shown in figure A5. The thermal conductivity of the insulation $\lambda_i = 0.12 \text{ W} \times \text{m}^{-1} \times ^\circ\text{C}^{-1}$. The column is placed in a fire compartment, type B, with an opening factor $A_w \sqrt{h_w}/A_t = 0.047 \text{ m}^{1/2}$. The fire load density $q_c = 195 \text{ MJ} \times \text{m}^{-2}$, including floor covering.



Cross section of a protected steel column, fire exposed on four sides, Example 1. Dimensions in mm

Fig. A5.

Determine the equivalent time of fire exposure t_e and the corresponding maximum steel temperature θ_{smax} .

A transformation of the real fire load density q_c and the real opening factor $A_w \sqrt{h_w}/A_t$ to an effective fire load density q_{cf} and an effective opening factor $(A_w \sqrt{h_w}/A_t)_f$ according to equation II-4 and table II.3 gives

$$K_f = 0.85 \text{ (fire compartment, type B)}$$

$$q_{cf} = 0.85 \times 195 = 166 \text{ MJ} \times \text{m}^{-2}$$

$$(A_w \sqrt{h_w} / A_{t_f}) = 0.85 \times 0.047 = 0.040 \text{ m}^{1/2}$$

The inside surface area of the protection per unit length

$$F_i = 4 \times 0.15 = 0.60 \text{ m}^2/\text{m}$$

The volume of the steel column per unit length

$$V = 0.15^2 - 0.138^2 = 3.46 \times 10^{-3} \text{ m}^3/\text{m}$$

With the insulation thickness $d_i = 0.015 \text{ m}$ and thermal conductivity $\lambda_i = 0.12 \text{ W} \times \text{m}^{-1} \times ^\circ\text{C}^{-1}$ we have

$$\frac{F_i \lambda_i}{V d_i} = \frac{0.60 \cdot 0.12}{3.46 \cdot 10^{-3} \cdot 0.015} = 1387 \text{ W} \cdot \text{m}^{-3} \cdot ^\circ\text{C}^{-1}$$

For $(A_w \sqrt{h_w} / A_{t_f}) = 0.04 \text{ m}^{1/2}$, $q_{cf} = 166 \text{ MJ} \times \text{m}^{-2}$ and $F_i \lambda_i / V d_i = 1387 \text{ W} \times \text{m}^{-3} \times ^\circ\text{C}^{-1}$, the diagrams in figure A4 give

$$t_e = 0.88 \text{ h} = 53 \text{ min}$$

The corresponding steel temperature θ_s follows from equation III-11 in chapter III

$$\theta_s = 0.025 t_e \left(\frac{F_i \lambda_i}{V d_i} \right)^{0.77} + 140 = 0.025 \cdot 53 \cdot (1387)^{0.77} + 140 = 490^\circ\text{C}$$

A more rough estimation of t_e can be made from equation II-7a. Hence, it follows

$$t_e = 0.067 \frac{166}{(0.040)^{1/2}} = 56 \text{ min}$$

Example 2

The fire resistance t_{fr} of a load bearing steel

structure with a complicated mode of fire behaviour has been determined in a standard fire resistance test according to ISO 834. The structure then collapsed after $t_{fr} = 90$ min.

The structure had a cross section and was provided with an insulation, corresponding to $F_i \lambda_i / V d_i = 300 \text{ W} \times \text{m}^{-3} \times \text{m}^0 \text{C}^{-1}$.

Check whether the structure will collapse or not in a design fire in a compartment, type A, with an opening factor $A_w \sqrt{h_w} / A_c = 0.02 \text{ m}^{1/2}$. The fire load density $q_c = 180 \text{ MJ} \times \text{m}^{-2}$.

From figure A4 we obtain

$$t_e = 1.72 \text{ h} = 103 \text{ min}$$

i.e. $t_e > t_{fr}$, which means that the structure will not survive in the compartment fire situation.

Equation III-11 in chapter III gives for the corresponding steel temperature

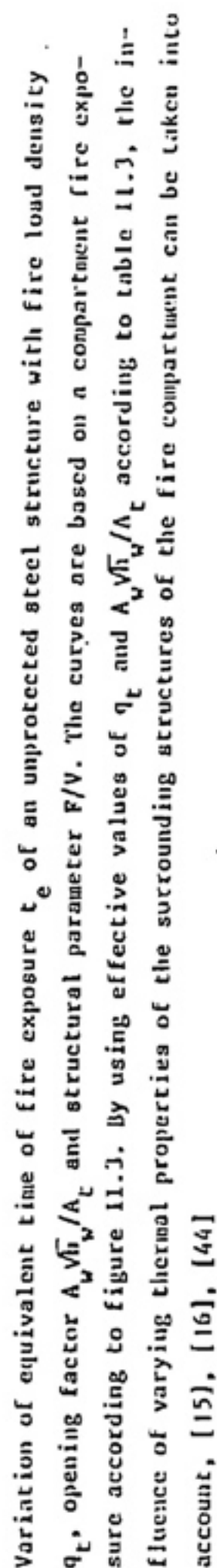
$$\theta_s = 0.025 t_e \left(\frac{F_i \lambda_i}{V d_i} \right)^{0.77} + 140 = 0.025 \cdot 103 \cdot (300)^{0.77} + 140 = 350^\circ \text{C}$$

An estimation of t_e from the formula according to equation II-7a gives

$$t_e = 0.067 \frac{180}{(0.02)^1} = 85 \text{ min}$$

i.e. a value which is about 17% on the unsafe side compared with the more accurate value determined from figure A4.

The results obtained in the two examples confirm the conclusions drawn from figure II.6.



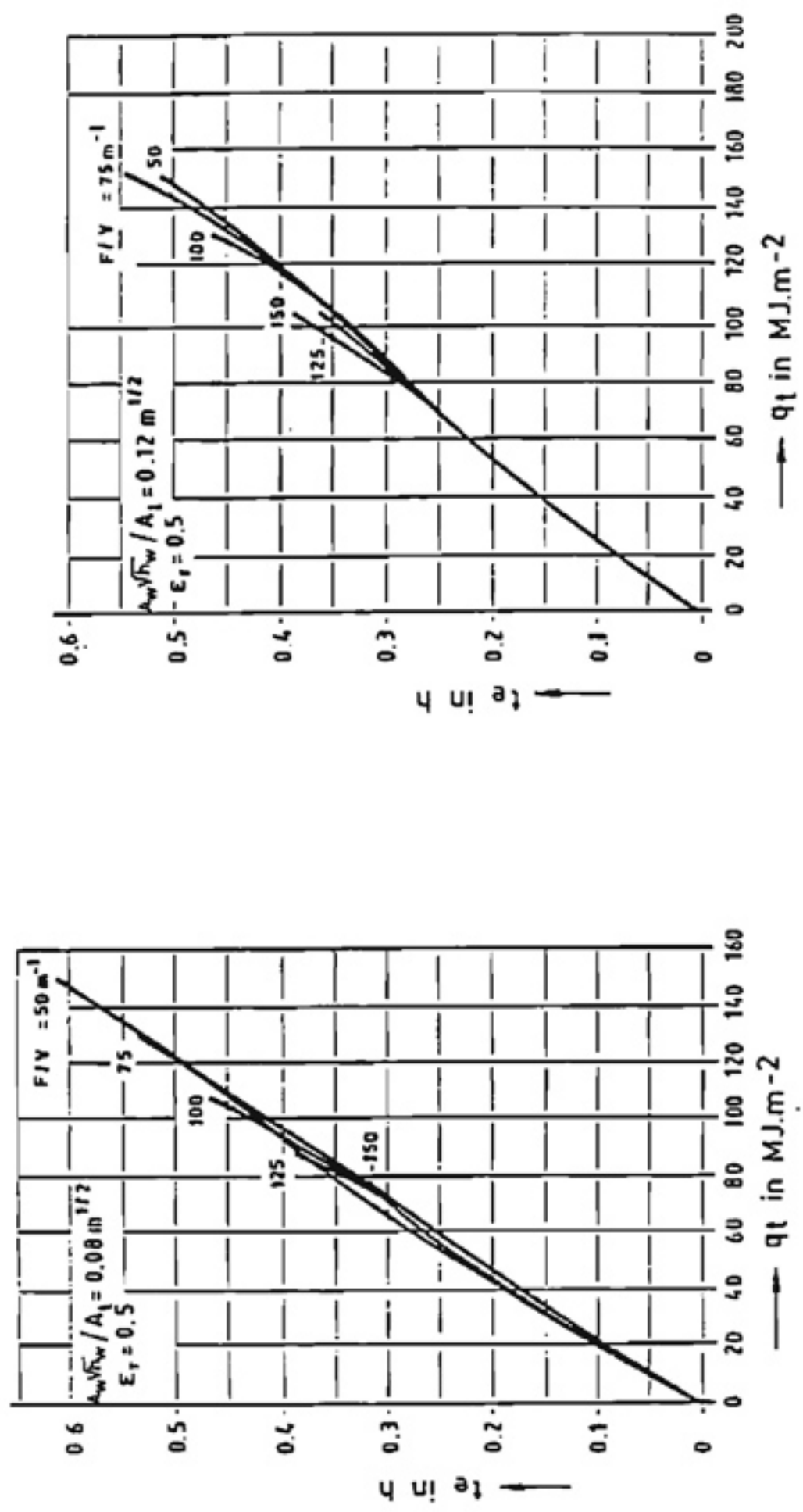


Fig. A3. Cont.

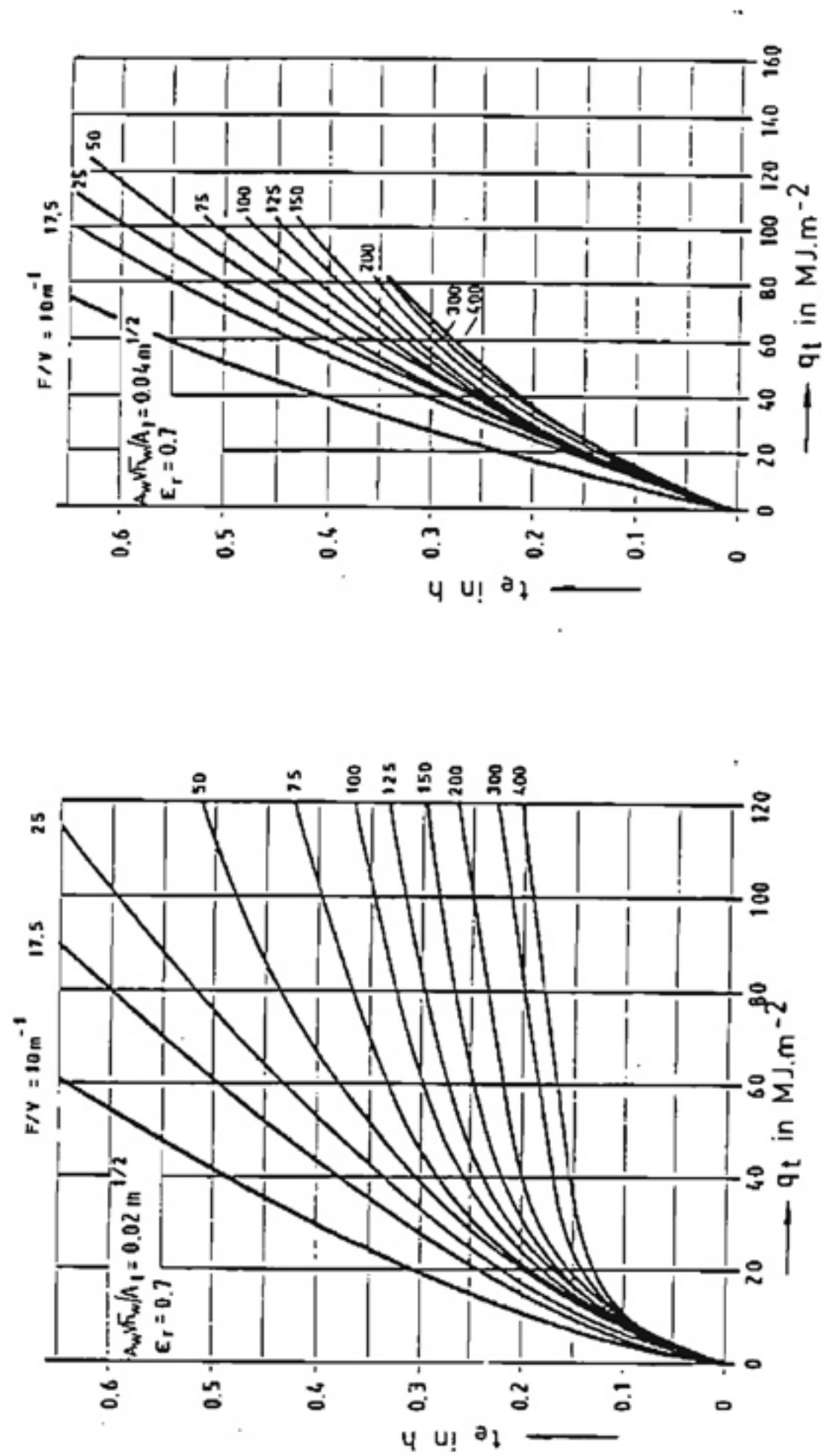


Fig. A3. Cont.

