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Autor: Witteveen, Jelle

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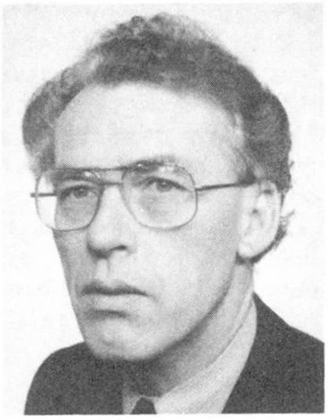
Design Methods for Fire exposed Steel Structures

Méthodes de calcul au feu des constructions métalliques

Methoden für die rechnerische Beurteilung des Brandverhaltens von Bauteilen

Jelle WITTEVEEN
Director
Institute TNO
Rijswijk, the Netherlands



Jelle Witteveen, born in 1933, received his civil engineering degree at Delft University of Technology. His research activities include studies of plastic behaviour of steel structures, policies for fire safety and fire engineering design.

SUMMARY

During the last decade much progress has been made in the analytical modelling of fire exposure and in the development of probabilistic methods of fire risk assessment. Analytical methods have been developed for the determination of the load bearing capacity of elements and structures at elevated temperatures as an alternative to the standard fire resistance test. This paper reviews design methods for structural fire safety, in particular for steel structures, which were developed in the last decade and are now being used and implemented in building regulations and structural codes.

RÉSUMÉ

Dans la dernière décade, un grand progrès a été fait dans la modélisation de l'exposition au feu et le développement de méthodes probabilistes pour l'évaluation du risque d'incendie. Des méthodes analytiques ont été développées pour la détermination de la capacité portante aux températures élevées d'éléments et de structures comme alternatives au test de résistance au feu standard. La présente contribution passe en revue les méthodes de calcul de la sécurité structurale à l'incendie, en particulier pour les structures métalliques; ces méthodes, développées durant la dernière décennie permettent de traiter des applications pratiques et d'améliorer les prescriptions en matière de bâtiments et les codes de calcul des structures.

ZUSAMMENFASSUNG

Im letzten Jahrzehnt erfolgten wichtige Fortschritte sowohl beim rechnerischen Erfassen des Brandverhaltens als auch bei der Beurteilung des Brandrisikos. Rechnerische Verfahren für die Ermittlung der Tragwiderstände von Bauteilen und von Tragwerken unter Brandeinwirkung wurden als Alternative zum Normbrandversuch (Ofentest) aufgestellt. Dieser Beitrag liefert eine Übersicht über Methoden zur rechnerischen Bestimmung des Brandwiderstandes von Tragkonstruktionen, insbesondere von Stahlkonstruktionen. Diese Methoden haben inzwischen sowohl in die Praxis als auch in nationalen Richtlinien Eingang gefunden.

1. 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 hand. Presently, the design system is generally 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. Although this grading system with the associated test procedures has been in existence for more than half a century, serious weaknesses can be observed. This applies to the rather arbitrary quantification of fire exposure, including safety considerations, as well as to deficiencies in test procedures, such as inadequate repeatability, reproducibility and simplifications with respect to actual conditions in the structure. The deficiencies in the present design system have certainly stimulated the development of rational methods of risk assessment and analytical modelling of thermal actions and structural response. This paper reviews design methods which have been developed in the last decade and are now becoming operational for practical application and implementation in building regulations and structural codes.

2. CONCEPTS IN STRUCTURAL FIRE ENGINEERING DESIGN

As discussed in the introduction, 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]. Both models can briefly be described as follows:

a. Heat exposure model H, for the determination of the rise of temperature as a function of time. The heat exposure model is supplemented in a probabilistic way, by factors which take into account the probability of occurrence of a large 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. 1) and the structural response models (S) (see horizontal row in Fig. 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 schematization, and consequently also with increased complexity of application (see chapters 3 and 4 for further details).

2.1. Heat exposure models

(H_1) 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.

(H_2) 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.

