

Zeitschrift: IABSE surveys = Revue AIPC = IVBH Berichte
Band: 7 (1983)
Heft: S-22: Safety concept and design for fire resistance of steel structures

Artikel: Safety concept and design for fire resistance of steel structures
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DOI: <https://doi.org/10.5169/seals-47728>

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Safety Concept and Design for Fire Resistance of Steel Structures

Concept de sécurité et résistance au feu des structures en acier

Sicherheitskonzept und Berechnung des Feuerwiderstands von Stahlkonstruktionen

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SUMMARY

This paper is aimed at describing the principles on which are based the current concepts of fire engineering design of steel structures. The first part deals with the design methods for fire safety through the definition of heat exposure and structural models. The second part reviews the state of art in structural fire calculation methods, according to different levels of assumption.

RÉSUMÉ

Cet article se propose de décrire les principes sur lesquels se fonde la conception et le calcul de la résistance au feu des structures métalliques. Sa première partie est consacrée à la présentation de modèles concernant à la fois la description de l'incendie et la manière de prendre en compte le comportement de la structure. La deuxième partie passe en revue les différentes méthodes de calcul de la résistance au feu des structures en acier.

ZUSAMMENFASSUNG

Der vorliegende Beitrag befasst sich mit den Prinzipien, die dem Sicherheitskonzept und der Berechnung des Feuerwiderstands von Stahlkonstruktionen zugrunde liegen. Der erste Teil beschreibt Modelle für den Brandverlauf und das Tragwerkverhalten. Der zweite Teil zeigt die verschiedenen Methoden zur Berechnung des Brandwiderstands von Stahlkonstruktionen.



1. INTRODUCTION

During the last two decades, remarkable progress has been made in understanding not only the parameters which influence the development of building fires but also the behaviour of fire exposed structural materials and structures. In particular, for steel structures this progress has resulted in the production of very detailed rules for the design and calculation of structural behaviour and load bearing capacity in fire.

Nevertheless, it must be admitted that up to now the greater part of the research effort in relation to the fire behaviour of steel structures has been confined to the two aspects mentioned above, although in recent years it has become obvious that, whatever progress may have been made in a better assessment of the rôle played by compartmentation and structural fire behaviour, the answer to the problem of the fire safety of building is an incomplete one.

It must be recalled that the fire safety of a building also depends on other preventive measures such as automatic fire detection and extinction systems, smoke control and extract systems, as well as the thorough analysis of the potential risks (fire load, oxygen supply, fire spread, fire exit...).

This paper reviews the most recent developments and new concepts in the field of fire resistance of steel structures and fire safety.

2. PRINCIPLES OF FIRE ENGINEERING

2.1. Objectives and analyses for fire safety in building

Fire safety objectives at large can be summarized as follows (1,2), :

- Reducing the risk of injury and death of people.
- Reducing the risk of damage and loss of the building, the contents and the environment.

Incorrect decisions regarding the level of fire protection are liable to have serious consequences :

- Too low a standard of fire safety may involve unacceptable risk for persons in the building as well as for fire fighting personnel and may lead to excessive monetary losses.
- Too high a standard of fire safety will entail unnecessary expense.

There is an evident need to develop principles and procedures which lead to optimal fire safety standards. Fire prevention measures and suppression in general serve both social and monetary interests simultaneously. The overall objective is an optimum return on investment in fire precautions in terms of lives and property saved. As far as monetary fire losses only are concerned, a cost benefit analysis balancing gains with losses should be made. In other cases, both monetary losses and social losses, such as injury and death by fire, have to be taken into account.

Ideally, a rational analysis for fire safety should comprise the following elements :

1. agreed levels of life and property safety,
2. quantitative methods assessing potential hazards,
3. quantitative methods assessing the effectiveness of protective measures and combinations to meet the required levels of safety for the identified potential hazards.

The present regulations for structural fire safety and the state of knowledge however, do not generally comply with the above conditions. Therefore, fire prevention measures for building and in particular the structural performance requirements are often a matter of controversy between authorities and designers. The three elements of a rational analysis are discussed briefly below :

Element 1.

The intended level of fire safety is usually not explicitly stated in building regulations and is not even known in all relevant aspects. A common feature of existing regulations is that requirements for the benefit of human lives and health are more strongly emphasized than those aiming at protection of property.

Element 2.

In assessing the potential hazards for structural safety, the authorities take into account not only the expected severity and duration of the fire, but also in a rather arbitrary way, factors such as the type of occupancy, the height, the situation of the building and the importance of the structure or structural element. The expected duration of the fire and the effect of the additional factors are "added" and expressed in one single parameter, i.e. the required fire resistance time (see § 2.3.).

Element 3.

A mixture of objectives for life and property safety and the arbitrary way in which the potential hazards for the structural safety are expressed in the required fire resistance time makes it difficult, if not impossible, to balance alternative protective measures to meet the same level of safety. The single parameter "required fire resistance time" puts the emphasis on structural fire protection and hampers an assessment of a reduction in structural fire protection when active measures such as early detection and sprinklers are employed.

During recent years a changing attitude to existing regulations and codes has become apparent, and attempts are being made to arrive at a more rational analysis for fire safety (see § 2.3.).

A related problem is the influence of insurance policies on fire safety in buildings (3).

Fire insurance companies have, through their grading and rating systems, an important influence on fire protection. The insurance industry affects decisions made about fire protection by private individuals and by private and public bodies. Consider, for example, the fire protection of a single building: in general, insurance companies require certain minimum prevention measures, otherwise the insurer will refuse to insure the building. When additional fire prevention measures are employed, such as sprinklers, detectors and compartmentation, the rating is reduced. Thus the owner of the building can decide whether or not to invest in preventive measures. Such an approach is the best that an individual can do to minimize total expenditure. The insurer however deals with large numbers of different risks. This is the reason why his criteria for assessing risks are global and simple and sometimes even inconsistent in particular aspects. Therefore insurance grading, based on fire losses at large, may be a poor indication for optimal fire protection for a particular building. Insurers show a general reluctance to bring their risk assessment, premiums, profit and loss accounts in detail into the open. Therefore no independent studies of the effect of insurance policies on fire protection are available.



2.2. Structural fire protection and alternative protective measures

Personal risk and structural damage can be prevented or limited by many measures which generally serve life and property safety simultaneously. If a limited budget is to be used, how should resources be allocated to provide the optimum level of fire protection? That is, how much effort should go into active measures such as early detection and sprinklers, and how much should go into passive measures provided by the building structure itself? Apart from some pilot studies, few data are available for defining the input and output of fire prevention measures in terms of costs and productivity.

In order to reduce injury and loss of life, the most effective protective measures are early detection, lay-out of escape routes and control of combustible materials. The risk of a large fire occurrence can be reduced by compartmentation, the use of automatic sprinkler systems and early detection.

One of the oldest and, where steel structures are concerned, the most restrictive fire prevention measures is an increase in the fire resistance of the structural members. It is important to appreciate that protection afforded by structural fire resistance alone does not generally ensure adequate reduction in material damage and personal risk. Indeed, experience shows that large fires often damage the building so badly that it has to be demolished regardless of whether the structure has collapsed or not. The investment in structural fire protection would be useless in that case. Examples are industrial fires where no sprinklers are available to avoid flash-over or partitions to limit the fire spread. Instead of an over designed fire resistance of the structural members the essential measures which should be considered for industrial buildings are:

- Sprinklers to avoid flash-over and fire growth.
- Partitions to limit the fire spread.
- Fire ventilation to reduce smoke and corrosion damage and to facilitate the fire fighting.

When, for reasons of life and property safety, sprinklers are installed, it can be argued that the fire resistance of structural elements can be reduced. Therefore methods should be developed to assess the reduction of structural fire protection when alternative protective measures are employed.

This matter becomes increasingly important because there is a growing use of automatic detection and extinguishing systems in industrial as well as in public buildings. Examples of the last category are the increasing number of large covered shopping developments and high rise buildings. The trend of increasing use of active fire prevention measures is connected with the improved standard of living and protection in the western countries.

Higher standards of life and property safety are demanded by public authorities as well as insurance companies. This change in approach to the design of fire safety can have an important implication. If fire safety is assured by other measures, it seems logical that reduction or even elimination of structural fire protection should be considered (see § 2.3.).