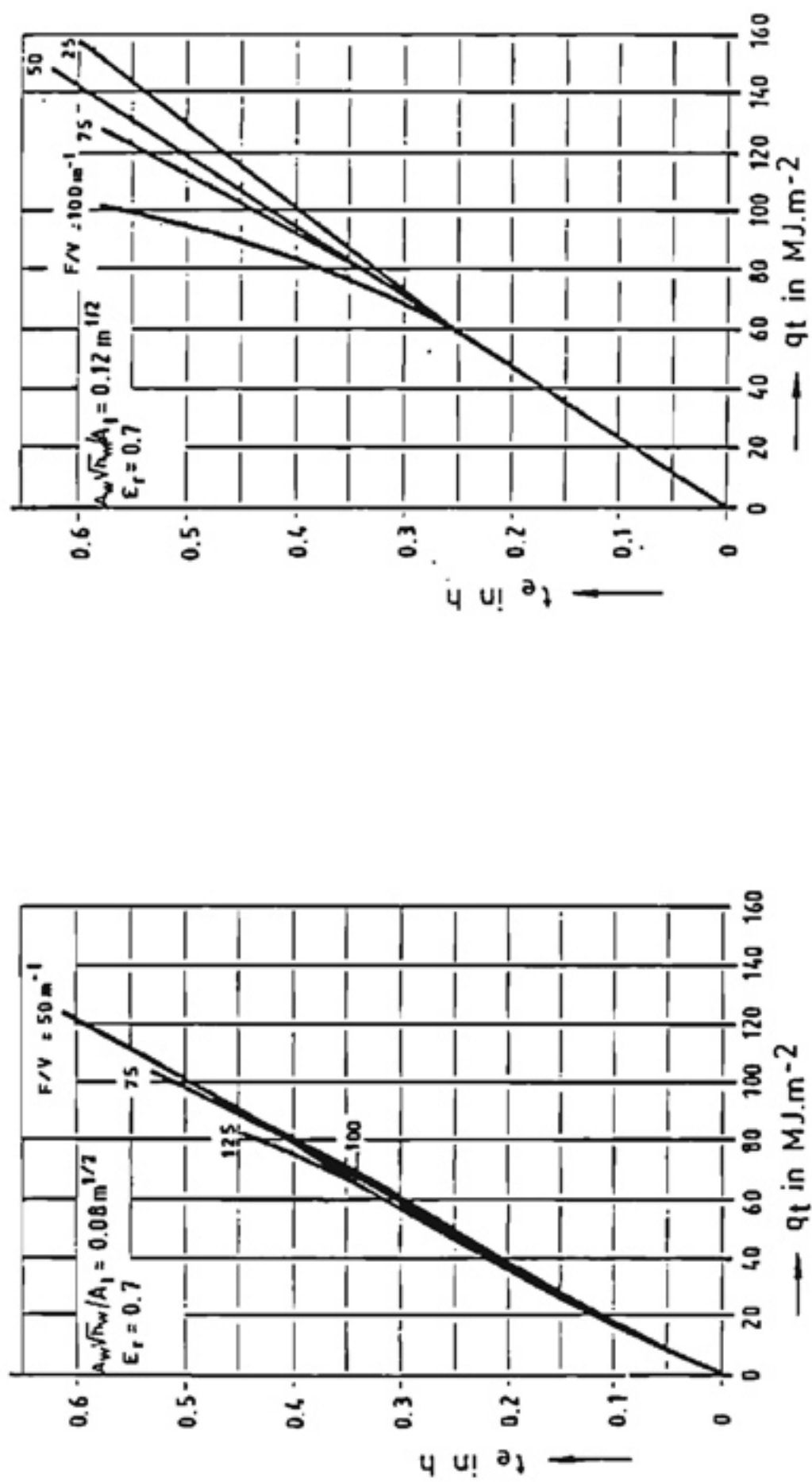
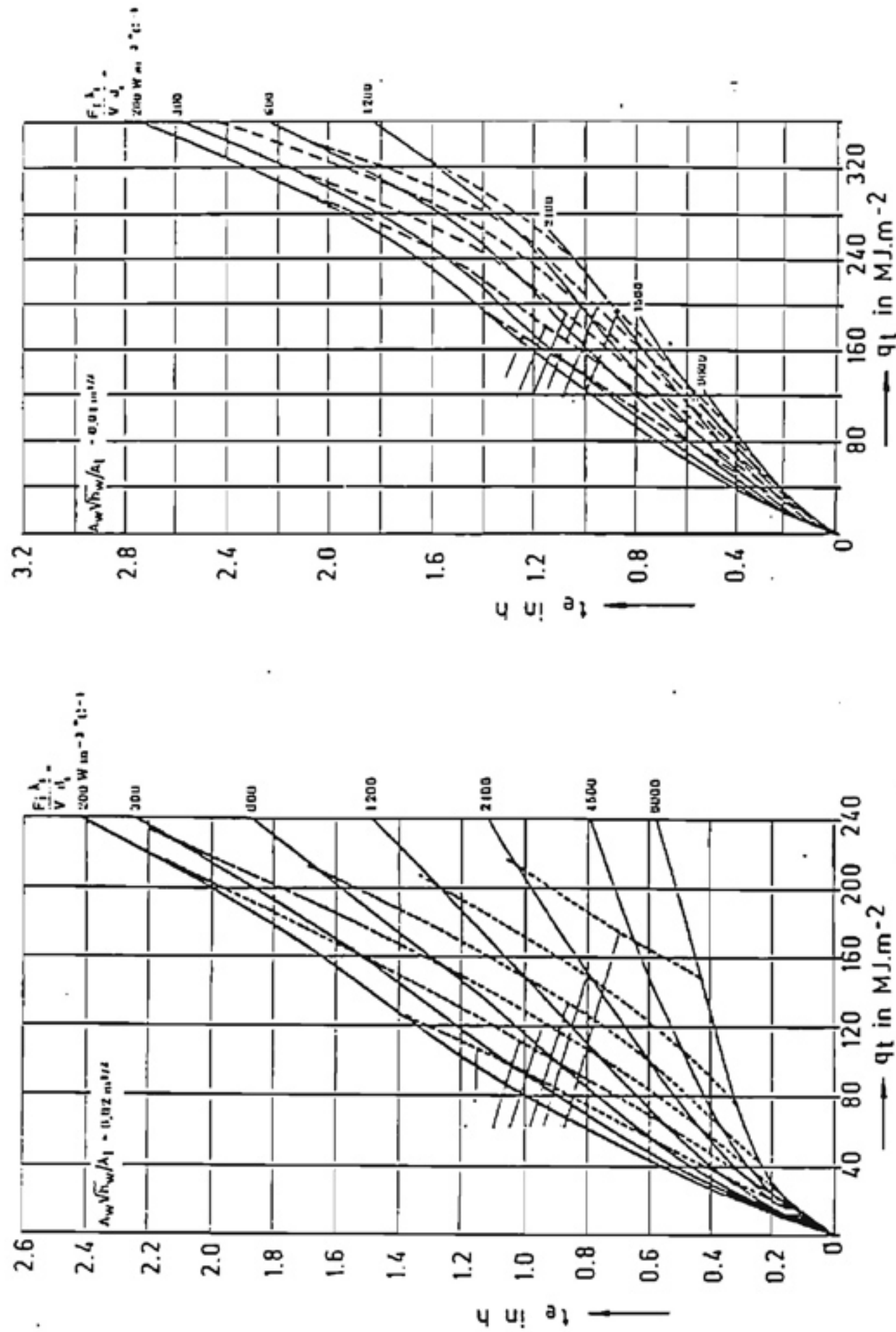


Fig. A3. Cont.



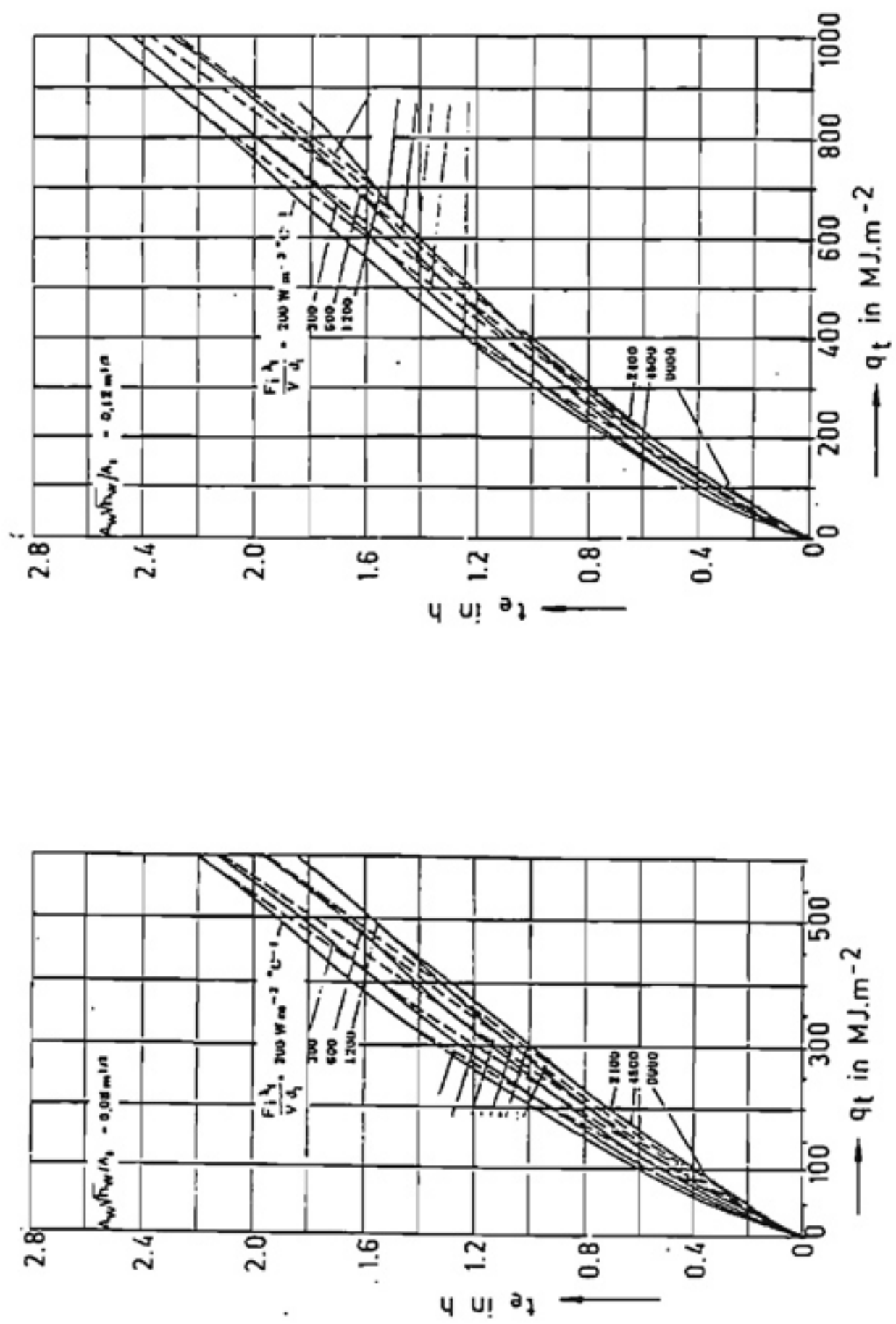


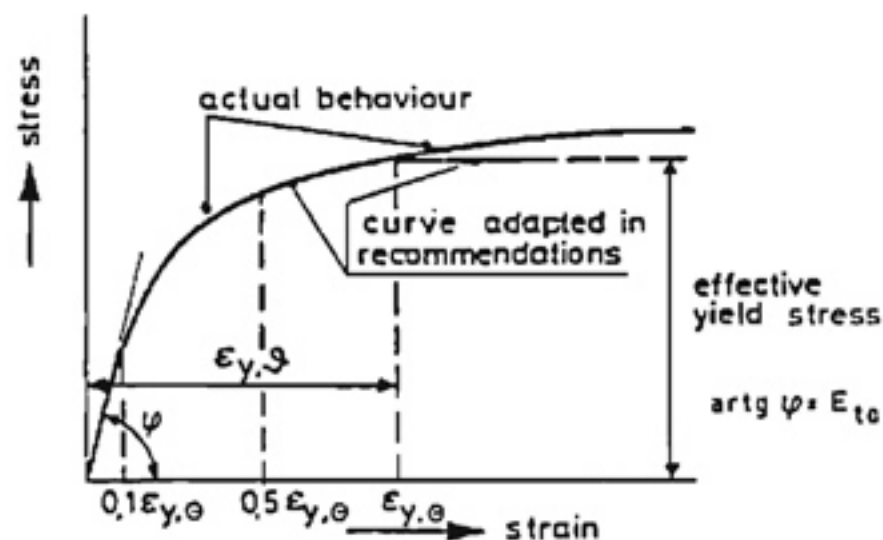
Fig. A1. Cont.

Appendix B: Strength and deformation properties of structural steel of various grades at elevated temperature

As explained in Section IV.2, the investigation of the strength and deformation properties of steel at elevated temperature has concentrated on grade Fe 360 (nominal yield stress 235 N/mm²). In practice, other grades of structural steel may also be used, more particularly those specified in Euronorm 25, namely, Fe 310, Fe 430 and Fe 510.