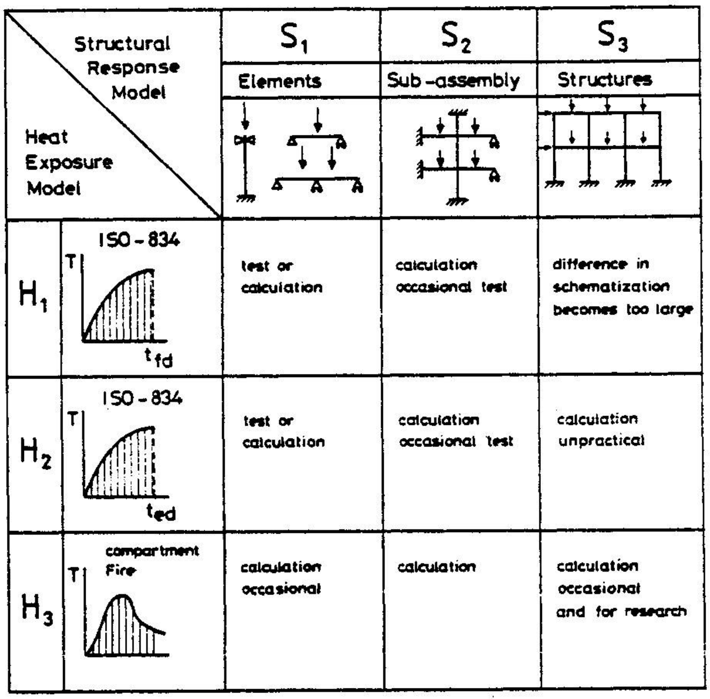
(H_3) A rise of temperature as a function of time characterized by an analytical determination of the gas temperature-time curve of a fully developed compartment fire (natural fire).

2.2. Structural response models

(S_1) 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.

(S_2) 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.

(S_3) 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.



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

Fig. 1 Matrix of heat exposure and structural response models in sequence of improved idealization [1]

Each combination of heat exposure model and structural response model, as an element of the matrix in Fig. 1, represents 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. 1, reference is made to this aspect [1, 2].

3. HEAT EXPOSURE MODELS

As discussed in the introduction most countries use a fire engineering design in which structural performance is connected to grading of elements of construction in a standard fire test (heat exposure model H_1). 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 building in question.

The standard temperature-time relationship according to ISO 834 [3] is given by the following formula (see Fig. 2):

$$T-T_o = 345 \log_{10} (8t + 1) \qquad (3.1)$$

where:
t = time, in minutes
T = furnace temperature at time t, in ^oC
T_o = furnace temperature at time t = 0, in ^oC.

The required time of fire duration is usually expressed in multiples of 30 minutes.

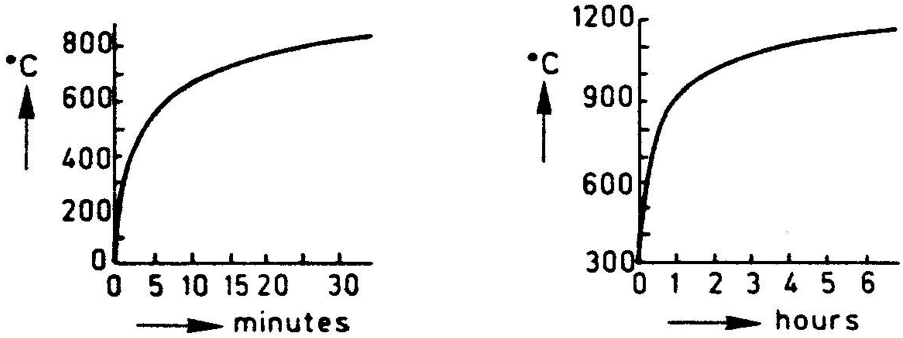


Fig. 2 Standard temperature-time curve [3]

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. It is possible to calculate a complete gas-temperature-time curve using heat balance equations (heat exposure model H_3) [4, 5, 6, 7]. 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]. Fig. 3 exemplifies the result of heat balance calculations for a fully developed compartment fire, with given thermal properties of the compartment and with varying values for the fire load density q and the ventilation factor $A\sqrt{h}/A_t$ [4].
The fire load density q is given by the relationship:

$$q = \frac{1}{A_t} \Sigma \mu_\nu m_\nu H_\nu \ (MJ.m^{-2}) \qquad (3.2)$$

where:
m_ν = total mass of combustible material ν (kg)
H_ν = calorific value of combustible material ν ($MJ.kg^{-1}$)
μ_ν = a fraction between 0 and 1, giving the real degree of combustion for each individual component ν of the fire load, generally assumed equal 1
A_t = total interior area of the surface bounding the fire compartment, including all openings (m^2)