An important item is the cost of fire protection (3). The cost of structural fire protection for steel elements depends on several factors, such as the materials used for protection, the type of structure to be protected and the degree of fire resistance required. Also an important factor is that some protective measures fulfil additional functions as partitions or suspended ceilings. A rough estimate of the cost of fire protection for European non-industrial buildings in steel is given in the accompanying table. The cost of fire protection is expressed as a percentage of the cost of the steel structure, for different required fire resistance times. In order to see these figures in their correct proportion one should bear in mind that generally the cost of the steel structure only forms 5 to 10% of the total building costs.

Fire protection (min)	Cost (%)
30	10
60	20
90	25
120	30

It is also emphasized that the figures cannot be used as a guide when making preliminary estimates. Deviations of say 30% upwards as well downwards are possible.

It appears that the cost of protection increases appreciably with increased fire resistance.

However, studies as well as examples reveal that when the fire protection is integrated into the design from the beginning of the study of the project the cost of fire protection can be considerably reduced (4).

It is particularly useful to consider combining functions such as anti-corrosion or aesthetic finishes with fire protection, or combining fire protection with thermal insulation for energy conservation.

2.3 Design methods for structural fire safety

2.3.1 Introduction

Internationally, the generally accepted method for the design of load bearing structural elements exposed to fire is based on a classification system, comprising two main components (5) :

1. A fire exposure according to ISO 834, with a required time of duration t_{fd} , stipulated in building regulations and codes for the structural application in question - usually expressed in multiples of 30 minutes.

2. A standard fire resistance test according to ISO 834 by which the fire resistance time t_{fr} of the structural element in question is determined experimentally - usually classified in multiples of 30 minutes.

The design implies a proof that the structural element has a fire resistance t_{fr} , which meets the required time of fire duration t_{fd} .

Although the classification system has been in use for over half a century, it has some serious weaknesses. These weaknesses apply to both components of the design procedure and can be summarized as follows :

1. The rise of temperature as a function of time according to ISO 834 and the fire duration are a rough approximation of the real gas-temperature time curve of a fully developed compartment fire.

The required time of fire duration is generally related, not only to the estimated fire exposure, but also to various safety considerations relevant for the building in question. This usually leads to a required time of fire duration, which is more severe than the actual fire exposure. The estimated fire exposure and the safety considerations are intermingled inextricably, which is a consequence of the fact that the building regulations do not provide any guidance as to the safety levels that they imply (2).

2. The specification of the fire resistance test according to ISO 834 is insufficient in several aspects, such as heat-flow 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 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 (6,7).



The deficiencies of the present classification system have certainly stimulated the development of rational methods of fire risk assessment and analytical modelling of thermal process as well as structural response, which potentially give the possibility of achieving solutions with greater economy and a defined and more uniform safety (8,9,10,11,12,13). Moreover, it is recognized that, following probabilistic design procedures in other fields of design for accidental events, structural fire engineering design should be probability based. In contrast to the present classification system, probabilistic design includes a methodology by which all relevant factors, such as safety considerations from both the human and economic point of view, probability of flash-over, uncertainties in fire exposure and structural response, the effect of fire brigade actions and sprinklers can be dealt with systematically (14,15).

2.3.2. Main elements for a structural fire engineering design

Generally a structural fire engineering design includes two main elements, corresponding to the components as described in the introduction.

1. A heat exposure model H , for the determination of the rise of temperature as a function of time.
2. A structural 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 structural member, under a defined load and subjected to the specified heat exposure, fulfils certain functional requirements, expressed by the limit states with respect to load bearing capacity, thermal insulation, fire integrity.

The available heat exposure models (H) and the structural models (S) can be characterized with respect to the type of thermal exposure and the type of structural system. The models are listed in sequence of improved schematic idealization, with a consequent increase in complexity of solution. For both types of model the listing starts with the components of the present deterministic classification system, discussed in paragraph 2.3.1. The improved models are probabilistic, including the explicit treatment of safety considerations and the effect of active protection measures, such as sprinklers (14,15).

The following probabilistic aspects are considered :

- intrinsic randomness of design parameters and properties.
- model uncertainties of the analytical models for the heat exposure and the structural response.
- assessment of frequency determined by the probability of flash-over, the effect of fire brigade actions, the reliability of detection systems and sprinklers.
- safety considerations from both the human and economic point of view such as the height, volume and occupancy of the buildings, the availability of escape routes and rescue facilities, as well as the consequences of reaching a limit state.

Heat exposure models

- (H_1) A rise of temperature as a function of time according to ISO 834.
The duration of the temperature rise is equal to the "required time of fire duration", expressed in building regulations and codes.
- (H_2) A rise of temperature as a function of time according to ISO 834.
The duration of the temperature 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 (see Chapter 3.2.3.).

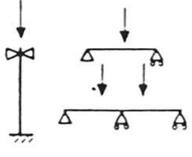
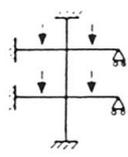
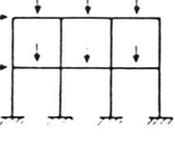
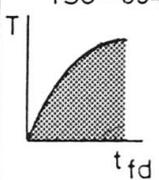
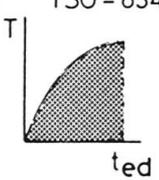
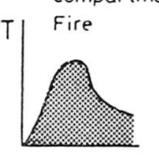


(H₃) A rise of temperature as a function of time characterized by an analytical determination of the gas temperature-time curve of a fully developed compartment fire.

Structural models

- (S₁) The load bearing structure is composed of 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 composed of 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 the norm.
- (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 an analytical design.

A Classification System for Methods of Structural Fire Engineering Design

Structural Model Heat Exposure Model		S ₁	S ₂	S ₃
		Elements	Sub-assembly	Structures
				
H ₁	ISO-834 	test or calculation (deterministic)	calculation occasional test (deterministic)	difference in schematization becomes too large
H ₂	ISO-834 	test or calculation (probabilistic)	calculation occasional test (probabilistic)	calculation (probabilistic) unpractical
H ₃	compartment Fire 	calculation (probabilistic) occasional	calculation (probabilistic)	calculation (probabilistic) occasional and for research

t_{fd} = required time of fire duration

t_{ed} = design equivalent time of fire exposure

Figure 1 : Matrix of heat exposure and structural models in sequence of improved idealization.



2.3.3. Combinations of heat exposure and structural models

In the table of fig.1 the heat exposure models and structural models are combined in a matrix in sequence of improved idealization. In principle, each element in the matrix represents a particular design procedure. The matrix therefore can be considered as a classification system for methods of structural fire engineering design. It is evident that not all models can be used in all combinations and the rule should be to provide a sensible pairing at each level of advancement. In the text of figure 1 reference is made to these aspects. In principle, a differentiated fire engineering design offers problem-oriented choice for the combination of heat exposure model and structural model.

The final choice may also depend on national preferences, the simplicity of application and on the particular design situation (14,15) :

The design method $H_1 - S_1$ and occasionally $H_1 - S_2$ with experimental verification of the fire resistance, corresponds to the vast majority of national building codes (see § 2.3.1.). In many countries improved methods based on heat exposure models H_2 and H_3 (8,9,10,13) have occasionally been used, but, except in Sweden, they are not yet 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. Recently the Fire Committee of the European Convention for Constructional Steelwork (ECCS) completed Recommendations providing a reference document for national codes of practice (11).

3. STATE OF THE ART IN STRUCTURAL FIRE CALCULATION METHODS

3.1 Limit state condition

Generally, the design criterion in a fire design requires that no limit state is reached during the fire exposure. Depending on the type of practical application, one, two or all of the following limit state conditions apply :

- limit state with respect to load bearing capacity.
- limit state with respect to insulation.
- limit state with respect to integrity.

For a load bearing structure, the design criterion implies that the minimum value of the load bearing capacity $R(t)$ during the fire exposure shall meet the load effect on the structure S , i.e. :

$$\min \{ R(t) \} - S \geq 0 \quad [3.1]$$

The criterion must be fulfilled for all relevant types of failure - bending failure, shear failure, torsion failure, instability failure, etc...

For a separating structure, the design criterion with respect to insulation can be formulated analogously as :

$$T_{cr} - \max \{ T_s(t) \} \geq 0 \quad [3.2]$$

where

T_{cr} = maximum temperature of the unexposed side of the separating structure, acceptable with respect to the requirement to prevent a fire spread from the fire compartment to an adjacent compartment, and

$T_s(t)$ = highest temperature on the unexposed side of the separating structure at time t of the relevant fire process.