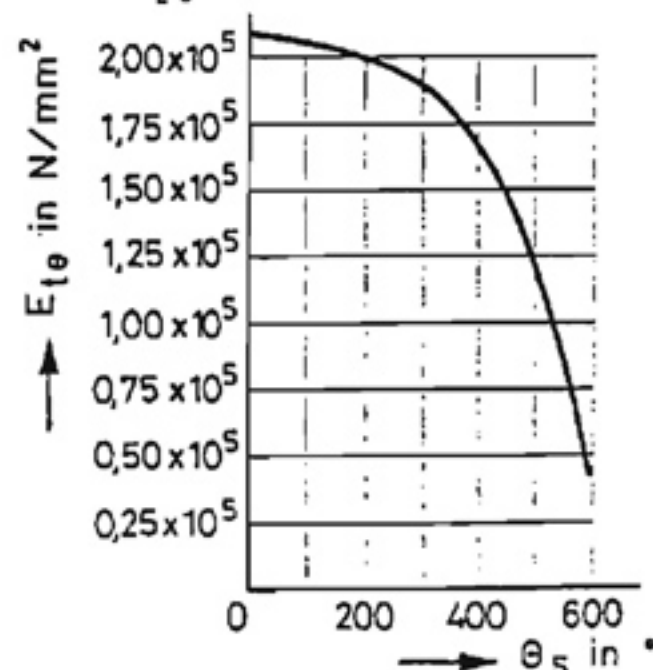
The strength and deformation properties of these grades under fire conditions have likewise been derived, on the basis of the following considerations:

- For all steel grades with properties between Fe 310 and Fe 510 the same relation between steel temperature θ_s and effective yield stress, $\sigma_{y\theta}$, expressed relative to the value at room temperature ($= \sigma_{y\theta}/\sigma_{y20}$), is valid. See Fig. IV.3
- For all steel grades with properties between Fe 310 and Fe 510 the modulus of elasticity $E_{t\theta}$ at the origin is independent of the steel grade. For an explanation of the meaning of $E_{t\theta}$ see Fig. B.1. The relation adopted for $E_{t\theta}$ as a function of temperature is shown in Fig. B.2.



Stress-strain relationship at elevated temperature (schematic)

Fig. B.1



$E_{t\theta}$ as a function of temperature

Fig. B.2

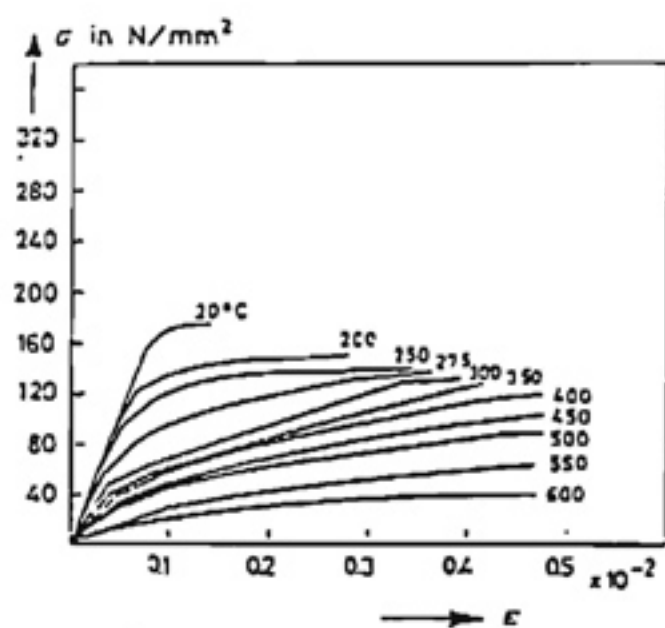
With reference to these two basic aspects, the elevated temperature properties of steel with a grade between Fe 310 and Fe 510 can be derived from those of grade Fe 360. The stress-strain diagrams for Fe 320, Fe 430 and Fe 510 determined in this way are given in Fig. B.3. For the sake of completeness stress-strain diagrams for Fe 360 have also been included. See also Fig. IV.2. A numerical presentation of these diagrams is given in Tables B.1a to B.1d. For the meaning of "effective yield strain" ($= \epsilon_{y\theta}$) see Fig. B.1. The yield stress adopted in calculations at room temperature corresponds to the minimum guaranteed values in accordance with Euronorm 25-72.

As an example of how the tables should be used, the relevant values for the stress-strain diagram of Fe 510 at 400 °C will now be given (see Table B.1d):

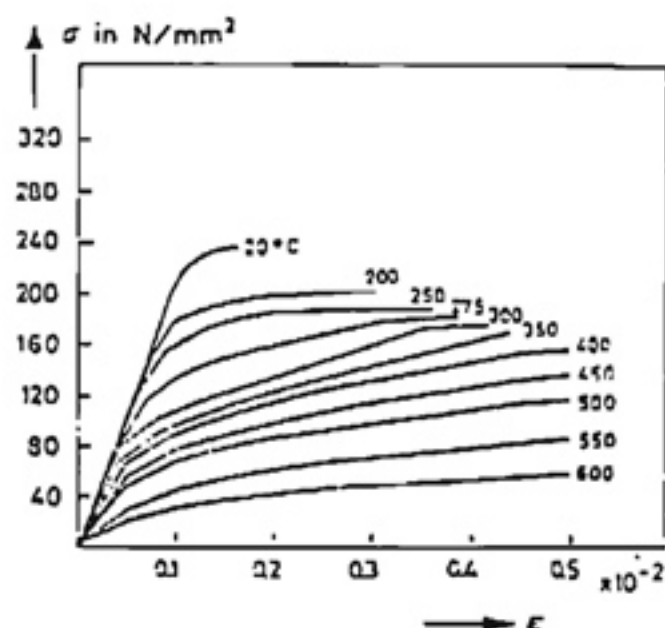
- effective yield stress $\sigma_{y\theta} = 234.6 \text{ N/mm}^2$
- effective yield strain $\epsilon_{y\theta} = 0.545 \text{ per cent (5.45 ‰)}$

The above data determine the point at which the stress-strain diagram at 400 °C becomes horizontal for strains larger than $\epsilon_{y\theta}$. For strains $< \epsilon_{y\theta}$ the associated stress can be read from the table, e.g., for $0.5 \epsilon_{y\theta} = 0.2775 \text{ per cent (2.775 ‰)}$ we find $\sigma = 198.3 \text{ N/mm}^2$.

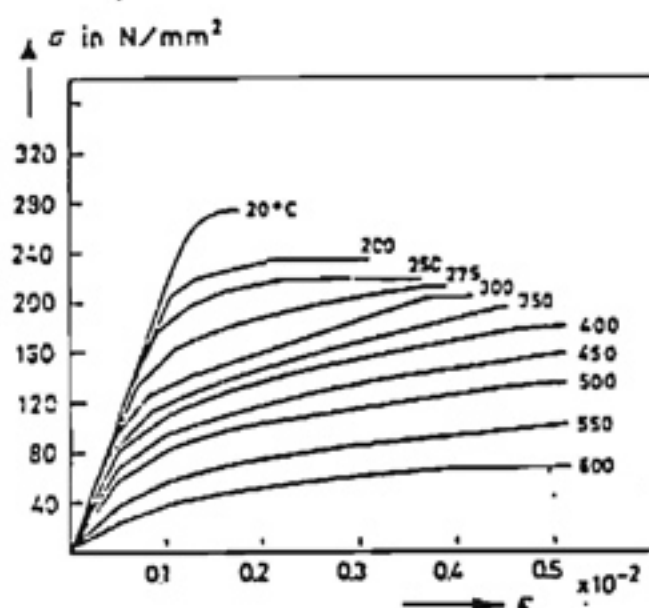
For a tabulated review of the yield stress and Youngs modulus as function of the steel temperature it is referred to Appendix D., Table D.1^a.



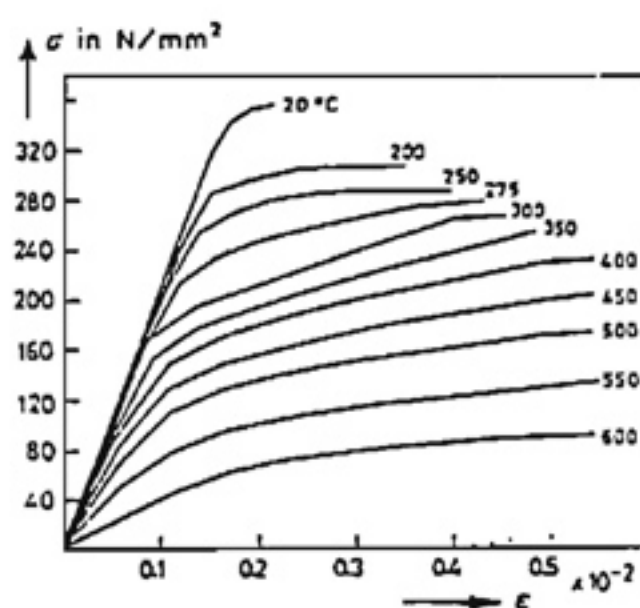
(a) Fe 310



(b) Fe 360



(c) Fe 430



(d) Fe 510

Stress-strain diagrams at elevated temperature, valid for various grades of steel (Note: only the portion of each curve up to the effective yield strain $\epsilon_{y\theta}$ has been drawn, i.e., the yield range is not shown).

Fig. B.3

Fe 310	Stress N/mm^2											
	20	200	250	275	300	350	400	450	500	550	600	
σ_y [°C] ϵ_{y0} [°/°] strain ϵ [°/°]	0.132	0.204	0.345	0.365	0.306	0.417	0.470	0.470	0.478	0.475	0.456	
0.1 ϵ_{y0}	25.4	56.0	62.7	56.5	49.4	43.7	42.4	34.0	30.4	14.6	13.5	
0.2 ϵ_{y0}	53.1	105.5	103.5	83.3	62.3	55.6	56.7	40.4	43.6	27.0	21.3	
0.3 ϵ_{y0}	80.2	129.5	121.0	97.1	73.0	66.6	60.9	57.0	53.3	35.5	26.1	
0.4 ϵ_{y0}	107.4	137.4	130.1	107.1	83.2	77.3	79.0	66.3	61.0	41.0	30.1	
0.5 ϵ_{y0}	133.8	142.6	134.4	113.6	93.6	86.7	88.2	74.8	66.6	46.5	33.2	
0.6 ϵ_{y0}	154.6	146.1	136.4	119.1	103.7	96.0	95.3	82.0	72.1	51.1	35.9	
0.7 ϵ_{y0}	165.9	147.5	137.3	124.9	113.2	104.3	101.2	87.7	76.9	54.6	30.7	
0.8 ϵ_{y0}	171.2	148.4	130.2	130.1	122.3	111.9	100.9	93.5	81.8	57.6	41.1	
0.9 ϵ_{y0}	174.5	149.3	139.1	132.8	120.6	119.4	115.4	99.1	87.4	61.4	41.9	
1.0 ϵ_{y0}	175.0	150.2	140.1	135.6	131.3	127.0	118.2	102.9	89.2	65.2	42.0	

(a) Grade Fe 310
Table B. 1: Stress strain relationships at elevated temperatures for various grades of steel
(ϵ_{y0} is the strain at which the effective yield stress is reached, see Fig. B.1).

Fe 360 σ_s [°C] ϵ_{y0} [‰] strain ϵ [‰]	Stress N/mm ²										
	20	200	250	275	300	350	400	450	500	550	600
0.160	0.160	0.310	0.370	0.390	0.410	0.440	0.500	0.500	0.500	0.500	0.500
0.1 ϵ_{y0}	33.0	62.0	72.0	75.0	78.0	73.0	70.0	50.0	50.0	30.0	10.0
0.2 ϵ_{y0}	66.0	124.0	134.0	120.0	102.0	93.0	90.0	78.0	60.0	46.0	32.0
0.3 ϵ_{y0}	99.0	171.0	164.0	139.0	115.0	105.0	104.0	89.0	79.0	56.0	39.0
0.4 ϵ_{y0}	132.0	186.0	177.0	152.0	126.0	117.0	115.0	98.0	88.0	62.0	44.0
0.5 ϵ_{y0}	165.0	193.0	183.0	160.0	137.0	127.0	125.0	107.0	94.0	68.0	48.0
0.6 ϵ_{y0}	197.0	198.0	186.0	166.0	140.0	137.0	133.0	115.0	100.0	73.0	51.0
0.7 ϵ_{y0}	220.0	200.0	187.0	172.0	150.0	146.0	139.0	121.0	105.0	77.0	54.0
0.8 ϵ_{y0}	229.0	201.0	188.0	178.0	160.0	154.0	147.0	127.0	110.0	80.0	57.0
0.9 ϵ_{y0}	234.0	202.0	189.0	181.0	175.0	162.0	154.0	133.0	116.0	84.0	58.0
1.0 ϵ_{y0}	235.0	203.0	190.0	184.0	178.0	170.0	157.0	137.0	110.0	80.0	59.0

(b) Grade Fe 360

Table B.1:
(Continued)