The ventilation factor of the fire compartment is given by the term $A\sqrt{h}/A_t$, in which:

A = total area of door and window openings (m^2)
h = mean value of the heights of the openings, weighted with respect to each individual opening area (m)

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 equation for the fire load and the fire compartment

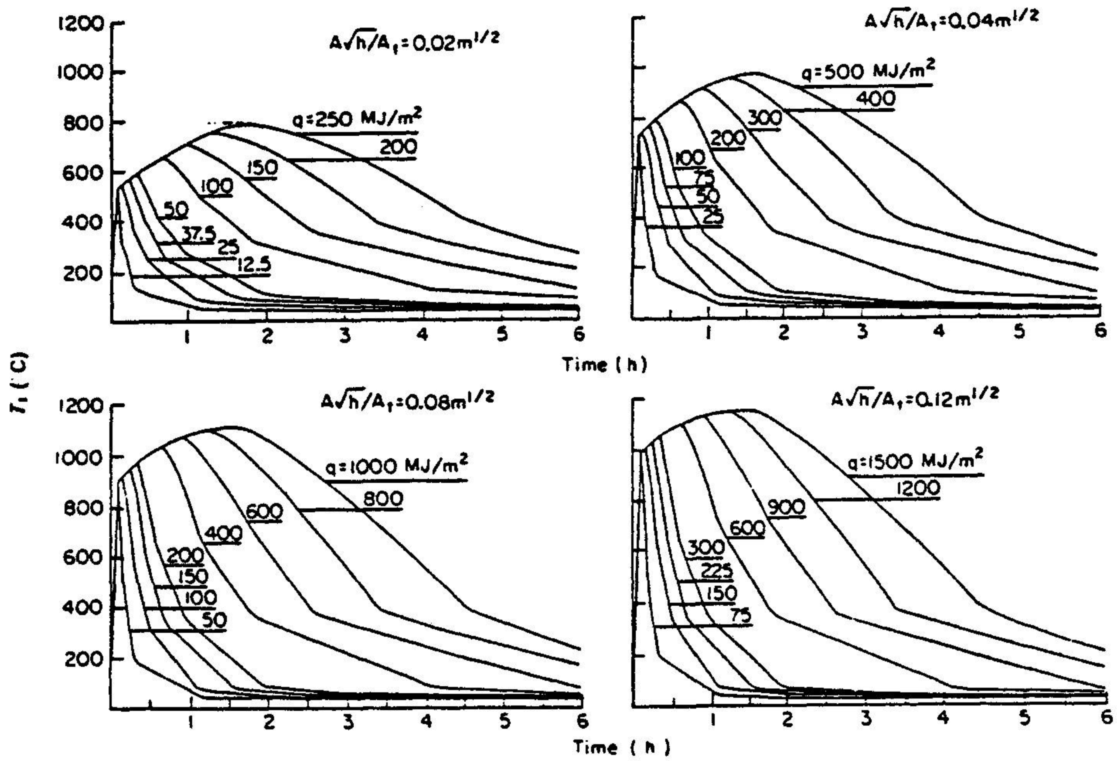


Fig. 3 Gas-temperature-time curves for a complete, fully developed compartment fire with varying values for the fire load density q and the opening factor $A\sqrt{h}/A_t$ according to Pettersson, Magnusson and Thor [4]

in question. For practical applications this requires a computer or a comprehensive set of design charts for different fire loads, ventilation factors and fire compartment characteristics [4]. 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 many 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 limited sense of 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 ultimate state of a representative structural element for a natural fire on one hand and for a thermal exposure according to the standard fire on the other hand. 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 structural element with respect to failure as the complete process of a natural fire. In a generalized approximate approach, the equivalent time of fire exposure is independent from the type of structural element and follows from the value of the fire load density, the geometry and ventilation characteristics of the fire compartment.
For steel structures, the equivalent time of fire exposure t_e can be expressed by [5]:

$$t_e = 0.067 \frac{q}{(A\sqrt{h}/A_t)^{0.5}} \text{ (min)} \qquad (3.3)$$

Safety considerations relevant to the building in question are related to the equivalent time of fire exposure via a set of partial factors (see chapter 5) [1, 8, 9].