For the requirement with respect to integrity, which can be decisive for some types of separating elements - for instance, doors - there is no analytically expressed design criterion available at present.

In the form given by Eqs. [3.1] and [3.2], the design criteria are directly adapted to structural fire design methods, based on the characteristics of the natural fire exposure - heat exposure model H_3 according to fig.1. In fire design methods, based on a thermal exposure according to the standard temperature-time curve as specified in ISO 834 - heat exposure models H_1 and H_2 - the time to reach the decisive limit state defines the fire resistance of the structural element t_{fr} and, consequently, the design criterion is transferred to the alternative form:

$$t_{fr} - t_{fd} \geq 0 \quad [3.3]$$

where t_{fd} is the required fire resistance or time of fire duration, specified in the building codes and regulations.

The design criterion then applies to load bearing as well as separating structural elements.

3.2 Type of physical model and related fire exposure

3.2.1 Exposure according to standard temperature-time curve

As discussed in 2.3.1, virtually all countries use a fire engineering design procedure for structural elements based on classification and standard fire resistance test according to ISO 834 (with fixed heating conditions). In the design, the results of such fire resistance tests are compared directly with the requirements given by the building codes and regulations. Fig.2 illustrates this design procedure.

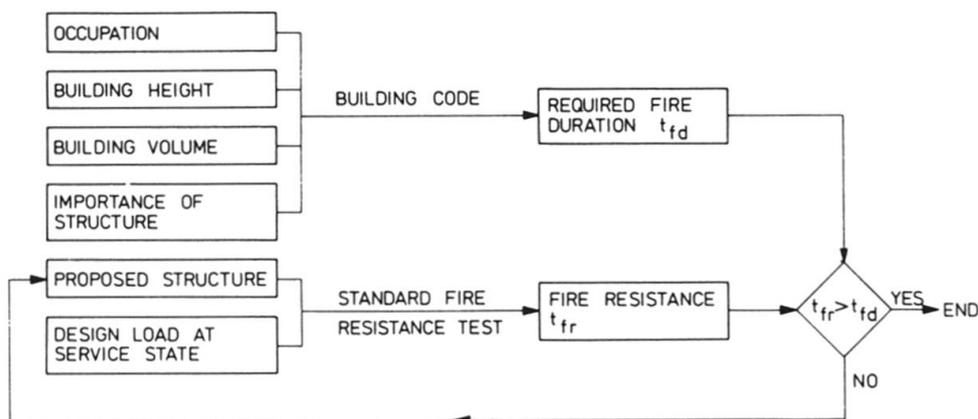


Figure 2 : Structural fire engineering design procedure used in most countries based on classification and results of standard fire resistance tests.

In the fire resistance 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 (5) - heat exposure model H_1

$$T - T_0 = 345 \log_{10}(8t + 1) \quad [3.4]$$

where

t = time, in minutes,

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

T_0 = furnace temperature at time $t = 0$, in $^{\circ}\text{C}$.

The important progress, made during the last ten years, in the development of computation methods for an analytical structural fire engineering design gives the opportunity for fire resistance to be determined by calculation for many practical applications. Consequently, more and more countries now permit a classification of load bearing structural elements with respect to fire to be formulated analytically, as an alternative to testing. This leads to a design procedure as shown in fig.3 (16).

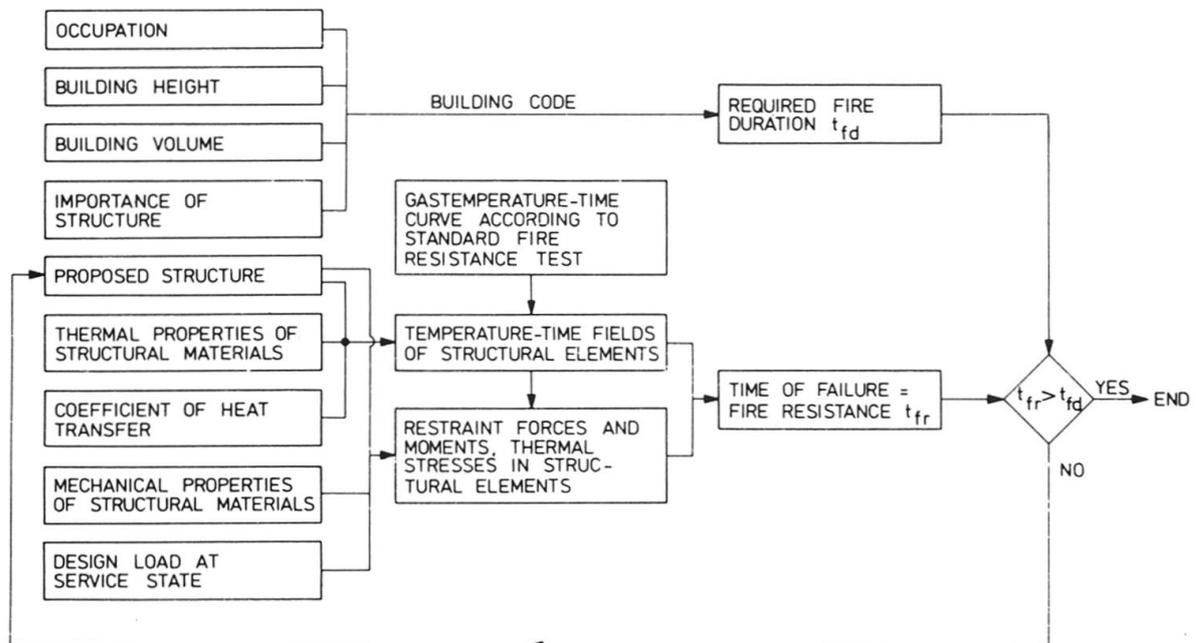


Fig. 3 : Analytical fire engineering design of load bearing structural elements, based on classification and thermal exposure according to the standard temperature-time curve, Eq. [3.4]

With the gas temperature-time curve according to Eq. [3.4] as thermal exposure, the temperature-time fields of the structural element can be calculated, using (a) the structural characteristics of the proposed structure, (b) the thermal properties of the structural materials, and (c) the coefficients of heat transfer for the various surfaces of the structure as further input data. Introducing (d) the mechanical properties of the structural materials, and (e) the load characteristics, the time variation of the restraint forces and moments, thermal stresses and load bearing capacity can then be determined. The time at which the load bearing capacity has decreased to the level of the design load at service state defines the time of failure or the fire resistance t_{fr} , and the design criterion to be satisfied is that $t_{fr} \geq t_{fd}$ - cf. Eq. [3.3].

3.2.2 Natural fire exposure

In applying data on the fire resistance of structural elements in practice, it is important to consider that the standard fire resistance test - whether experimental or calculated - 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 corresponding calculations do is to grade structural elements and the building codes and regulations, then require different grades of element according to the circumstances.

These deficiencies have given rise to the development of analytical structural fire design methods, based on the characteristics of natural compartment fires and on well-defined functional requirements and performance criteria. Such analytical design methods have now reached a comparatively advanced level, especially as far as fire exposed steel structures are concerned. To aid the practical application, design diagrams and tables have been systematically produced and published, giving directly, on the one hand, the temperature of the fire exposed structure, and on the other, a transfer of this information to the corresponding load bearing capacity of the structure - cf. , for instance (8), (11), (17), (20), (23).

In general, the design methods fall into two groups with respect to the use of the basic data of the compartment fire. The methods of the first group are characterized by a design procedure, based directly on differentiated gas temperature-time curves of the complete process of a natural fire development - heat exposure model H_3 . The characteristic of the methods of the second group is a design procedure with the varying properties of a natural fire development taken into account over an equivalent time of fire exposure, related to the heating according to the standard temperature-time curve - heat exposure model H_2 .

The physical model for a structural fire engineering design, based on the heat exposure model H_3 , is shown summarily in fig. 4, for a load bearing structure.