Fe 430 σ_u [MPa] ϵ_{yk} [%] σ_u [MPa] ϵ_{yk} [%]	Stress σ / mm^2										
	20	200	250	275	300	350	400	450	500	550	600
σ_u [MPa] ϵ_{yk} [%]	0.170	0.327	0.387	0.407	0.426	0.456	0.515	0.515	0.515	0.517	0.530
0.1 ϵ_{yk}	37.7	65.0	76.5	80.5	84.6	82.1	80.8	68.9	58.5	30.1	19.4
0.2 ϵ_{yk}	74.6	132.2	147.1	140.1	129.1	119.1	113.6	98.2	84.7	57.9	36.8
0.3 ϵ_{yk}	111.6	190.5	190.4	167.3	143.2	131.1	127.3	109.9	96.3	69.3	47.4
0.4 ϵ_{yk}	140.5	218.4	207.9	181.5	154.6	143.2	139.0	119.2	105.0	76.2	53.2
0.5 ϵ_{yk}	165.5	226.7	215.4	190.8	166.0	154.0	149.5	128.4	112.4	82.2	57.7
0.6 ϵ_{yk}	222.4	232.3	210.9	197.3	177.4	164.3	158.0	136.9	118.5	87.6	61.1
0.7 ϵ_{yk}	253.1	235.0	220.2	203.5	188.0	173.8	164.3	143.2	123.0	91.9	64.2
0.8 ϵ_{yk}	267.7	236.1	221.2	209.8	198.4	182.1	172.4	149.4	128.9	95.0	67.5
0.9 ϵ_{yk}	273.6	237.1	222.3	213.2	205.9	190.4	179.7	155.6	135.0	99.1	68.7
1.0 ϵ_{yk}	275.0	238.2	223.3	216.3	209.1	198.7	182.9	159.7	137.2	103.2	69.8

Table B.1:
(Continued)

(c) Grade Fe 430

Fe 510 strain ϵ [‰] $\frac{\sigma}{\sigma_c}$ $\frac{\sigma}{\sigma_c}$ [‰]	Stress N/mm ²											
	20	200	250	275	300	350	400	450	500	550	600	
0.217	0.217	0.362	0.421	0.440	0.459	0.487	0.545	0.544	0.544	0.550	0.509	
0.1 ϵ_{y0}	46.1	73.2	83.3	85.9	88.3	90.3	93.9	83.0	69.5	49.2	21.4	
0.2 ϵ_{y0}	91.7	145.9	165.5	170.6	173.5	164.1	156.1	134.7	114.8	79.6	43.0	
0.3 ϵ_{y0}	136.7	218.9	235.7	219.6	190.1	182.5	173.4	151.1	130.2	95.2	61.4	
0.4 ϵ_{y0}	181.4	276.1	267.1	239.3	211.7	195.5	106.9	161.6	141.1	104.3	71.0	
0.5 ϵ_{y0}	226.2	293.0	279.3	252.1	223.9	207.0	190.3	171.3	149.1	110.7	76.0	
0.6 ϵ_{y0}	271.0	300.5	284.6	259.0	236.2	218.0	208.0	100.5	155.5	116.8	81.2	
0.7 ϵ_{y0}	314.9	304.0	286.6	266.5	247.9	229.2	215.0	187.7	161.3	121.6	84.7	
0.8 ϵ_{y0}	343.5	306.3	287.6	273.4	259.2	238.3	223.1	194.1	166.6	125.1	88.4	
0.9 ϵ_{y0}	352.7	307.4	288.0	277.5	267.7	247.1	231.1	200.7	173.1	129.2	90.2	
1.0 ϵ_{y0}	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 B.1:
(Continued)

Appendix C: Behaviour of steel elements in structural systems

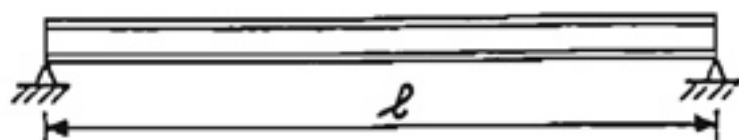
C.1 Introduction

The European Recommendations for the Fire Safety of Steel Structures concentrate on the behaviour of single load bearing steel elements and structural assemblies under standard fire conditions. This approach is fully in line with standard fire resistance testing procedures, commonly accepted today. In a level 1 design i.e. a simple classification system of components exposed to the standard fire, the representation of the complex structural model by such a simplified model is acceptable, because the level of required fire resistance generally includes a large safety margin and the classification system is supported by many years of experience. However, in more advanced levels of design based upon non-standard or compartment fire conditions, the analysis based on structural components only, becomes questionable because the uncertainty of the structural model may become too large in comparison with the fire development and heat-flow models. For this reason, this Appendix provides some background information on the behaviour of fire exposed steel elements as a part of a complete structural system. In addition, some simple design rules, implicitly accounting for the statical system involved, are presented.

C.2 Axial restraint of beams and columns

In the discussion of the behaviour of the continuous beam on three supports as shown in Fig. IV.8 it was noted that its collapse behaviour is not affected by thermal expansion. However, a situation in which the effect of restrained expansion must be reckoned with - even though the temperature distribution in the beam is uniform - is shown in Fig. C.1. The beam in this case is conceived as supported on two hinge-type supports which do not allow horizontal movement. If the beam could expand freely, its elongation would be :

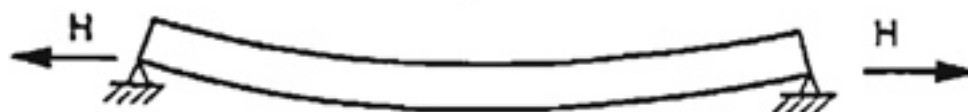
$$\Delta l = \alpha_t \Delta \theta_s \cdot l.$$



a) situation at room temperature ($\theta_s = 20^\circ\text{C}$)



b) situation in an initial stage of the fire ($\theta_s = 20 + (\Delta\theta_s)_1$)



c) situation in a later stage of the fire ($\theta_s = 20 + (\Delta\theta_s)_2$)

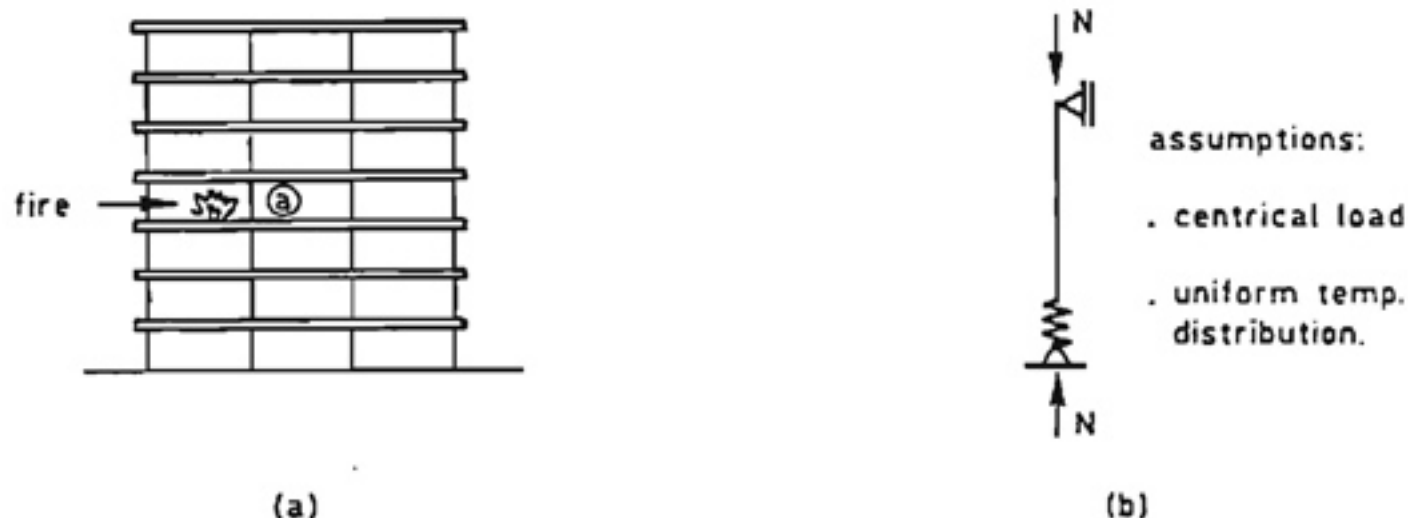
Effect of restrained thermal expansion

Fig. C.1

In practice thermal expansion is prevented by the supports, so that a horizontal reaction H will develop at each of them. The forces H act at the bottom flange and are at first directed inwards. As has already been mentioned in Section IV.3, forces of considerable magnitude can be expected in such cases. It must therefore be checked whether the bearings can indeed resist the horizontal forces. In general, the effect envisaged here occurs at relatively low steel temperatures, at which the stiffness of the steel beam has not yet diminished appreciably. It should be noted that the assumption of complete restraint of thermal expansion is often not realistic. This, too, has already been pointed out in Section IV.3.

If the bearings are assumed indeed to be sufficiently strong to resist the reactions, so that they provide sufficient restraint against expansion, the behaviour pattern will change considerably with rising temperature of the steel, because due to the decrease in the stiffness of the steel, deflection of the beam will greatly increase. As a result, the beam will be, as it were, suspended from its supports. In other words: the initial compressive force in the beam "changes its sign", i.e., becomes a tensile force (so-called membrane force). The effect of this upon the critical temperature will in general be favourable, a fact which can be advantageously utilized in the case, for example, of steel roof sheeting riveted onto roof beams or purlins.

Columns may also be subjected to axial restraint under fire conditions. A situation in which such a phenomenon must be expected is shown in Fig. C.2^a. The relevant mechanical model is given in Fig. C.2^b, in which the spring represents the restraint due to adjacent structural members.

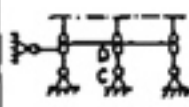
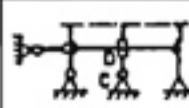
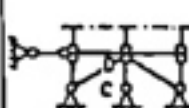


Partially restrained expansion of column "a"

Fig. C.2

It is presumed that a local fire outbreak occurs near column "a", forming part of a statically indeterminate building frame. The temperature of the steel rises, so that the column strives to expand, but is partly prevented from doing this by the rest of the building, since this does not undergo a rise in temperature. Therefore the load on the column will increase, with the result that the stability of the column will in principle be adversely affected. This aspect could be a point of criticism of the basic model, dealt with in Section IV.7. Therefore it will be discussed here in some detail.

Even without a further discussion of the theoretical background it will be evident that the critical temperature is in principle dependent on the slenderness ratio (λ), the initial stress (σ_1) and the degree of restraint imposed upon thermal expansion. The last-mentioned parameter will be determined by the dimensions of the column itself and also by the stiffness of the surrounding structure. A conservative assumption in calculating this stiffness is that the surrounding structure does not rise at all in temperature and that it behaves in a linearly elastic manner. In the tentative calculations reported in [2], a building comprising several storeys was considered. The structural supporting system of each storey comprises identical two-bay portal frames, the centre column of which is a rocker, i.e., hinged at top and bottom. Only the centre column of the ground-floor portal frame is assumed to undergo a rise in temperature. The degree of axial restraint can be varied by varying the number of storeys in the model. For the same purpose, various versions of the frame were included in the investigations, as shown schematically in Table C.1. Some of the results of the calculations are also given in this Table. It is noted that the critical temperatures presented, are calculated using the stress-strain relationship of Fig. IV.2, adopting the concept of fictitious initial deflections. Therefore the results cannot be compared directly with the critical temperatures calculated using the buckling curves presented in Fig. IV.11. Since the aim of the present discussion is relative rather than absolute, this factor does not affect the conclusions to be drawn.

structural system	number of storeys	column C.D.1				
		profile slenderness ratio	mean stress in N/mm^2	critical temperature (free expansion) in $^{\circ}C$	critical temperature (axial restraint) in $^{\circ}C$	effect of axial restraint on the critical temperature of column. C.D. in $^{\circ}C$.
	1	HE 120 B $\lambda = 80$	70.6	410 $^{\circ}C$	370 $^{\circ}C$	40 $^{\circ}C$
	5	HE 240 B $\lambda = 40$	116.0	410 $^{\circ}C$	400 $^{\circ}C$	10 $^{\circ}C$
	1	HE 120 B $\lambda = 80$	62.1	460 $^{\circ}C$	420 $^{\circ}C$	40 $^{\circ}C$
	5	HE 240 B $\lambda = 40$	102.5	460 $^{\circ}C$	432 $^{\circ}C$	28 $^{\circ}C$
	1	HE 120 B $\lambda = 80$	70.6	410 $^{\circ}C$	270 $^{\circ}C$	140 $^{\circ}C$
	5	HE 240 B $\lambda = 40$	116.0	410 $^{\circ}C$	320 $^{\circ}C$	90 $^{\circ}C$

The effect of (partially) restrained thermal expansion on the critical temperature of centrally loaded columns.

Table C.1

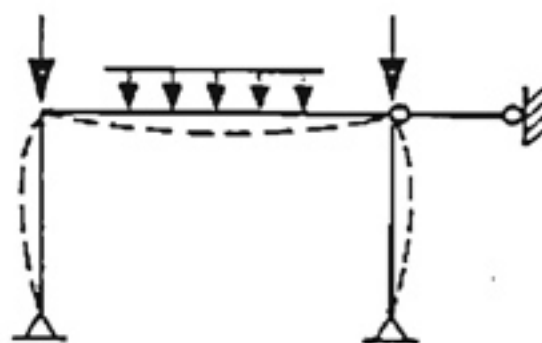
It appears from the Table that if thermal expansion is restrained by flexurally loaded structural steel members (beams) without composite action, the effect of this on the critical temperature is negligible (maximum difference in relation to the basic case is about 40 $^{\circ}C$). If thermal expansion is restrained by members in which a normal force is developed (e.g. members in a wind bracing), the decline in critical temperature will be more severe. According to Table C.1 this may be as much as 100 - 150 $^{\circ}C$. However, the assumption that only the column is heated, while the adjacent bracing is not affected by the fire, is not realistic. In practice, all members in the vicinity of the fire will rise in temperature and consequently the above mentioned temperature difference of 100 - 150 $^{\circ}C$ is an overestimation. A decidedly serious situation - not included in the table - arises if thermal expansion is prevented by dividing walls or partitions. Because of diaphragm action, these components are relatively very stiff. It is to be noted, however, that a design in which the partitions develop a stiffening action of

the kind mentioned here, will be unsatisfactory for reasons other than fire safety as well. For example, cracking of the partitions under the influence of service conditions of the building - such as deformation of the steel frame or unequal settlement of the subsoil - are cases in point. The best advice would appear to be that the partitions should be so designed as to allow some deformation of the frame without loading the partitions. In general, this will also ensure that the thermal expansion of the frame will not be restrained, at least not by the partitions in the building.