4. STRUCTURAL RESPONSE MODELS

As discussed in the introduction, most countries use a method of verification based on grading of elements of construction in a standard fire resistance test, with fixed heating conditions according to equation (3.1). Because of limited dimensions of furnaces, only relatively small elements can be tested with simplified end-conditions. In the matrix of Fig. 1 the method applies to structural model S_1 and occasionally S_2. Internationally, the standard fire resistance test according to ISO 834 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 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 [10, 11]. An illustration is given in Fig. 4, which shows some results of a correlation test series on composite columns carried out in different laboratories [12].

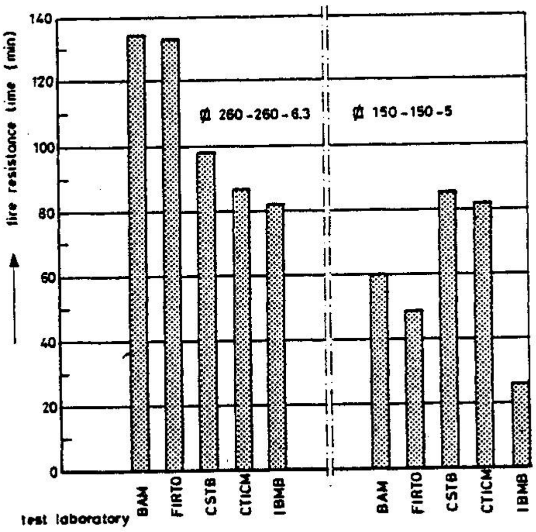


Fig. 4 Some results of fire resistance tests on identical concrete filled hollow steel sections obtained in various test laboratories [12]

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 models. Generally these models 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.

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 (see chapter 6). 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, 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.

5. PROBABILITY BASED METHODS OF STRUCTURAL FIRE ENGINEERING DESIGN

As discussed in chapter 2, each combination of heat exposure model and structural model represents a particular design procedure. In principle, a differentiated fire engineering design offers a problem-oriented choice for the combination of heat exposure model and structural model as a design method. The final choice may also depend on national preferences, the simplicity of application and on the particular design situation [1, 2].

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. In many countries improved methods based on the heat exposure models H_2 and H_3 [4, 5, 6, 7, 8, 9], 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 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 (see chapter 6) [13]. The Recommendations apply to design methods based on heat exposure models H_1 and H_2 and structural response models S_1 and S_2.
Generally, the design criterion in a fire engineering design requires that no limit state is reached during the fire exposure. 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.:

$$\min \{R_{(t)}\} - S > 0 \tag{5.1}$$

In this formula the design criterion is adapted to design methods based on a natural fire, i.e. heat exposure model H_3. For design methods based on the standard temperature-time curve i.e. heat exposure models H_1 and H_2, the design criterion is expressed in a time domain, e.g.:

$$t_{fr} - t_{fd} > 0 \tag{5.2}$$

where t_{fr} is the time in which the limit state of the structural element is reached, i.e. the fire resistance of the structural element, t_{fd} is the required fire duration specified in the building regulations (heat exposure model H_1) or calculated on the basis of heat exposure model H_2.

In the design methods based on heat exposure model H_2 and H_3, the following probabilistic aspects should be considered (heat exposure model H_1 implicitly includes these aspects).

- Intrinsic randomness of design parameters and properties.
- Model uncertainties of the analytical models for the heat exposure and the structural response.
- Assessment of frequency, such as the probability of occurrence of a large fire, the effect of fire brigade actions, the reliability of sprinklers.
- Safety considerations from both the human and economic point of view such as, the height, volume and occupancy of the building, the availability of escape routes and rescue facilities as well as the consequence of violating a limit state.