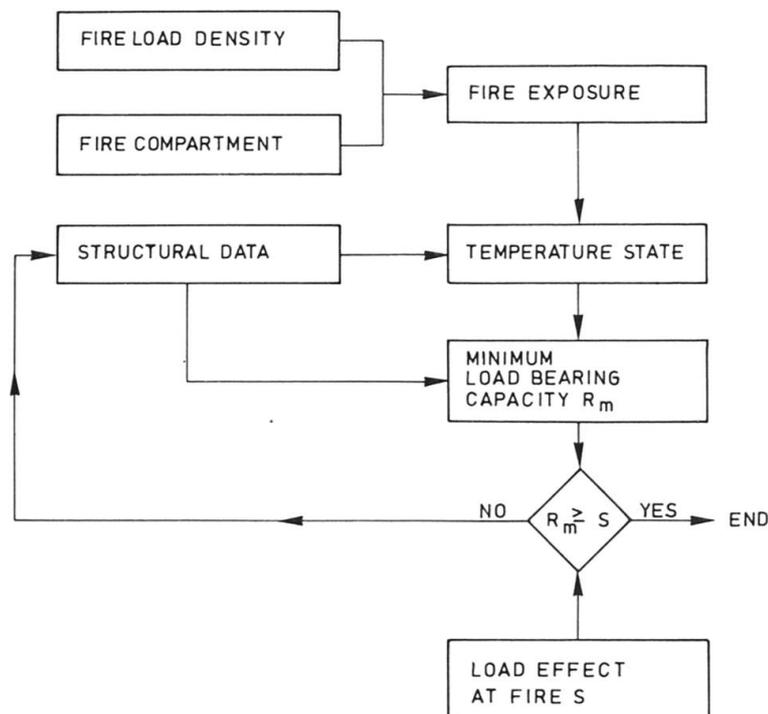


Figure 4 : Physical model for an analytical fire engineering design of load bearing structures, based directly on the characteristics of the natural compartment fire-heat exposure model H_3 .

The design procedure starts by a determination of the fire exposure, given by, for instance, the gas temperature-time curve of the natural compartment fire. In the individual practical application, the fire exposure then can be obtained either by heat and mass balance calculations for the fire compartment cf., (8), (20), (24) to (30) or directly from a systematized design basis of the type exemplified by fig. 5 (8), (20), (25).

The combustion characteristics of the fire load and the geometrical, ventilation and thermal properties of the fire compartment are the important factors.

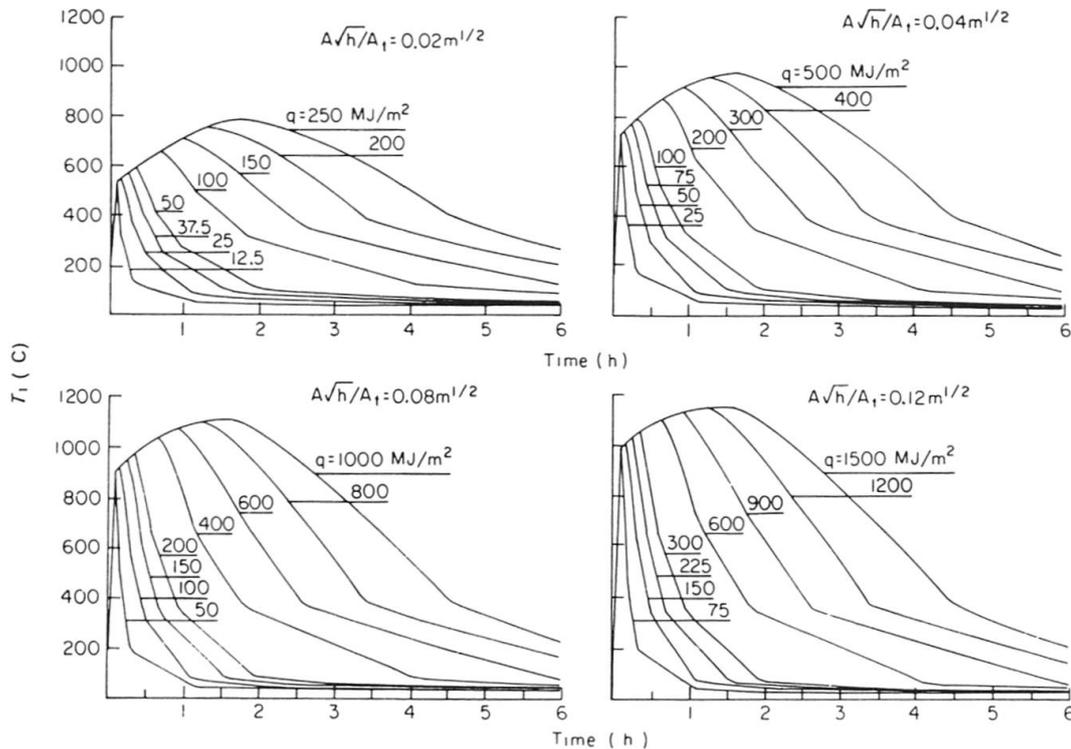


Figure 5 : Gas temperature-time curves for a complete, fully developed compartment fire with varying values for the fire load density q and the opening factor $A\sqrt{h}/A_t$. A is the total opening area of the fire compartment, h is a weighted mean value of the height of the openings based on their size, and A_t is the total internal surrounding area of the fire compartment, including openings. Fire compartment, type A (8), (20), (25).

The gas temperature-time curves $T_t - t$ in fig.5 apply to a fire compartment with surrounding structures, made of a material with a thermal conductivity $\lambda = 0.81 \text{ W}\cdot\text{m}^{-1}\cdot\text{°C}^{-1}$ and a heat capacity $\rho C_D = 1.67 \text{ MJ}\cdot\text{m}^{-3}\cdot\text{°C}^{-1}$, fire compartment type A. Such a surrounding material corresponds roughly to an average of brick, concrete, and aerated concrete. For fire compartments with surrounding structures, whose thermal properties deviate from compartment type A, the actual fire process can be transferred to a gas temperature-time curve for fire compartment type A by using an effective fire load density, q_f , and an effective opening factor $(A\sqrt{h}/A_t)_f$, calculated from the real fire load density q and the real opening factor $A\sqrt{h}/A_t$ according to the formulae :

$$q_f = K_f q ; (A\sqrt{h}/A_t)_f = K_f A\sqrt{h}/A_t \quad [3.5]$$

In (8), (17), and (20), the coefficient K_F is given for seven types of fire compartments defined by their surrounding structures.

The fire load density q is given by the relationship :

$$q = \frac{1}{A_t} \sum \mu_v m_v H_v \quad [3.6]$$

where :

m_v = total mass of combustible material v (kg)

H_v = calorific value of combustible material v
(MJ.kg⁻¹)

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

A_t = total interior area of the surfaces bounding the fire compartment, including all openings (m²).

In the opening factor of the fire compartment $A\sqrt{h}/A_t$

A = total area of door and window openings (m²), and

h = mean value of the heights of the openings, weighted with respect to each individual opening area (m).

The gas temperature-time curves according to fig. 5 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, industrial buildings and sport halls - the curves give an unsatisfactory description of the real fire exposure. At present, there is no validated design basis available for the determination of the fire exposure in compartments with a very large volume.

Returning to the physical model, as shown in fig. 4, in the next step, the fire exposure is transferred analytically to transient temperature fields in the exposed structure and then a determination is carried out of the time variation of the load bearing capacity $R(t)$.

A comparison between the minimum value R_m of the load bearing capacity $R(t)$ during the relevant fire process and the load effect at fire S decides whether the structure can fulfil its required function or not during the fire exposure, as specified by the limit state condition according to Eq. [3.1].

For a separating structure, the physical model gives the transient temperature state, defining the maximum value, $\max \{ T_S(t) \}$, of the highest temperature on the unexposed side of the structure during the relevant fire process. The corresponding limit state condition follows Eq. [3.2] with respect to the required function of insulation. The supplementary limit state condition regarding the integrity function has to be proved experimentally, when required, in either a fire resistance test or a simplified small scale test.

3.2.3. Equivalent time of fire exposure

The design scheme for a fire engineering design of a load bearing structural element, based on the heat exposure model H_2 , is illustrated in fig. 6 (14). The design comprises a determination of the ultimate state of the structural element on one hand for a natural fire exposure, on the other hand for a thermal exposure according to the standard fire resistance test, ISO 834 - Eq. [3.4].