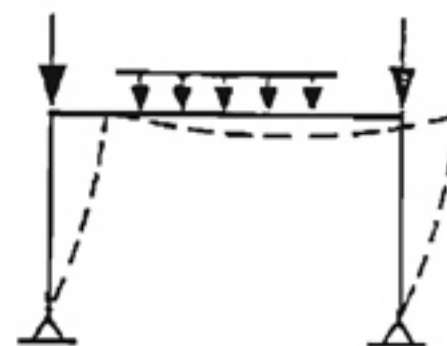
The conclusion therefore is that in general axial restraint may be omitted. It should be recalled also that in the standard fire resistance test procedure the loading is normally kept constant.

C.3 Eccentrically loaded columns

Eccentrically loaded columns occur in portal frame structures. Examples are shown in Fig. C.3.



(a) braced frame



(b) unbraced frame

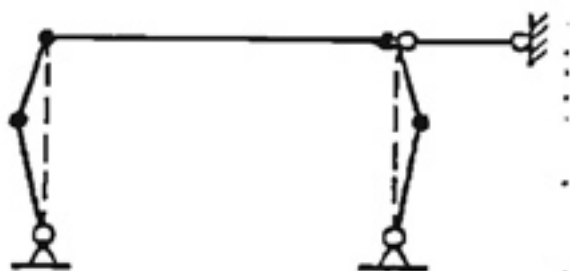
Examples of eccentrically loaded columns

Fig. C.3

In the structure illustrated in Fig. C.3^a the horizontal forces are transmitted to the subsoil via special arrangements, e.g., a rigid core or wind bracing. For this reason the

term "braced frame" is employed. In the structural analysis it is assumed that the joints undergo no displacement in relation to one another; more particularly, there is no "sideways". A characteristic collapse mode is shown in Fig. C.4^a.

If these arrangements are absent, the horizontal loads and the forces and bending moments due to horizontal displacements ("sideways") have to be resisted and transmitted by the frame itself, which in this case is referred to as an "unbraced frame". A characteristic collapse mode for this type of framed structure is shown in Fig. C.4^b.



(a) braced frame



(b) unbraced frame

Characteristic collapse mode for frames.

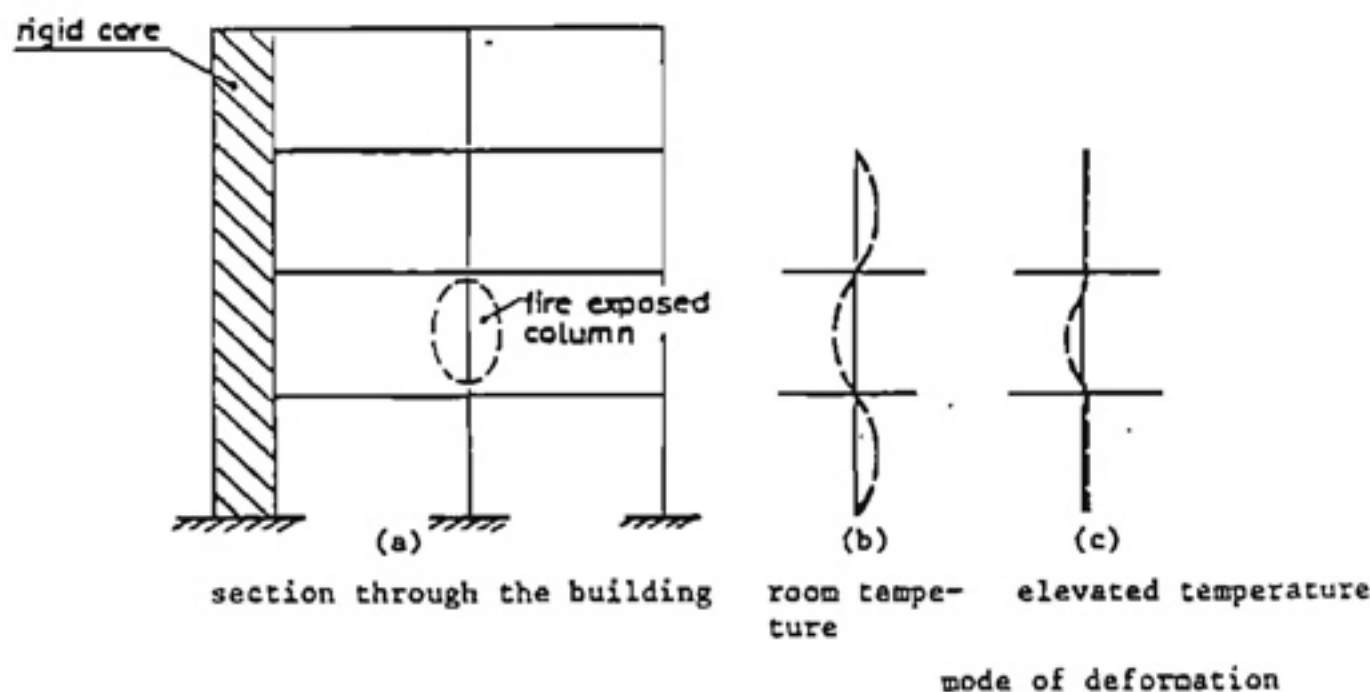
Fig. C.4

In view of the difference in collapse behaviour, the calculation of the critical temperature of columns in braced frames will differ from that in unbraced frames. In both cases, however, the calculation is rather complicated, even if highly simplifying assumptions are made. This is due, on the one hand, to the curvature of the stress-strain diagrams for steel at elevated temperature (so-called physical non-linearity: see Fig. IV.2) and, on the other hand, to the fact that the deformations will have a major effect upon the pattern of forces in the structure at collapse (so-called geometric non-linearity). Computer programmes are available from which the calculation of the critical temperature is possible in principle [3], [4].

It is a very laborious calculation, however. Besides, more particularly in the case of unbraced frames, it involves making assumptions which have a major effect on the result. In actual practice, therefore, calculations as discussed above will often be too elaborate for the purpose concerned. Hence there is a need for simple rules with which an approximate estimate of the critical temperature can be made. In deriving such rules it is an obvious decision to distinguish between columns in braced and in unbraced frames respectively.

Columns in braced frames

Just as in the case of centrally loaded columns, the critical temperature will be affected by the magnitude of the load and by the slenderness ratio. A problem in connection with this is: how should the slenderness ratio under fire conditions be determined? In design calculations for room temperature conditions, the slenderness ratio is usually based upon the column height. The underlying consideration is that, in the event of loss of stability, a deformation pattern as shown in Fig. C.5^b will develop ("checkerboard" loading).

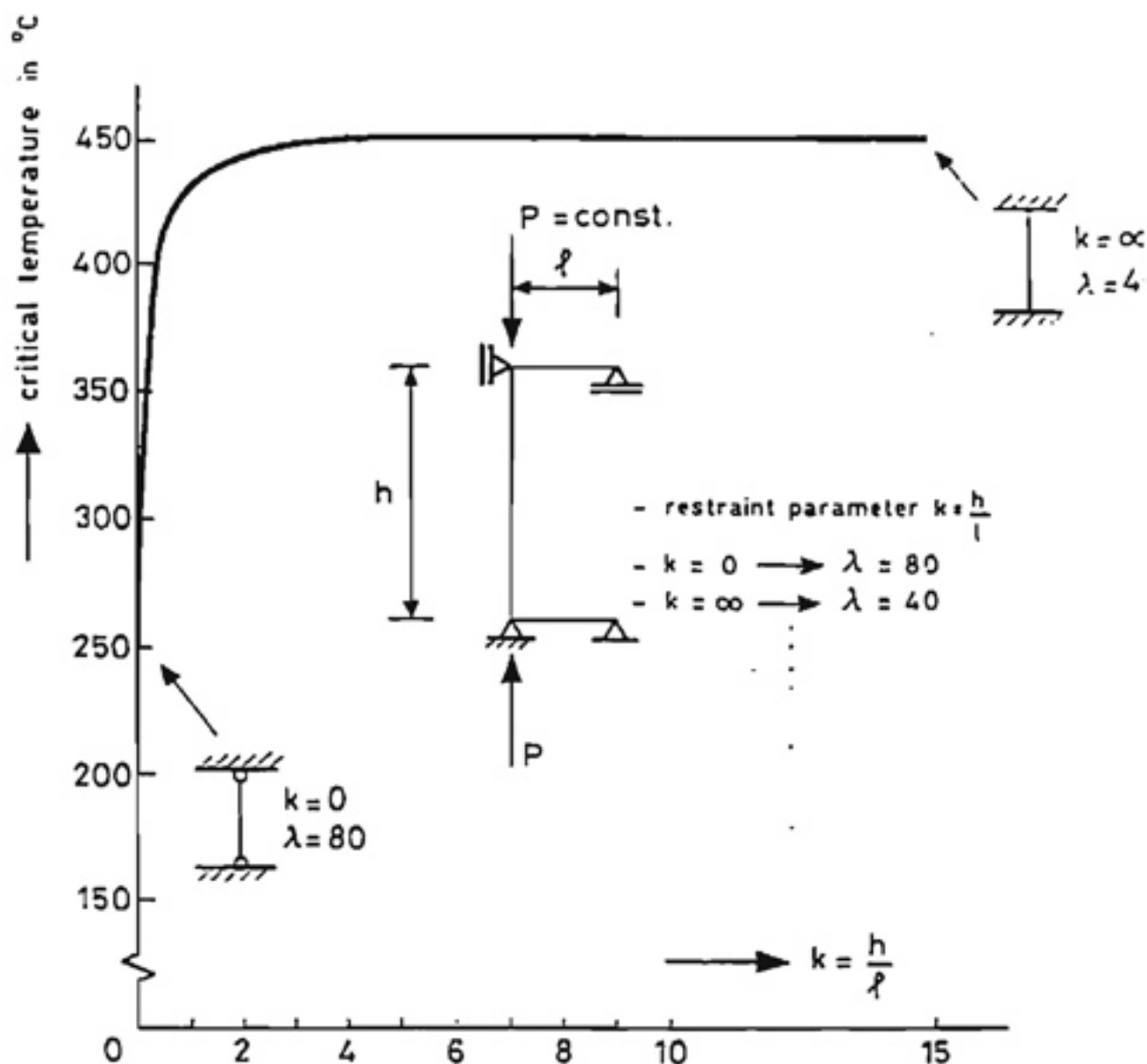


Structural behaviour of columns in braced frames (schematic)

Fig. C.5

In view of this deflection pattern it is not justifiable to rely on support from the columns above and below the column concerned. The elastic restraint due to the beams involved is negligible. During a fire a fundamentally different situation is liable to arise, more particularly if the fire is confined to one storey of the building. The columns exposed to the fire will undergo a loss of stiffness, so that structural stability will be endangered. However, the columns in the storeys above and below will be protected from the fire by the floors, so that the temperature of those columns will not rise and their stiffness not be reduced. Under such circumstances it is permissible to rely on a certain amount of restraint of the "hot" columns by the columns above and below them, as indicated schematically in Fig. C.5^c.

From tentative calculations it emerges that even a very partial degree of restraint has a significant effect on the critical temperature [5]. This effect is further illustrated in Fig. C.6.



The critical temperature of a column as function of the degree of moment restraint.

Fig. C.6

In this diagram the critical temperature of a column has been plotted as a function of the degree of end restraint. The elastic restraint is conceived as being due to beams which are connected to the top and base of the column and which have the same section properties as the column. See Fig. C.6. The relative stiffness of such a restraint is characterized by the quotient of column height and beam length ($k = h/l$). For $k = \infty$ the column is completely restrained; $k = 0$ corresponds to a column, hinged at both ends. It appears from Fig. C.6 that

even for a relatively low degree of restraint the corresponding critical temperatures are equal to those for complete restraint. Besides, in actual practice, the upper and lower ends of the column are likely to lag behind in temperature rise. The effect of this has not been incorporated into the diagram, but will likewise favourably influence the critical temperature. This being so, it is proposed, as an approximate design rule, that the slenderness ratio of a column under fire conditions should be based on an effective length equal to half the system length. Obviously, this rule will have to be modified if, for example, there is no upper column because the column exposed to fire happens to be in the top storey of the building. In that case only the base of the column may be conceived as restrained and the slenderness ratio under fire conditions should be based on an effective length equal to 0.7 times the system length of that column. Furthermore, the above-mentioned rule of thumb can permissibly be applied only if the fire is confined to one storey of the building.