Introducing these sources in a probabilistic manner into the design means that they must be expressed in numerical values. The level of the probabilistic analysis may well be limited to a semi-probabilistic approach, in which the aspects mentioned above are clustered and expressed in partial factors and characteristic values are used for action and response effects.
For the design method $H_2 - S_2$ with an analytical structural model, this probabilistic design format reads [1, 2, 8, 9]:

$$\frac{t_f}{\gamma_f} - \gamma_{n1}\,\gamma_{n2}\,\gamma_e\,t_e \geq 0 \qquad (5.3)$$

The structural response model represents the first term of the equation and the heat exposure model the second term.

t_f = analytically determined fire resistance time of a sub-assembly
t_e equivalent time of fire exposure for the fire load and the fire compartment in question
γ_f =partial factor taking into account intrinsic randomness of design parameters and material properties at elevated temperatures, uncertainty in loads and load combinations, as well as uncertainty in the analytical structural response model
γ_e = partial factor taking into account the uncertainty in specifying the fire load, ventilation characteristics of the fire compartment and the thermal properties of the enclosure, as well as uncertainty in the heat exposure model
γ_{n1} = partial factor taking into account the assessment of frequency
γ_{n2} = partial factor taking into account the safety considerations

The partial factors γ follow from statistical data and socio-economic optimization supplemented by engineering judgement [8, 9]. The design can be simplified by using unified γ factors for certain classes of buildings, such as appartment buildings, schools, offices etc.
Finally it should be emphasized that a transition from a purely deterministic classification system to probability based methods of design, including analytical design methods as an alternative to the standard fire resistance test, requires improvement and extension of the concepts outlined, as well as extensive calibration to existing code requirements [1, 2, 8, 9, 14, 15].

6. BEHAVIOUR OF STRUCTURAL STEEL AT FIRE EXPOSURE

The analytical model for the calculation of the load bearing capacity of structural steel exposed to fire includes two steps, i.e.

1. A calculation of the temperature distribution within the structure during the heating process.
2. A transformation of the temperature distribution 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 at the specified loading.

The design basis will be summarized below and is focussed on simplified models, equivalent to conventional methods of structural design at room temperatures [13].

6.1. Steel temperature as a function of time

The analysis of the temperature distribution within the fire exposed structure during the heating process, may be generally based on the following simplified assumptions:

- constant thermal properties of structural and insulation materials assumed to be the average for the temperature range,
- the steel is assumed to offer no resistance to heat flow and therefore to be at a uniform temperature,
- the resistance to heat flow between the inner surface of the insulation material and the steel is assumed to be zero.

Under these conditions, the temperature distribution in the steel can be calculated with classical one-dimensional heat flow theory [4, 16, 17, 18].
Under the given assumptions, the resistance of unprotected steel members to heat flow is governed only by convection and radiation. The coefficient of heat transfer due to convection from the fire to the exposed surface of the steel member α_c, is considered to be constant with a value: $\alpha_c = 25\ W/m^2\ {}^oC$.
The coefficient of heat transfer due to radiation α_r, is a function of the gas and steel temperatures and can be determined from the Stefan-Bolzmann law of radiation. The resultant emissivity ε_r of the flames, gases and exposed surfaces which appears in this formula, may be considered constant with a value of $\varepsilon_r = 0.5$, giving a conservative solution.
For a fire exposed unprotected steel structure, the energy balance equation gives the following formula for a determination of the steel temperature:

$$\Delta T_s = \frac{\alpha}{c_s\,\rho_s} \cdot \frac{F}{A} \cdot (T_t - T_s)\ \Delta t\ [{}^oC] \qquad (6.1)$$

in which: $\alpha = \alpha_c + \alpha_r$ $[W/m^2\ {}^oC]$
T_t = gas temperature at time t $[{}^oC]$
T_s = steel temperature $[{}^oC]$
ΔT_s = change of steel temperature during time step Δt
c_s = specific heat of steel $[J/kg\ {}^oC]$
ρ_s = density of steel $[kg/m^3]$
F = fire exposed surface per unit length [m]
A = volume of steel per unit length $[m^2]$

The resistance to heat flow of insulated steel members is governed by convection, radiation and the thermal conductivity of the insulation material. For practical applications however, the influence of convection and radiation can be neglected. Also a distinction is made between lightly insulated members, for which the heat capacity of the insulation material can be neglected, and heavily insulated members, for which the heat capacity of the insulation is taken into account in an approximate way.
For lightly insulated materials, the energy balance equation is:

$$\Delta T_s = \frac{\frac{\lambda}{d}}{c_s\,\rho_s} \cdot \frac{F}{A} \cdot (T_t - T_s)\ \Delta t\ [{}^oC] \qquad (6.2)$$

in which: λ = thermal conductivity insulation $[W/m\ {}^oC]$
d = thickness of insulation [m]

For heavily insulated members half of the heat-capacity of the insulation is added to the heat-capacity $c_s\ \rho_s\ A$ of the steel.
In [4] and [13, 17] design tables are given for natural fire exposure and standard fire exposure respectivily.
During deformation at fire exposure, cracks or openings may occur. In order to include these effects for determining the thermal conductivity of the

insulation material, apart from small scale experiments, at least one full scale test on a loaded member must be performed [17, 19].