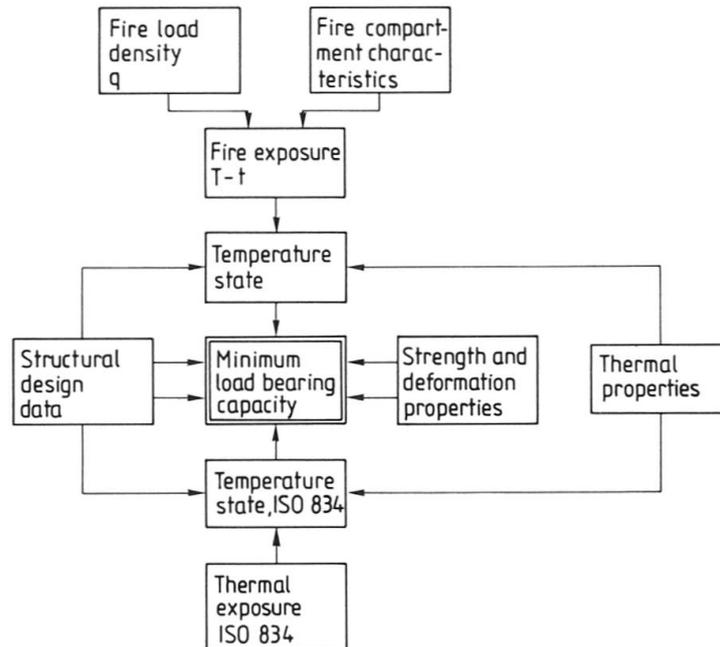


Figure 6 : Procedure for a determination of the equivalent time of fire exposure.

For the two types of exposure, the temperature state and the related load bearing capacity are determined for the structural element. Input information is data on the structural design and the thermal, strength and deformation properties of the structural materials. The minimum load bearing capacity of the structural element during the relevant natural compartment fire, put equal to the minimum load bearing capacity at the thermal exposure according to the standard fire resistance test, gives the equivalent time of fire exposure t_e . The minimum load bearing capacity may be defined by a critical value of a maximum deflection, or a maximum rate of deflection or a maximum temperature.

To be precise, the equivalent time of fire exposure t_e depends not only on the parameters influencing the natural compartment fire, but also on a number of structural parameters. For fire exposed steel structures, (8), (20), and (31) give a design basis which facilitates a practical approach to determining this form of the equivalent time of fire exposure.

More roughly, t_e can be described as dependent only on factors affecting the compartment fire according to the following approximate formula (31) :

$$t_e = 0.067 \frac{q_f}{(A\sqrt{h}/A_t)^{0.5}} \text{ (min)} \quad [3.7]$$

verified for application to unprotected and protected steel structures. In the formula, q_f is the effective fire load density per unit area of the surfaces bounding the fire compartment ($\text{MJ}\cdot\text{m}^{-2}$) and $(A\sqrt{h}/A_t)_f$ the effective opening factor of the fire compartment ($\text{m}^{1/2}$), calculated according to Eqs. [3.5] and [3.6]. Written in this form, the formula enables the influence of varying thermal properties of the surrounding structures of the fire compartment to be taken into account.

The formula has the same limitations with respect to the size of fire compartment as stated in 3.2.2. for the gas temperature-time curves according to fig.5.

The design criterion in a fire engineering design based on the heat exposure model H_2 is that the fire resistance of the structural element t_{fr} shall meet the required fire resistance, expressed as the equivalent time of fire exposure t_e , i.e. - cf. Eq. [3.3].

$$t_{fr} - t_e \geq 0 \quad [3.8]$$

The fire resistance t_{fr} then can be obtained either experimentally by standard fire resistance tests according to ISO 834 - or a corresponding national standard - or by calculation.

3.3. A probability based structural fire engineering design

The modern development of functionally well-defined, analytical structural fire design methods includes a probabilistic approach, based on either a system of partial safety coefficients or the safety index concept (9), (10), (14), (32), (33).

A probability based structural fire design should originate from validated models, describing the relevant physical processes and strictly specifying the connected uncertainties and reliability models. Only design methods, based on the heat exposure models H_2 and H_3 , fulfil these requirements from a conceptual point of view.

For the probabilistic model to be integrated with the physical model, various levels of ambition can be distinguished :

- an exact evaluation of the failure probability $P(R < S)$ using multi-dimensional integration or Monte Carlo simulation,
- an approximate evaluation of the failure probability $P(R < S)$ based on first order reliability methods (FORM), and
- a practical design format calculation, based on partial safety factors and taking into account characteristic values for action effects and response capacities.

For practical purposes, an exact evaluation of failure probability is not possible. Also, the FORM approximations are too cumbersome for everyday design and more simplified practical design formats have to be used.

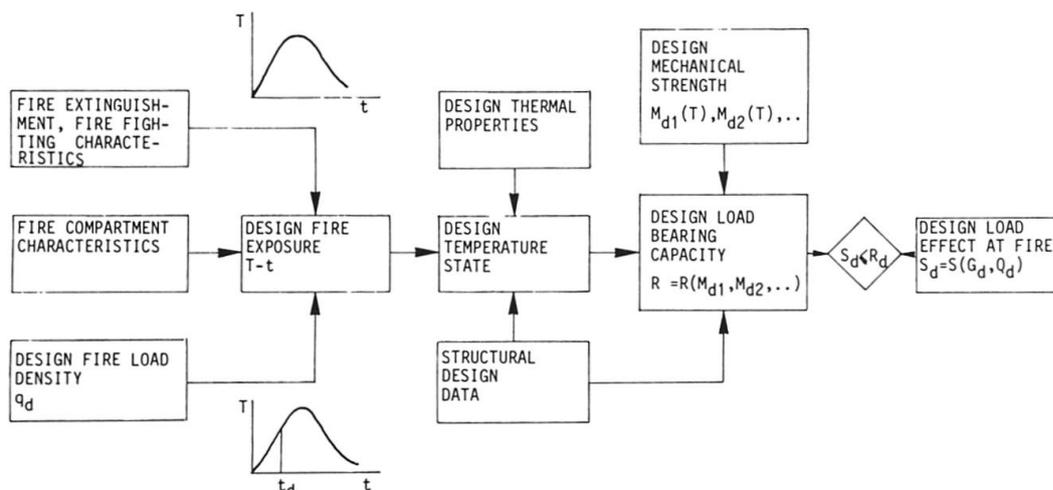


Figure 7: Procedure for a practical design format calculation of a load bearing structure, exposed to a natural compartment fire - heat exposure model H_3 .

Fig. 7 summarises a practical design format calculation for a fire exposed load bearing structure, using the heat exposure model H_3 (14), (32) to (34).



From the design fire load density q_d and the geometrical, ventilation and thermal characteristics of the fire compartment, the design fire exposure is determined, given as the gas temperature-time curve $T-t$ of the fully developed compartment fire and obtained either from a systematized design basis or by heat and mass balance calculations.

Together with the structural design data, the design thermal properties and the design mechanical strength of the structural materials, the design fire exposure provides the design temperature state and the related design load bearing capacity R_d for the lowest value of the load bearing capacity during the relevant fire process.

A direct comparison between the design load effect at fire S_d finally establishes whether or not the structure can fulfil its required function on exposure to fire, i.e the design format condition to be proved is

$$R_d - S_d \geq 0 \quad [3.9]$$

Depending on the type of practical application, the fulfilment of the condition has to be verified for either the complete fire process or a limited part of it t_d , determined by the time necessary for the fire to be extinguished under the most severe conditions or by the design evacuation time for the building.

The probabilistic influences are taken into account by specifying characteristic values and related partial safety factors for the fire load density, such structural design data as imperfections, the thermal properties, the mechanical strength and the loading. The partial safety factors are then derived by a probabilistic analysis, based on a first order reliability method (FORM), with the following effects and influences taken into consideration.

- the uncertainty in specifying the fire load density,
- the uncertainty in specifying the ventilation characteristics of the fire compartment and the thermal properties of the structures surrounding the fire compartment,
- the uncertainty of the analytical model for the determination of the compartment fire and its thermal exposure on the structure,
- the uncertainty in specifying the design data of the structure, dimensions, positions of reinforcement, imperfections, etc... ,
- the uncertainty in specifying the thermal and mechanical properties of the structural materials,
- the uncertainties of the analytical models for the calculation of the heat transfer to and within the structure and the ultimate load bearing capacity of the structure,
- the uncertainty in specifying the loads,
- the uncertainty of the model, describing the load effect on the structure,
- the probability of occurrence of a fully developed compartment fire,
- the efficiency of the fire brigade actions,
- the effect of an installed extinction system, and
- the consequences of a structural failure.

For a structural fire engineering design, based on the heat exposure model H_2 , the practical design format can be given in the following form (14) :

$$\frac{t_{fr}}{\gamma_f} \geq \gamma_{n1} \gamma_{n2} \gamma_e t_e \quad [3.10]$$

in which t_{fr} is the fire resistance of the structural element, t_e equivalent time of fire exposure according to Eq. [3.7], and γ_f , γ_{n1} , γ_{n2} and γ_e partial safety factors, taking into account all uncertainties in the design system.