Besides the slenderness ratio, the load will also have an influence on the critical temperature. As distinct from the load acting on a centrically loaded column, it will comprise not only a normal force, but also bending moments which are transmitted via the top and base to the column. A complicating factor is that these top and bottom moments will change during the fire. Pending the results of further research, it is proposed that the effect of the top and bottom moments be neglected in determining the critical temperature, i.e., the analysis is performed on the assumption that the column is centrically loaded. The critical temperature can then be determined on the basis of the buckling curves discussed in Section IV.7. This approach is, however, liable to produce unsafe solutions. For this reason an upper limit of 550 °C must be applied to the critical temperature calculated in this way, i.e., no higher value than this is allowed to be adopted.

Columns in unbraced frames

As already explained at the beginning of this Appendix, the behaviour of unbraced frames is much more complicated to describe than that of braced frames, because of the occurrence of frame instability. Such instability will not occur in relatively low unbraced frames, however. It is therefore proposed that the rules of thumb for determining the critical temperature of columns in braced frames, as discussed above, should also be applied to unbraced frames, provided that the latter do not comprise more than two storeys.

The rule cannot justifiably be applied to unbraced frames more than two storeys in height. In such cases the critical temperature of the columns will have to be determined from a calculation in which the frame instability is explicitly taken into account. If such a calculation is not performed, an upper limit of 300°C must be imposed for the critical temperature, i.e., no higher value than this is allowed to be adopted in the structural calculations. This relatively low value is imposed for reasons already explained in the foregoing. See also [5].

Appendix D: Design Tables

Table D.1^a : Steel properties at elevated temperatures

Temperature	Yield stress		E-modulus	Specific heat	Elongation	
θ_s °C	$\frac{\sigma_{y\theta}}{\sigma_{y20}}$	Fe 360 N/mm ²	Fe 510 N/mm ²	E_{θ} N/mm ²	c_s kJ/kg°C	$\Delta l/l$ %
20	1.000	235	355	210000	0.470	0.03
300	0.778	183	276	192460	0.564	0.366
310	0.766	180	272	191190	0.569	0.380
320	0.754	177	268	189820	0.573	0.395
330	0.742	174	263	188340	0.577	0.410
340	0.729	171	259	186730	0.582	0.424
350	0.717	168	254	184980	0.587	0.439
360	0.703	165	250	183070	0.591	0.454
370	0.690	162	245	180990	0.596	0.469
380	0.676	159	240	178730	0.601	0.484
390	0.661	155	235	176260	0.606	0.499
400	0.647	152	230	173560	0.611	0.514
410	0.632	148	224	170620	0.616	0.529
420	0.616	145	219	167420	0.621	0.545
430	0.601	141	213	163930	0.626	0.560
440	0.585	137	208	160130	0.632	0.575
450	0.568	134	202	156010	0.637	0.591
460	0.551	130	196	151530	0.642	0.607
470	0.534	126	190	146670	0.648	0.622
480	0.516	121	183	141410	0.654	0.638
490	0.498	117	177	135720	0.659	0.654
500	0.480	113	170	129570	0.665	0.670
510	0.461	108	164	122940	0.671	0.686
520	0.441	104	157	115790	0.677	0.702
530	0.422	99	150	108100	0.683	0.718
540	0.401	94	142	99830	0.689	0.735
550	0.381	89	135	90960	0.695	0.751
560	0.359	84	128	81450	0.701	0.767
570	0.338	79	120	71270	0.707	0.784
580	0.315	74	112	60380	0.714	0.801
590	0.293	69	104	48750	0.720	0.817
600	0.269	64	96	36350	0.727	0.834
610	0.248	58	88		0.733	0.851
620	0.228	54	81		0.740	0.868
630	0.210	49	75		0.747	0.885
640	0.194	46	69		0.754	0.902
650	0.180	42	64		0.761	0.919
660	0.167	39	59		0.768	0.936
670	0.155	36	55		0.775	0.954
680	0.144	34	51		0.782	0.971
690	0.134	32	48		0.789	0.988
700	0.125	29	44		0.796	1.006

Table D.1^b : Yield stress of steel at elevated temperatures
(Equations IV.1^{a,b})

θ °C	$\frac{\sigma_{y\theta}}{\sigma_{y20}}$	θ °C	$\frac{\sigma_{y\theta}}{\sigma_{y20}}$	θ °C	$\frac{\sigma_{y\theta}}{\sigma_{y20}}$	θ °C	$\frac{\sigma_{y\theta}}{\sigma_{y20}}$	θ °C	$\frac{\sigma_{y\theta}}{\sigma_{y20}}$
300	0.778	350	0.716	400	0.647	450	0.568	500	0.480
301	0.777	351	0.715	401	0.645	451	0.566	501	0.478
302	0.776	352	0.714	402	0.644	452	0.565	502	0.476
303	0.775	353	0.713	403	0.642	453	0.563	503	0.474
304	0.774	354	0.711	404	0.641	454	0.561	504	0.472
305	0.772	355	0.710	405	0.639	455	0.560	505	0.470
306	0.771	356	0.709	406	0.638	456	0.558	506	0.468
307	0.770	357	0.707	407	0.636	457	0.556	507	0.466
308	0.769	358	0.706	408	0.635	458	0.555	508	0.465
309	0.768	359	0.705	409	0.633	459	0.553	509	0.463
310	0.766	360	0.703	410	0.632	460	0.551	510	0.461
311	0.765	361	0.702	411	0.630	461	0.549	511	0.459
312	0.764	362	0.700	412	0.629	462	0.548	512	0.457
313	0.763	363	0.699	413	0.627	463	0.546	513	0.455
314	0.762	364	0.698	414	0.626	464	0.544	514	0.453
315	0.761	365	0.696	415	0.624	465	0.543	515	0.451
316	0.759	366	0.695	416	0.622	466	0.541	516	0.449
317	0.758	367	0.694	417	0.621	467	0.539	517	0.447
318	0.757	368	0.692	418	0.619	468	0.537	518	0.445
319	0.756	369	0.691	419	0.618	469	0.536	519	0.443
320	0.754	370	0.690	420	0.616	470	0.534	520	0.441
321	0.753	371	0.688	421	0.615	471	0.532	521	0.439
322	0.752	372	0.687	422	0.613	472	0.530	522	0.437
323	0.751	373	0.685	423	0.612	473	0.529	523	0.435
324	0.750	374	0.684	424	0.610	474	0.527	524	0.433
325	0.748	375	0.683	425	0.608	475	0.525	525	0.431
326	0.747	376	0.681	426	0.607	476	0.523	526	0.429
327	0.746	377	0.680	427	0.605	477	0.522	527	0.428
328	0.745	378	0.678	428	0.604	478	0.520	528	0.426
329	0.743	379	0.677	429	0.602	479	0.518	529	0.424
330	0.742	380	0.676	430	0.601	480	0.516	530	0.422
331	0.741	381	0.674	431	0.599	481	0.514	531	0.420
332	0.740	382	0.673	432	0.597	482	0.513	532	0.417
333	0.738	383	0.671	433	0.596	483	0.511	533	0.415
334	0.737	384	0.670	434	0.594	484	0.509	534	0.413
335	0.736	385	0.668	435	0.593	485	0.507	535	0.411
336	0.735	386	0.667	436	0.591	486	0.505	536	0.409
337	0.733	387	0.666	437	0.589	487	0.504	537	0.407
338	0.732	388	0.664	438	0.588	488	0.502	538	0.405
339	0.731	389	0.663	439	0.586	489	0.500	539	0.403
340	0.729	390	0.661	440	0.584	490	0.498	540	0.401
341	0.728	391	0.660	441	0.583	491	0.496	541	0.399
342	0.727	392	0.658	442	0.581	492	0.494	542	0.397
343	0.726	393	0.657	443	0.580	493	0.493	543	0.395
344	0.724	394	0.655	444	0.578	494	0.491	544	0.393
345	0.723	395	0.654	445	0.576	495	0.489	545	0.391
346	0.722	396	0.653	446	0.575	496	0.487	546	0.389
347	0.720	397	0.651	447	0.573	497	0.485	547	0.387
348	0.719	398	0.650	448	0.571	498	0.483	548	0.385
349	0.718	399	0.648	449	0.570	499	0.482	549	0.383

Table D.1^b (continued)

θ °C	$\frac{\sigma_{y\theta}}{\sigma_{y20}}$	θ °C	$\frac{\sigma_{y\theta}}{\sigma_{y20}}$	θ °C	$\frac{\sigma_{y\theta}}{\sigma_{y20}}$
550	0.380	600	0.270	650	0.180
551	0.378	601	0.268	651	0.179
552	0.376	602	0.265	652	0.177
553	0.374	603	0.263	653	0.176
554	0.372	604	0.261	654	0.175
555	0.370	605	0.259	655	0.173
556	0.368	606	0.256	656	0.172
557	0.366	607	0.254	657	0.171
558	0.364	608	0.252	658	0.169
559	0.361	609	0.250	659	0.168
560	0.359	610	0.248	660	0.167
561	0.357	611	0.246	661	0.166
562	0.355	612	0.244	662	0.164
563	0.353	613	0.242	663	0.163
564	0.351	614	0.240	664	0.162
565	0.348	615	0.238	665	0.161
566	0.346	616	0.236	666	0.160
567	0.344	617	0.234	667	0.158
568	0.342	618	0.232	668	0.157
569	0.340	619	0.230	669	0.156
570	0.337	620	0.228	670	0.155
571	0.335	621	0.226	671	0.154
572	0.333	622	0.224	672	0.153
573	0.331	623	0.222	673	0.152
574	0.329	624	0.221	674	0.150
575	0.326	625	0.219	675	0.149
576	0.324	626	0.217	676	0.148
577	0.322	627	0.215	677	0.147
578	0.320	628	0.214	678	0.146
579	0.318	629	0.212	679	0.145
580	0.315	630	0.210	680	0.144
581	0.313	631	0.209	681	0.143
582	0.311	632	0.207	682	0.142
583	0.308	633	0.205	683	0.141
584	0.306	634	0.204	684	0.140
585	0.304	635	0.202	685	0.139
586	0.302	636	0.201	686	0.138
587	0.299	637	0.199	687	0.137
588	0.297	638	0.197	688	0.136
589	0.295	639	0.196	689	0.135
590	0.292	640	0.194	690	0.134
591	0.290	641	0.193	691	0.133
592	0.288	642	0.191	692	0.132
593	0.286	643	0.190	693	0.131
594	0.283	644	0.188	694	0.130
595	0.281	645	0.187	695	0.129
596	0.279	646	0.186	696	0.128
597	0.276	647	0.184	697	0.127
598	0.274	648	0.183	698	0.126
599	0.272	649	0.181	699	0.126

Table D.2: Mean temperatures in unprotected steel members under standard fire exposure
 (initial temperature: 20°C; time step $\Delta t = 0.5$ min.)

TIME Mn	SECTION FACTOR $F/V \text{ m}^{-1}$											
	10	15	20	25	30	40	50	60	70	80	90	100
15	95	131	164	196	226	282	333	378	418	454	486	514
30	215	296	367	429	484	572	637	685	721	747	767	781
45	343	460	552	625	682	760	808	836	854	865	872	877
60	467	603	700	767	814	869	897	911	919	924	928	930
75	580	721	809	864	898	934	949	957	961	964	966	967
90	678	814	889	930	954	976	985	990	993	995	996	997

Table D.3: Mean temperatures in protected steel members under standard fire exposure
(initial temperature: 20°C; time step $\Delta t = 0.5$ min.)

TIME Mn	30	40	50	60	70	80	90	100	150	200	250	300	400	500
SECTION FACTOR F_d/V m ⁻¹														
INSULATION $d_s/\lambda_s = 0.90$ m ² ·C/W														
15			28	29	31	32	33	35	42	49	57	64	77	91
30			38	41	44	48	51	55	71	87	103	118	147	174
45			48	54	59	65	70	76	101	126	150	172	214	252
60			60	68	75	83	90	97	132	165	196	225	277	324
75			72	81	91	100	110	119	163	203	240	274	336	389
90			83	95	107	118	130	141	192	239	282	321	390	448
105			95	109	123	136	149	162	221	275	322	365	439	501
120			107	123	139	154	169	183	250	308	360	406	485	549
INSULATION $d_s/\lambda_s = 0.80$ m ² ·C/W														
15			28	30	32	33	35	37	45	53	61	69	84	99
30			40	44	47	51	55	59	77	95	112	129	160	190
45			52	58	64	70	76	82	111	138	164	188	233	274
60			65	73	82	90	98	106	144	180	214	245	301	351
75			78	88	99	110	120	130	178	221	261	298	363	419
90			91	104	117	129	142	154	210	261	307	348	420	480
105			104	119	134	149	163	177	242	299	349	394	471	534
120			117	135	152	169	185	201	272	335	389	437	518	583
INSULATION $d_s/\lambda_s = 0.70$ m ² ·C/W														
15			30	31	33	35	37	39	48	57	66	75	92	109
30			42	47	51	55	60	64	85	105	124	142	177	209
45			56	63	70	77	84	90	122	152	181	208	257	301
60			71	80	90	99	108	117	160	199	236	269	330	382
75			85	97	109	121	133	144	197	244	288	327	395	453
90			100	115	129	143	157	170	232	287	336	380	455	516
105			115	132	149	165	181	196	267	328	381	429	508	572
120			130	149	168	187	205	222	300	366	424	474	556	622
INSULATION $d_s/\lambda_s = 0.60$ m ² ·C/W														
15			31	33	36	38	40	42	53	63	73	83	103	121
30			46	51	56	61	66	71	94	117	139	159	198	234
45			62	70	78	86	93	101	137	171	203	232	286	333
60			78	89	100	111	121	131	180	223	264	300	365	420
75			95	109	123	136	149	162	221	273	320	362	434	494
90			112	129	145	161	177	191	260	320	372	418	496	559
105			129	149	168	186	203	221	298	364	420	470	551	616
120			146	168	189	210	230	249	334	405	465	517	601	666