The partial safety factor γ_e covers the uncertainties of the fire load density and the fire compartment characteristics, including the uncertainties of the analytical models for a determination of the fire exposure. The partial safety factor γ_f considers 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 transient temperature state and the load bearing capacity if the fire resistance is evaluated analytically. The additional partial safety factors γ_{n1} and γ_{n2} include the effect of the probability of a fully developed compartment fire and the consequences of a structural failure. Then γ_{n2} is a partial safety factor due to average reliability requirements, and γ_{n1} a correction factor due to deviations from average reliability requirements, correcting for instance, for the effect of a sprinkler system or the efficiency of the fire brigade actions.

3.4. Temperature distribution in structural steel elements at fire exposure

For a fire exposed, uninsulated steel structure, the energy balance equation gives the following formula for a determination of the steel temperature-time curve $T_s - t$ (fig. 8).

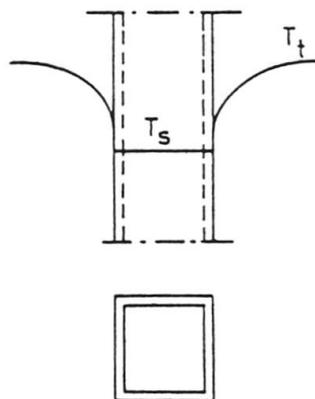


Figure 8 : Fire exposed, uninsulated steel structure.
 T_t = gas temperature within fire compartment, T_s = steel temperature at time t.

$$\Delta T_s = \frac{\alpha}{\rho_s C_{ps}} \frac{F_s}{V_s} (T_t - T_s) \Delta t \quad (^\circ\text{C}) \quad [3.11]$$

where

ΔT_s = change of steel temperature ($^\circ\text{C}$) during time step Δt (s)

α = coefficient of heat transfer at fire exposed surface of structure ($\text{W}\cdot\text{m}^{-2} \quad ^\circ\text{C}^{-1}$),

ρ_s = density of steel material (7850 kg m^{-3}),

C_{ps} = specific heat of steel material ($\text{J}\cdot\text{kg}^{-1} \quad ^\circ\text{C}^{-1}$),

F_s = fire exposed surface of steel structure per unit length (m)

V_s = volume of steel structure per unit length (m^2),

T_t = gas temperature ($^\circ\text{C}$) within fire compartment at time t (s).



Eq. [3.11] presupposes that the steel temperature T_s is uniformly distributed over the cross section of the structure at any time t .

The coefficient of heat transfer α can be calculated from the approximate formula

$$\alpha = 23 + \frac{5.77 \varepsilon}{T_t - T_s} \left[\left(\frac{T_t + 273}{100} \right)^4 - \left(\frac{T_s + 273}{100} \right)^4 \right] \quad (\text{W/m}^2 \cdot \text{°C}^{-1}) \quad [3.12]$$

giving an accuracy which is sufficient for ordinary practical purposes.

ε is the resultant emissivity which for practical applications can be chosen according to the following table, giving values which are generally on the safe side.

1. Column, fire exposed on all sides	$\varepsilon = 0.7$
2. Column, outside a facade	$= 0.3$
3. Floor structure, composed of steel beams with a concrete slab on the lower flange of the beams	$= 0.5$
4. Steel beams with a floor slab on the upper flange of the beams	
a) Beams of I cross-section with width/height ≥ 0.5	$= 0.5$
b) Beams of I cross-section with width/height < 0.5	$= 0.7$
c) Beams of box cross-section and trusses	$= 0.7$

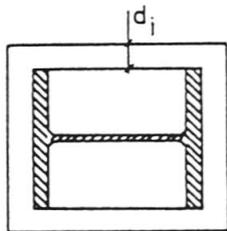
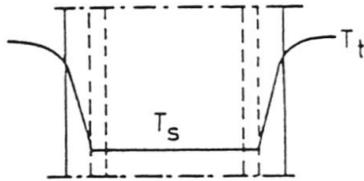
In (8), (17), (20), more accurate values are given for ε in the case 4 of application.

For a given gas temperature-time curve $T_g - t$, the steel temperature T can be directly calculated from Eqs. [3.11] and [3.12] with allowance for the temperature dependence of C_{ps} and α . Such computations have been carried out in a systematized way giving design tables as published in (11), (18), (19), (22), (23), for a thermal exposure according to the standard temperature-time curve and in (8), (17), (20), for a natural compartment fire exposure according to fig.5. The first set of tables give the steel temperature as a function of the time of exposure t for varying values of F_s/V_s ratio and the resultant emissivity ε . From the second set of tables, the maximum steel temperature $T_{s,max}$ during a complete compartment fire can be determined directly as a function of the effective fire load density q_f , the effective opening factor $(A\sqrt{h}/A_t)_f$, the F_s/V_s ratio and the resultant emissivity ε .

Similarly for a fire exposed insulated steel structure, a simplified energy balance equation gives the following formula for a direct determination of the steel temperature-time curve $T_s - t$ (fig.9).

Figure 9 : Fire exposed, insulated steel structure

 T_t = gas temperature within the fire compartment

 T_s = steel temperature at time t .


$$T_s = \frac{A_i}{(1/\alpha + d_i/\lambda_i) \rho_s C_{ps} V_s} (T_t - T_s) \Delta t \quad (^\circ\text{C}) \quad [3.13]$$

with the additional quantities

 A_i = internal enclosing surface area of insulation per unit length (m)

 d_i = thickness of insulation (m),

 λ_i = thermal conductivity of insulating material ($\text{W}\cdot\text{m}^{-1}\cdot^\circ\text{C}^{-1}$)

Eq. [3.13] presupposes that the steel temperature T_s is uniformly distributed over the cross-section of the structure at any time t , that for the insulation the temperature gradient is linear and the heat absorption negligible and that the heat transfer is one-dimensional.

Computations, originating from Eqs. [3.12] and [3.13] provide a systematized design basis for a practical fire design. Such a design basis is published in (11), (18), (19), (22), (23), for a thermal exposure according to the standard temperature-time curve, giving the steel temperature as a function of the time of exposure t for varying values of the A_i/V_s and d_i/λ_i ratios. (8), (17), (20) include a corresponding design basis for a natural compartment fire exposure giving the maximum steel temperature $T_{s,\max}$ for varying values of the effective fire load density $q_{f,e}$, the effective opening factor $(A\sqrt{h}/A_t)_f$ and the A_i/V_s and d_i/λ_i ratios.

For a specific insulating material, systematized design diagrams or tables can be computed very accurately with regard to the temperature dependence of the thermal properties of the steel as well as the insulating material. The influence of an initial moisture content and of a disintegration of the insulating material can be considered, too. Practically, such a determination can be carried out over a numerical data processing by computers on the basis of a finite difference or a finite element method. A great number of design tables computed according to such an accurate procedure, are presented in (8).

The design basis referred to generally assumes the steel temperature to be uniformly distributed over the cross-section of the beam or column at any time t . A more accurate theory which enables a determination of the temperature variation over the cross-section of the steel structure, is presented in (35), (36), together with computer routines. The algorithm described can easily be coupled to most finite element programs.

An illustration of the capability of the theory is given in fig. 10, which shows calculated temperature distribution along the line of symmetry of a gypsum insulated steel beam with a concrete slab at the top flange, at selected times of thermal exposure according to the standard temperature-time curve.

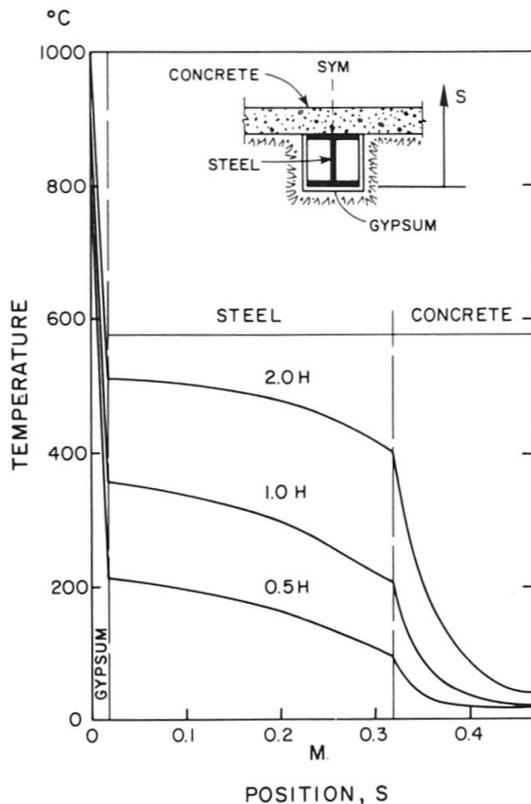


Figure 10 : Calculated temperature distribution along line of symmetry of a steel beam, insulated by a 16 mm gypsum board (density 770 kg.m^{-3}) and carrying a 150 mm concrete slab on top flange, at selected times of a thermal exposure according to the standard temperature-time curve, Eq. [3.4]- (35), (36).

3.5 Load bearing capacity of steel structures at fire exposure

A transformation of the transient temperature state of a fire exposed structure or a structural element to data on the structural behaviour and load bearing capacity requires access to validated mathematical models of the mechanical behaviour of the structural material in the temperature range associated with fires.

For steel, such models have been available for many years - cf., for instance, (37) to (40). The models decompose the total strain into thermal strain, instantaneous elastic and plastic strain, and time and temperature dependent creep strain. Some of the models operate with temperature compensated time t_T according to DORN (37), defined by the formula

$$t_T = \int_0^t e^{-\Delta H/R\bar{T}} dt \quad [3.14]$$

where

- H = activation energy required for creep (J.mol^{-1})
- R = universal gas constant ($\text{J.mol}^{-1} \cdot \text{K}^{-1}$) and
- \bar{T} = absolute temperature (K)

Analytical models for a determination of the mechanical behaviour and load bearing capacity of fire exposed isostatic and hyperstatic steel beams, columns and frames are presented in, for instance, (38) - (43). The most general method is the one described in (40), based on a finite element elastic-plastic-creep analysis including the influence of geometrical non-linearities of the structure.



A simplified design basis, giving directly the load bearing capacity for a design load effect can be found in (8), (11), (17) to (23). The design basis can be used for the thermal exposure given by the standard temperature-time curve or the natural fire concept. The design basis is illustrated by figs. 11, 12 and 15. Figures 11 and 12, (8), (20), give the load bearing capacity (M_{kr} , P_{kr} , q_{kr}) of fire exposed beams of constant I cross section at different types of loading and support conditions, as a function of the steel beam temperature T_s . The design curves in fig. 11 apply to a slow rate of heating - assumed to be $4^\circ\text{C}\cdot\text{min}^{-1}$, followed by a cooling with a rate of $1.33^\circ\text{C}\cdot\text{min}^{-1}$ - and fig.12 gives the correction $\Delta\beta$ of the load-bearing capacity coefficient β due to a more rapid rate of heating. In the formula for load-bearing capacity

σ_s = yield stress of steel material at room temperature (MPa),

L = span of beam (m)

W = elastic modulus of beam cross section (m^3)

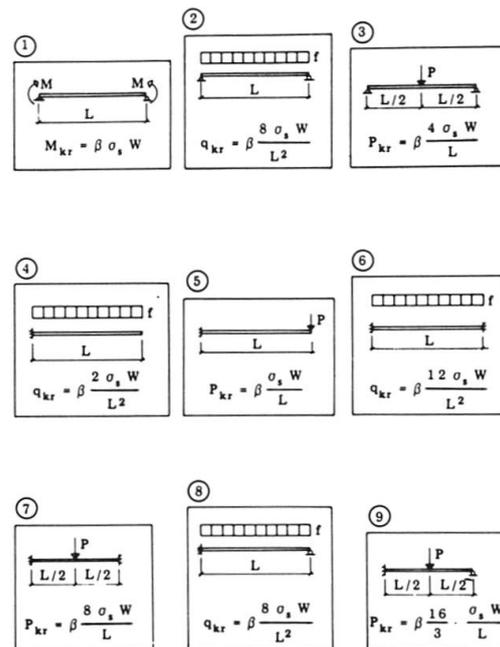
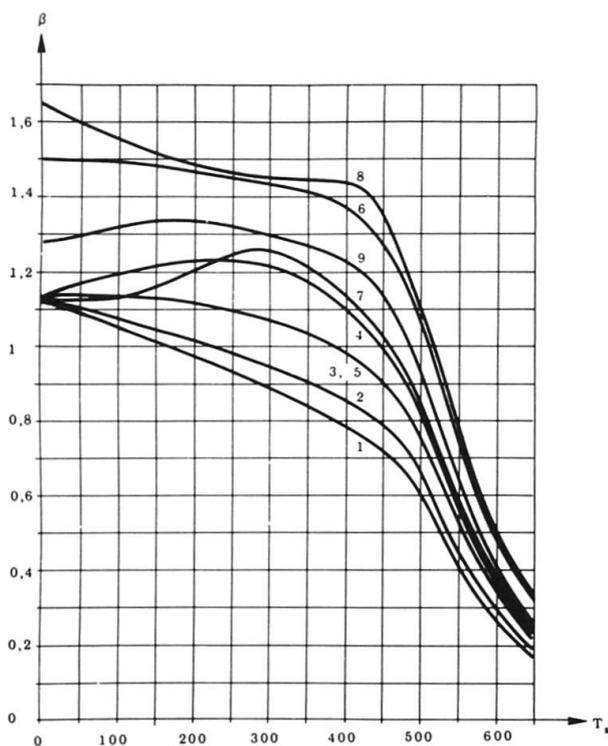


Figure 11 : Coefficient β for determination of critical load (M_{kr} , P_{kr} , q_{kr}) for fire exposed steel beams of I cross section at different types of loading and support conditions as a function of the steel beam temperature T_s . The curves have been calculated for a slow rate of heating of $4^\circ\text{C}\cdot\text{min}^{-1}$ and a subsequent cooling, assumed to be one third of the rate of heating (8), (20).



The design curves in figures 11 and 12 have been determined on the basis of the deformation curve of the fire exposed beams calculated by an analytical model, presented in (38), which takes into account the softly rounded shape of the shape of the stress-strain curve of steel at elevated temperatures as well as the influence of creep strain. As can be seen from fig. 12, this influence of creep begins to be noticeable for ordinary structural steels at temperatures in excess of about 450 °C.

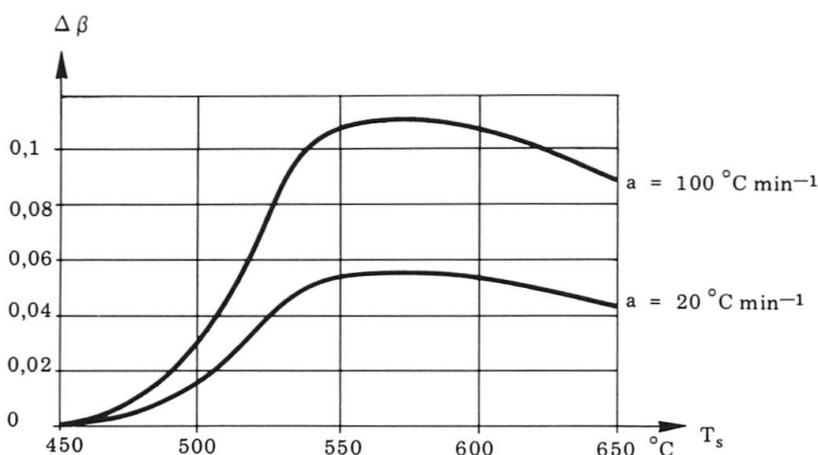


Figure 12 : Increase $\Delta\beta$ of coefficient β determined according to fig. 11, for a rate of heating $a \geq 4^\circ\text{C} \cdot \text{min}^{-1}$, as a function of the steel beam temperature T_s (8), (20).

In the European Recommendations for the fire safety of steel structures (11), an alternative simplified approach is given for the determination of the load bearing capacity of a steel structure at uniform elevated temperature T_s . The elementary theory of plasticity is directly applied, related to an effective yield stress σ_{s,T_s} in which the influence of creep is included implicitly.

The basic stress-strain curves are exemplified in fig. 13 for the steel grade Fe 360. The large gap between the curves for 200 and 300°C is due to the so-called "thermally activated flow" (41). For an ultimate limit state design, the curves are cut off at certain stress levels, defining the effective yield stress σ_{s,T_s} as a function of the steel temperature T_s - fig.14.