Table D.3 (continued)

TIME	SECTION FACTOR $F_t/V \text{ m}^{-1}$												
Mn	30	40	50	60	70	80	90	100	150	200	250	300	400
INSULATION $d_t/\lambda_t = 0.50 \text{ m}^2\text{C/W}$													
15			33	36	38	41	44	46	59	71	83	95	117
30			51	57	63	69	74	80	108	134	159	182	226
45			70	79	88	98	107	115	157	196	232	265	324
60			89	102	115	127	139	151	206	255	299	339	409
75			109	125	141	156	171	186	252	310	361	406	482
90			129	148	167	185	203	219	296	361	418	466	547
105			148	171	192	213	233	252	338	409	469	520	603
120			168	193	217	241	263	284	377	453	516	569	653
INSULATION $d_t/\lambda_t = 0.40 \text{ m}^2\text{C/W}$													
15			36	40	43	46	49	52	68	83	97	111	138
30			58	65	73	80	87	94	127	158	187	214	264
45			81	93	104	115	126	136	186	231	271	308	373
60			105	120	135	150	164	178	242	298	347	391	465
75			129	148	167	185	202	219	295	360	415	464	543
90			152	175	197	219	239	258	345	416	476	527	609
105			176	202	227	251	274	296	391	468	531	583	666
120			199	228	256	283	308	332	434	515	580	633	716
INSULATION $d_t/\lambda_t = 0.30 \text{ m}^2\text{C/W}$													
15	33	37	41	46	50	54	58	62	82	101	119	136	169
30	50	60	70	79	88	97	106	115	157	195	230	262	320
45	69	84	99	114	128	142	155	168	229	282	329	371	442
60	88	109	129	149	168	186	203	220	296	360	415	463	540
75	108	134	159	183	206	228	249	269	358	430	490	540	620
90	127	159	189	217	244	269	293	316	414	492	555	607	687
105	147	183	218	250	280	308	335	360	466	547	612	664	743
120	166	207	246	281	314	345	374	401	513	597	662	714	792

Table D.3 (continued)

TIME	SECTION FACTOR F_d/V m ⁻¹												
Mn	30	40	50	60	70	80	90	100	150	200	250	300	400
INSULATION $d_f/\lambda_f = 0.20$ m ² ·C/W													
15	39	45	51	57	63	69	74	80	108	134	158	181	
30	64	78	91	104	117	130	142	154	209	258	302	341	
45	90	112	133	153	172	191	208	225	302	366	421	467	
60	118	147	174	200	225	249	271	292	385	458	518	568	
75	145	181	215	246	276	303	329	354	457	536	598	648	
90	172	214	253	290	323	354	383	410	521	603	666	715	
105	198	247	291	331	368	402	433	461	577	660	723	771	
120	224	278	326	370	409	445	478	508	627	711	772	818	
TIME	SECTION FACTOR F_d/V m ⁻¹												
Mn	10	20	30	40	50	60	70	80	90	100	150	200	250
INSULATION $d_f/\lambda_f = 0.10$ m ² ·C/W													
15			54	65	76	86	97	107	117	126	172	212	249
30			99	123	146	168	189	209	228	247	327	393	447
45			146	182	216	247	276	304	329	353	454	529	588
60			192	239	281	320	355	388	418	445	555	633	690
75			237	292	342	386	426	462	494	523	637	714	768
90			279	343	398	446	489	527	560	591	705	779	829
105			320	390	449	500	545	584	619	649	762	832	878
120			359	434	496	550	595	635	670	700	810	876	919
INSULATION $d_f/\lambda_f = 0.05$ m ² ·C/W													
15	40	60	78	96	114	131	147	163	178	193	259	315	
30	68	112	153	191	226	259	289	317	343	368	467	540	
45	98	167	228	282	330	373	411	445	476	505	612	683	
60	129	221	298	364	421	469	512	549	582	611	716	780	
75	160	273	363	438	499	551	595	633	666	694	793	850	
90	191	322	423	503	568	621	665	703	734	762	853	902	
105	222	368	477	561	628	681	725	761	791	817	901	943	
120	251	411	527	613	680	733	776	811	839	864	940	977	
INSULATION $d_f/\lambda_f = 0.04$ m ² ·C/W													
15	44	66	88	109	129	148	166	184	201	217	290		
30	76	128	175	218	257	293	326	356	384	409	512		
45	112	191	260	319	371	417	457	493	525	553	658		
60	148	252	338	409	469	519	563	600	632	660	759		
75	184	311	409	488	552	604	648	685	716	743	832		
90	220	365	473	557	622	675	718	753	783	808	888		
105	254	416	531	617	683	735	776	810	838	861	932		
120	288	463	583	670	735	786	826	857	883	904	967		

Table D.3 (continued)

TIME	SECTION FACTOR $F_t/V \text{ m}^{-1}$											
Mn	10	15	20	25	30	40	50	60	70	80	90	100
INSULATION $d_t/\lambda_t = 0.03 \text{ m}^2\text{C/W}$												
15	48	62	75	88	101	126	149	172	193	213	232	251
30	88	120	150	178	204	254	298	338	374	406	436	463
45	131	180	224	265	303	369	426	474	516	553	584	612
60	175	239	296	346	392	468	531	583	626	662	693	719
75	218	296	362	420	470	553	618	669	711	746	774	798
90	260	349	423	486	539	625	689	740	780	812	838	859
105	300	399	479	545	600	686	750	798	835	865	888	907
120	339	446	530	598	654	739	801	847	881	908	929	946
INSULATION $d_t/\lambda_t = 0.02 \text{ m}^2\text{C/W}$												
15	56	73	89	106	121	151	180	206	231	255	277	299
30	107	146	182	217	249	306	357	402	441	476	507	535
45	161	221	275	323	366	440	501	552	594	630	661	687
60	216	293	360	418	468	550	614	665	705	738	766	788
75	269	361	437	501	554	639	702	751	788	817	841	860
90	320	424	506	573	628	713	773	818	851	877	897	913
105	369	481	568	637	692	774	831	871	901	924	941	954
120	415	534	623	692	746	825	878	915	942	961	975	986
INSULATION $d_t/\lambda_t = 0.01 \text{ m}^2\text{C/W}$												
15	68	91	113	135	155	194	230	263	294	322	349	373
30	139	191	239	283	323	393	451	501	543	579	609	635
45	215	292	360	418	469	551	615	664	703	734	759	780
60	289	387	467	532	587	671	731	775	808	832	851	866
75	360	472	559	627	682	762	815	853	879	898	912	922
90	426	547	637	706	758	832	878	909	930	945	955	963
105	487	614	705	771	820	887	927	953	969	981	989	994
120	542	673	762	826	871	931	965	987	1000	1009	1015	1020
WITHOUT CLADDING $d_t/\lambda_t = 0$												
15	95	131	164	196	225	282	333	378	418	454	486	514
30	215	296	367	429	484	572	637	685	721	747	767	781
45	343	460	552	625	682	760	808	836	854	865	872	877
60	467	603	700	767	814	869	897	911	919	924	928	930
75	580	721	809	864	898	934	949	957	961	964	966	967
90	678	814	889	930	954	976	985	990	993	995	996	997
105	762	886	946	977	993	1007	1013	1016	1018	1020	1021	1022
120	833	942	989	1011	1021	1031	1036	1038	1040	1041	1042	1043

Table D.4: ECCS-buckling curves at room temperature (C-curves)

λ	Buckling stresses N/mm^2									
	0	1	2	3	4	5	6	7	8	9
FE 360										
10	235.0	235.0	235.0	235.0	235.0	235.0	235.0	235.0	235.0	234.7
20	233.5	232.2	230.9	229.6	228.4	227.1	225.8	224.6	223.3	222.0
30	220.7	219.5	218.2	216.9	215.6	214.3	213.0	211.7	210.3	209.0
40	207.7	206.3	205.0	203.6	202.3	200.9	199.5	198.1	196.7	195.3
50	193.9	192.4	191.0	189.6	188.1	186.6	185.2	183.7	182.2	180.7
60	179.2	177.7	176.1	174.6	173.1	171.5	170.0	170.0	168.4	166.9
70	163.7	162.2	160.6	159.0	157.5	155.9	154.3	152.8	151.2	149.6
80	148.1	146.5	144.9	143.4	141.8	140.3	138.8	137.3	135.7	134.2
90	132.7	131.2	129.8	128.3	126.8	125.4	124.0	122.5	121.1	119.7
100	118.3	117.0	115.6	114.3	112.9	111.6	110.3	109.0	107.8	106.5
110	105.3	104.0	102.8	101.6	100.5	99.3	98.1	97.0	95.9	94.8
120	93.7	92.6	91.5	90.5	89.4	88.4	87.4	86.4	85.4	84.5
130	83.5	82.6	81.6	80.7	79.8	78.9	78.0	77.2	76.3	75.5
140	74.7	73.9	72.3	71.5	70.7	69.9	69.2	68.5	67.7	67.0
150	67.0	66.3	65.6	64.9	64.2	63.6	62.9	62.3	61.6	61.0
160	60.4	59.8	59.2	58.6	58.0	57.4	56.8	56.3	55.7	55.2
170	54.6	54.1	53.6	53.0	52.5	52.0	51.5	51.0	50.6	50.1
180	49.6	49.1	48.7	48.2	47.8	47.3	46.9	46.5	46.1	45.6
190	45.2	44.8	44.4	44.0	43.6	43.2	42.9	42.5	42.1	41.7
FE 510										
10	335.0	355.0	355.0	355.0	355.0	355.0	353.3	350.9	348.6	346.2
20	343.9	341.5	339.2	336.8	334.4	332.1	329.7	327.3	324.9	322.4
30	320.0	317.5	315.1	312.6	310.1	307.6	305.0	302.5	299.9	297.3
40	294.7	292.0	289.4	286.7	284.0	281.2	278.5	275.7	273.0	270.2
50	267.3	264.5	261.7	258.8	255.9	253.0	250.1	247.2	244.3	241.4
60	238.5	235.6	232.7	229.7	226.8	223.9	221.0	218.1	215.3	212.4
70	209.6	206.7	203.9	201.1	198.4	195.6	192.9	190.2	187.5	184.5
80	182.3	179.7	177.2	174.6	172.1	169.7	167.3	164.9	162.5	160.2
90	157.9	155.6	153.4	151.2	149.1	146.9	144.8	142.8	140.8	138.8
100	136.8	134.9	133.0	131.1	129.3	127.5	125.7	124.0	122.3	120.6
110	119.0	117.3	115.8	114.2	112.7	111.1	109.7	108.2	106.8	105.4
120	104.0	102.6	101.3	100.0	98.7	97.4	96.2	95.0	93.8	92.6
130	91.4	90.3	89.2	88.1	87.0	85.9	84.9	83.8	82.8	81.8
140	80.9	79.9	79.0	78.0	77.1	76.2	75.3	74.5	73.6	72.8
150	71.9	71.1	70.3	69.5	68.8	68.0	67.2	66.5	65.8	65.1
160	64.4	63.7	63.0	62.3	61.7	61.0	60.4	59.7	59.1	58.5
170	57.9	57.3	56.7	56.1	55.6	55.0	54.4	53.9	53.4	52.8
180	52.3	51.8	51.3	50.8	50.3	49.8	49.3	48.9	48.4	48.0
190	47.5	47.1	46.6	46.2	45.8	45.3	44.9	44.5	44.1	43.3

Table D.5: Section factors of various steel sections $F_d/V \text{ m}^{-1}$

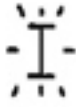
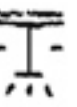
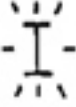
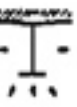
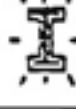
IPE					IPN				
80	430.6	329.8	370.4	269.6	80	401.1	321.9	345.6	266.5
100	389.3	301.0	335.9	247.6	100	349.1	283.0	301.9	235.8
120	359.1	278.8	310.6	230.3	120	309.2	250.7	268.3	209.9
140	335.4	259.8	290.9	215.2	140	274.3	225.1	238.3	189.1
160	309.5	240.8	268.7	200.0	160	252.2	205.3	219.7	172.8
180	292.1	226.8	254.0	188.7	180	229.4	187.8	200.0	158.4
200	269.5	210.5	234.4	175.4	200	211.6	173.1	184.8	146.3
220	253.9	197.6	221.0	164.7	220	195.7	160.6	171.0	135.9
240	235.5	184.1	204.9	153.5	240	183.1	150.1	160.1	127.1
270	226.6	176.5	197.2	147.1	260	169.7	139.7	148.5	118.5
300	215.6	167.3	187.7	139.4	280	158.1	130.6	138.6	111.1
330	199.7	156.5	174.1	131.0	300	149.1	123.0	131.0	104.9
360	185.7	145.8	162.3	122.4	320	140.1	115.9	123.3	99.1
400	174.0	137.3	152.7	116.0	340	132.5	109.9	116.7	94.1
450	163.0	129.6	143.7	110.3	360	124.6	103.6	109.9	88.9
500	150.0	120.7	132.8	103.4	380	118.7	98.9	104.8	85.0
550	140.3	113.4	124.6	97.8	400	112.7	94.1	99.6	80.9
600	129.5	105.1	115.4	91.0	425	106.8	89.1	94.5	76.7
750x137	143.5	116.4	128.5	101.3	450	100.7	84.4	89.1	72.8
750x147	133.9	108.6	119.8	94.5	475	95.1	80.1	84.2	69.2
750x161	123.4	100.2	110.4	87.2	500	90.6	76.1	80.3	65.8
750x173	114.5	93.0	102.4	80.9	550	84.5	70.4	75.1	61.0
750x185	107.9	87.7	96.5	76.4	600	75.6	64.2	67.1	55.7
750x196	101.7	82.8	91.0	72.1					
750x210	95.7	77.9	85.6	67.8					
750x222	90.6	73.9	81.1	64.4					