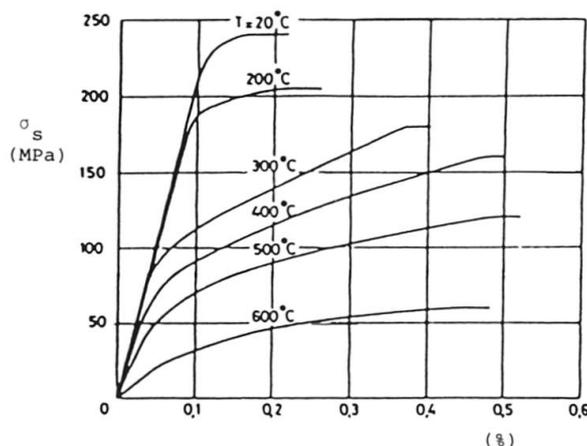


Figure 13 : Stress-strain curves at elevated temperatures T_s for steel grade Fe 360. In the curves, the influence of creep is included implicitly (11).

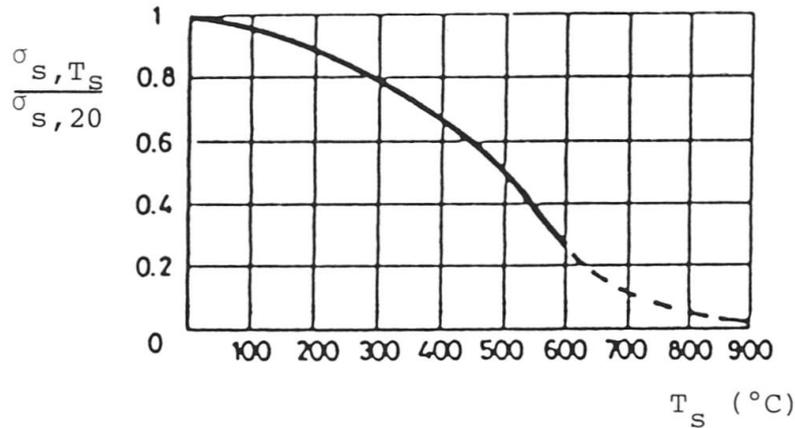


Figure 14 : Quotient between effective yield stress T_{s, T_s} at elevated temperature T_s and yield stress at room temperature as a function of steel temperature T_s . The curve applies to steel grades Fe 360 to Fe 510 with an accuracy, which is sufficient for practical purposes (11)

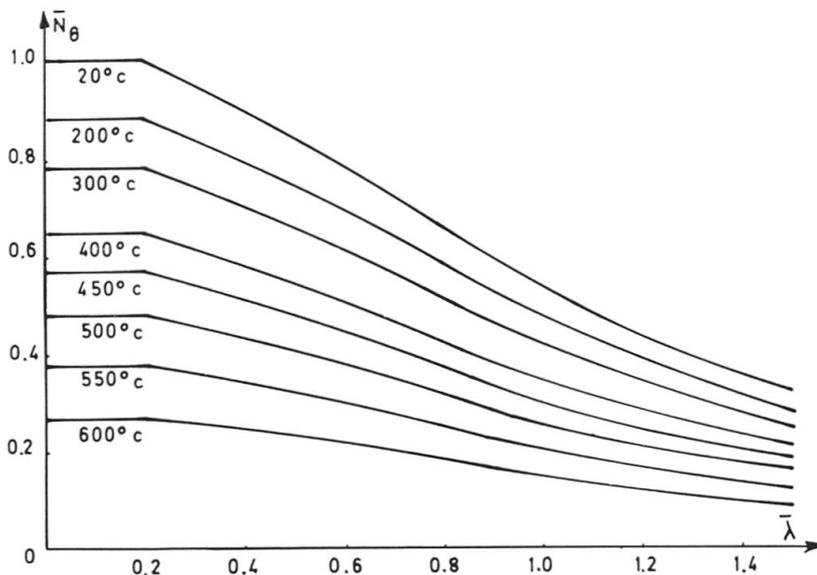


Figure 15 : Relationship between non-dimensional buckling load \bar{N}_θ and slenderness factor $\bar{\lambda}$ at varying steel temperature T_s for axially compressed steel columns (11), (44).

The curves in fig. 15 (11), (44) give the variation with the steel temperature T of the non-dimensional buckling load \bar{N}_θ for axially compressed columns as a function of the slenderness factor

$$\bar{\lambda} = \frac{\lambda}{\pi \sqrt{E/\sigma_s}} \quad [3.15]$$

where

λ = column slenderness ratio

E = modulus of elasticity at room temperature, and

σ_s = yield strength at room temperature

The curves are experimentally validated by tests made recently in several European countries. The curves are applicable under the presumption that the column is unrestrained with respect to longitudinal expansion during the fire exposure. For a fire design of columns, partly restrained to longitudinal expansion, see reference (8).

3.6 Consistency between analytical and experimental approaches

The analytical determination of the fire resistance of load bearing structural elements as an alternative to testing has raised a problem of inconsistency, recently analysed in (45), as concerns steel structures.

Due to high costs, a fire resistance test is usually limited to one test specimen - in a few countries to two test specimens. For a single test specimen, the actual material quality represents a random sample from a wide variety. Consequently, a standard fire resistance test is generally carried out on a test specimen with a load bearing capacity which is greater - most often significantly greater - than the load bearing capacity related to the characteristic value of the material strength. In current practice, no corrections are made of the test results with respect to this.

An analytical determination of the load bearing capacity of a structural element is based on the characteristic value of the material strength. This gives an analytically determined fire resistance which is lower - normally significantly lower - than the corresponding value derived from a standard fire resistance test.

Simplified methods for a calculation of the temperature of fire exposed steel structural elements are, as a rule, based on the assumption of a uniformly distributed temperature over the cross section and along the structure at each time of fire exposure. In certain types of steel structures, for example beams with a slab on the upper flange, a considerable temperature variation arises over the cross section as well as in the longitudinal direction during a fire resistance test. A simplified method, which neglects this influence gives a further underestimation of the fire resistance in relation to the corresponding result obtained in a fire resistance test.

In (45) alternative methods of correction are outlined for obtaining better agreement between the analytical and experimental approaches. One of these methods is developed further to a design basis that can be applied easily in practice. In principle, the method implies that the analytically determined load bearing capacity R is multiplied by a correction factor f , which is a function of the uniformly distributed calculated steel temperature T_s and the type of structural element. Fig. 16 gives the correction factor derived and practically applied in the ECCS Recommendations for the fire safety of steel structures (11). The method of correction is a rough approach and should be seen as a temporary solution of the problem.

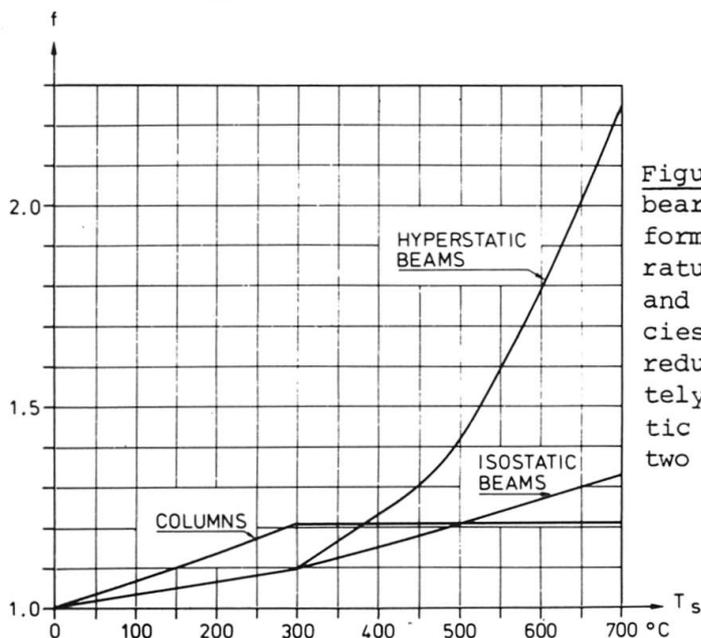


Figure 16 : Correction factor f for load bearing capacity R as a function of uniformly distributed calculated steel temperature T_s for columns, isostatic beams, and hyperstatic beams with two redundancies. For hyperstatic beams with only one redundancy, f can be chosen as approximately the average of the values for isostatic beams and hyperstatic beams with two redundancies (45).

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