Table D.5 (continued)

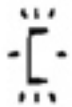
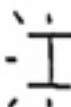
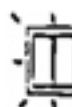
UPN					HEA				
80	283.6	227.3	242.7	186.4	100	354.6	244.9	290.5	180.8
100	275.6	222.2	238.5	185.2	120	360.6	246.9	295.9	182.2
120	255.3	205.9	222.9	173.5	140	341.7	232.8	230.9	172.0
140	239.7	196.1	210.3	166.7	160	296.8	202.9	244.1	150.2
160	227.5	187.5	200.4	160.4	180	278.7	190.0	229.5	140.7
180	218.2	178.6	193.2	153.6	200	256.1	174.9	210.8	129.6
200	205.3	170.8	182.0	147.5	220	242.3	165.2	199.6	122.4
220	192.0	160.4	170.6	139.0	240	225.1	153.7	185.3	114.0
240	183.2	153.7	163.1	133.6	260	213.7	146.2	176.0	108.5
260	172.7	144.9	154.0	126.3	280	204.1	139.4	168.3	103.6
280	167.0	140.7	149.2	122.9	300	191.7	131.1	158.0	97.4
300	161.6	136.1	144.6	119.0	320	183.9	127.1	152.2	95.4
320	129.6	110.8	116.4	97.6	340	176.8	123.4	146.9	93.5
350	135.4	116.4	122.5	103.5	360	170.1	119.9	142.0	91.7
380	138.1	119.9	125.4	107.2	400	160.6	115.2	135.1	89.7
400	129.2	111.5	117.2	99.5	450	156.1	114.1	132.5	90.5
					500	151.7	112.8	129.8	90.9
					550	142.3	107.6	122.7	87.9
					600	138.5	106.2	120.2	87.9
					650	134.8	104.7	117.7	87.6
					700	129.2	101.6	113.5	85.9
					800	121.8	97.9	108.0	84.2
					900	113.3	92.8	101.4	80.9
					1000	108.3	90.0	97.7	79.4
					L 1000	105.1	83.3	92.9	71.2
					X 1000	100.4	78.1	88.0	65.7
									

Table D.5 (continued)

HEA					HEB				
100	265.1	184.9	217.9	137.7	100	218.1	153.8	179.6	115.
120	267.6	185.0	220.2	137.5	120	201.8	141.2	166.5	105.
140	252.9	173.9	208.3	129.3	140	187.2	130.2	154.7	97.
160	230.9	160.8	189.7	119.6	160	169.1	117.9	139.6	88.
180	225.2	155.0	185.4	115.2	180	157.7	110.3	130.2	82.
200	211.9	145.0	174.7	107.8	200	147.2	102.4	121.6	76.
220	196.0	133.7	161.7	99.5	220	139.6	96.7	115.4	72.
240	178.4	122.4	147.1	91.1	240	130.2	90.6	107.5	67.
260	170.5	117.5	140.6	87.6	260	126.7	87.8	104.7	65.
280	164.4	113.1	135.7	84.3	280	123.3	85.2	102.0	63.
300	152.9	104.9	126.2	78.2	300	116.0	80.5	95.9	60.
320	141.5	98.1	117.4	74.0	320	109.7	76.9	91.1	58.
340	134.1	94.4	111.6	71.9	340	105.9	74.9	88.4	57.
360	128.2	91.0	107.1	70.0	360	102.4	73.1	85.8	56.
400	120.1	86.8	101.3	67.9	400	97.6	70.8	82.4	55.
450	112.9	83.1	96.1	66.3	450	91.3	68.8	77.5	55.
500	106.8	80.0	91.6	64.8	500	88.9	67.1	76.3	54.
550	104.3	79.3	90.2	65.2	550	87.4	66.9	75.6	55.
600	102.0	78.6	88.7	65.3	600	85.9	66.7	74.8	55.
650	99.6	77.8	87.2	65.4	650	84.6	66.4	74.1	55.
700	96.2	76.0	84.6	64.5	700	82.2	65.3	72.4	55.
800	94.4	76.3	83.9	65.8	800	81.2	65.8	72.2	56.
900	90.4	74.3	81.0	64.9	900	78.4	64.6	70.3	56.
1000	89.2	74.4	80.6	65.7	1000	77.8	65.0	70.3	57.
L 1000	85.5	68.0	75.7	58.2	L 1000	74.4	59.3	65.9	50.
X 1000	79.9	62.3	70.1	52.6	X 1000	73.0	57.0	64.1	48.







Table D.5 (continued)

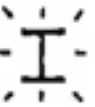
HEM				
100	116.4	85.0	96.4	65.0
120	111.1	80.1	92.2	61.1
140	103.6	75.9	85.5	57.8
160	99.9	71.3	82.8	54.2
180	96.2	68.1	79.8	51.7
200	91.4	64.9	75.7	49.2
220	88.4	62.4	73.2	47.3
240	73.1	51.9	60.7	39.5
260	71.5	50.8	59.3	38.6
280	70.4	49.8	58.4	37.8
300	60.4	42.9	50.1	32.7
320	59.9	42.8	50.0	32.9
340	60.2	43.4	50.4	33.7
360	60.5	44.1	50.9	34.4
400	61.4	45.4	52.0	35.9
450	62.6	46.8	53.5	37.7
500	63.3	48.2	54.4	39.3
550	64.3	49.5	55.7	40.9
600	65.2	50.9	56.8	42.5
650	66.0	52.1	57.9	43.9
700	66.8	53.3	58.9	45.8
800	67.9	55.3	60.4	47.8
900	69.3	57.2	62.1	50.1
1000	70.5	59.0	63.7	52.2
L 1000	67.3	58.8	59.7	46.1
X 1000	66.1	51.7	58.1	43.7





Table D.5 (continued)

HD				
210 x 6	204	140	169	105
52	180	124	149	93
59	161	111	134	84
71	134	93	111	70
87	111	78	93	59
100	98	69	81	52
118	84	60	70	46
138	74	53	62	40
161	65	46	54	36
198	54	39	45	30
249	45	33	37	26
260x73	160	109	132	82
80	146	100	121	75
89	132	91	110	68
101	117	81	97	61
115	104	72	86	54
131	92	64	77	49
149	82	57	68	44
167	74	52	62	40
191	66	46	55	35
219	58	41	48	32
252	51	37	43	28
288	46	33	38	26
329	41	30	34	23
310x97	144	99	120	74
107	132	90	109	68
117	121	83	101	63
130	110	76	91	57
143	100	69	83	52
158	91	63	76	48
179	81	57	68	43
202	73	51	61	39
227	65	46	54	35
253	59	42	50	32
283	54	38	45	29
313	49	35	41	27
343	45	33	38	25
375	42	30	35	23
415	39	28	32	22
454	36	26	30	20
500	33	24	28	19
				

Table D.5 (continued)

HD				
360x134	125	85	103	63
148	114	78	95	58
162	104	71	87	53
179	95	65	79	48
196	88	60	73	45
400x187	94	64	78	47
216	82	56	68	42
237	75	51	62	38
262	69	47	57	35
288	63	43	52	32
314	58	40	48	30
347	53	37	44	28
388	49	34	40	25
422	45	31	37	23
463	41	29	34	22
509	38	27	31	20
551	35	25	29	19
593	33	23	28	18
634	31	22	26	17
678	30	21	25	16
744	27	20	23	15
818	25	18	21	14
900	23	17	19	13
990	22	16	18	12
1086	20	15	17	12
				

NOTATION

A_f	= floor area of the fire compartment	[m ²]
A_s	= cross section area of the steel member	[mm ²]
A_t	= total interior area of the surfaces, bounding the fire compartment, including all openings	[m ²]
A_w	= total area of door and window openings	[m ²]
E	= modulus of elasticity of steel at room temperature	[N/mm ²]
$E_{t\theta}$	= 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]
H	= horizontal force	[N]
H_v	= calorific value of combustible material v	[MJ/kg]
H_{vF}	= calorific value of moist combustible material v	[MJ/kg]
K	= coefficient of the total heat transfer	[W/m ² ·C]
K_f	= coefficient for transforming a real fire load density ($=q_t$) and a real opening factor of a fire compartment ($=A_w\sqrt{hw}/A_t$) to effective values	[-]
M_p	= plastic moment at room temperature	[Nm]
$M_{p\theta}$	= plastic moment at elevated temperature	[Nm]
N	= design load on a column under fire conditions	[M]
$\bar{N}_{cr}$	= dimensionless ECCS-buckling load at room temperature	[-]
$\bar{N}_{cr\theta}$	= dimensionless buckling load at elevated temperature	[-]
P	= design load under fire conditions	[N] or [N/m ²]
P_e	= ultimate elastic load at room temperature	[N] or [N/m ²]
P_p	= ultimate plastic load at room temperature	[N] or [N/m ²]
P_θ	= actual load during fire	[N] or [N/m ²]
Q	= total quantity of heat, released on complete combustion of all combustible materials in a fire compartment	[J]
or:	= heat flow to structural steel element	[J/m ²]
$\bar{Q}$	= total fire load in kg wood equivalent	[kg]
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]
a	= distance between mid span plastic moment and end support	[m]
b	= flange width of the steel member	[m]
c _b	= specific heat of the bounding structural components of the fire compartment	[J/kg]
c _i	= specific heat of the insulation material	[J/kg°C]
c _s	= specific heat of steel	[J/kg°C]
d _i	= thickness of the insulation	[m]
d _s	= web or flange thickness of the steel member	[m]
h	= height of steel profile, column, storey	[mm]
h _w	= mean value of the heights of window and door openings in a fire compartment, weighted with respect to each individual opening area	[m]
k	= relative stiffness	[-]
l	= length, span of beam or frame	[mm], [m]
m _v	= total mass of combustible material	[kg]
p	= moisture content by weight of the insulation material	[%]
q	= design load under fire conditions	[N/m ²]
q _e	= ultimate elastic load at room temperature	[N/m ¹]
q _f	= fire load density, per unit floor area	[MJ/m ²]
q _{ff}	= effective fire load density, per unit floor area	[MJ/m ²]
q _p	= ultimate plastic load at room temperature	[N/m ¹]
q _t	= fire load density, per unit area of bounding surfaces	[MJ/m ²]
q _{tf}	= effective fire load density, per unit area of bounding surfaces	[MJ/m ²]
q _g	= rate of heat loss through structural components, bounding the fire compartment	[W/m ²]
q _g	= rate of heat loss by convection in the openings of the fire compartment	[W/m ²]

$\dot{q}_R$	= rate of heat loss by radiation through the openings of the fire compartment	[W/m ²]
$\dot{q}_t$	= rate of heat release by combustion inside the fire compartment	[W/m ²]
t	= time	[sec], [min], [h]
t_d	= standard fire duration	[min]
t_e	= equivalent time of fire duration	[min]
t_{fr}	= fire resistance	[min], [h]
t_{rf}	= required fire resistance	[min]
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 ·C]
or:	= coefficient in the ECCS-buckling curves dependent of type of cross section	[-]
or:	= factor (> 1) representing the plastic reserve of a steel beam due to a possible reduction of bending moments ($= \frac{Z}{W}$)	[-]
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]
α_t	= specific thermal elongation of steel ($= \Delta l/l^\circ C$)	[-]
γ	= degree of loading related to the ultimate plastic load ($= P/P_p$)	[-]
γ^*	= degree of loading related to the ultimate elastic load ($= P/P_e$)	[-]

$\gamma_e, \gamma_f, \gamma_{n1}, \gamma_{n2}$	= partial safety factors	[-]
δ	= maximum deflection	[mm]
ϵ	= strain	[%]
$\epsilon_{y\theta}$	= 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 < 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	[-]
$\bar{\lambda}_\theta$	= modified slenderness ratio at elevated temperature	[-]
λ_b	= thermal conductivity of the bounding structural components of the fire compartment	[W/m°C]
λ_i	= thermal conductivity of the insulation	[W/m°C]
u_v	= fraction between 0 and 1, giving the real degree of combustion for each individual component of the fire load	[-]
ρ_b	= density of the material of the bounding structural components of the fire compartment	[kg/m ²]
ρ_i	= density of the insulation material	[kg/m ³]
ρ_s	= density of steel	[kg/m ³]
σ	= stress	[N/mm ²]
σ_{cr20}	= ECCS buckling stress at room temperature	[N/mm ²]
$\sigma_{cr\theta}$	= ECCS buckling stress at elevated temperature	[N/mm ²]
σ_i	= initial stress	[N/mm ²]
σ_{y20}	= nominal yield stress at room temperature	[N/mm ²]
$\sigma_{y\theta}$	= effective yield stress at elevated temperature	[N/mm ²]

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