6.2. Steel properties and structural analysis at elevated temperatures

In general a structure under fire exposure is subjected to a constant load and a temperature increase as a function of time. Depending on the type and thickness of the insulation, the rate of heating can vary. Research reported in [20, 21] has shown that for practical heating rates and for temperatures not over 600 °C, the deformation behaviour under constant load can be considered as independent of the heating rate. Consequently a family of stress-strain relationships for different temperatures must exist, in which the influence of high temperature creep is implicitly included (Fig. 5).

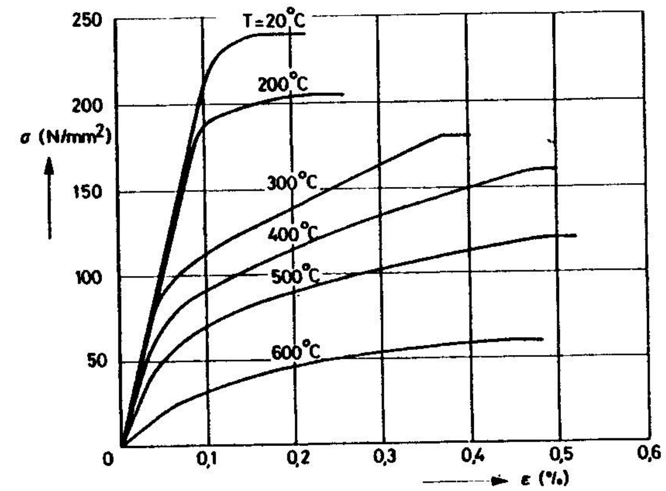


Fig. 5 Stress-strain relationships for Fe 360 at elevated temperatures

The gap between the curves applying to 200 °C and 300 °C is due to so-called "thermally activated flow" [20, 21]. Applying the elementary theory of plasticity, the curved stress-strain diagrams are cut off at certain stress levels. The horizontal plateau is defined as the effective yield stress. In Fig. 6, the effective yield stress variation with steel temperature is given as a fraction of the yield stress at room temperature.

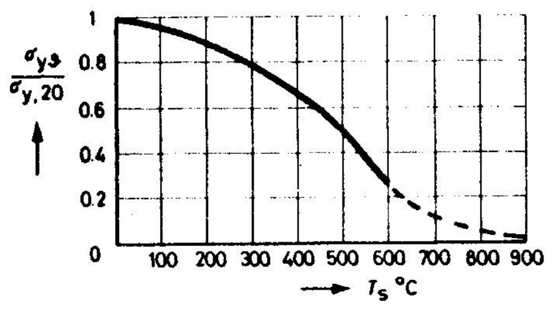


Fig. 6 Effective yield stress $\sigma_{y\vartheta}$ at elevated temperatures expressed as a fraction of the yield stress at room temperature $\sigma_{y,20}$ (Fe 360 - Fe 510)

The structural analysis of fire exposed structures may be generally based on the following simplified assumptions:

- a time dependent uniform temperature distribution over the height of the cross section and along the members,

- mechanical properties of steel at elevated temperatures which are assumed to be independent of time, i.e. creep effects are included implicitly (fig. 5, 6),
- the load is assumed to be constant and equal to the design load at the service state e.g. dead load + characteristic live loads.

Due to the non-linear stress-strain relationships of steel at elevated temperatures, the linear theory of elasticity cannot be applied and use has to be made of the theory of plasticity.
Two design methods are available, identical to those used in structural analysis at room temperature:

- a limit state design according to the elementary theory of plasticity in those cases where a similar design is allowed at room temperature,
- an incremental elasto-plastic analysis.

The first method is suitable when the limit state at elevated temperatures can be defined by structural collapse, i.e. beams in braced frames. At a given temperature the ultimate load can be calculated from the temperature dependent effective yield stress $\sigma_{y,\nu}$ (Fig. 6). This is illustrated in Fig. 7 [13, 17, 22].

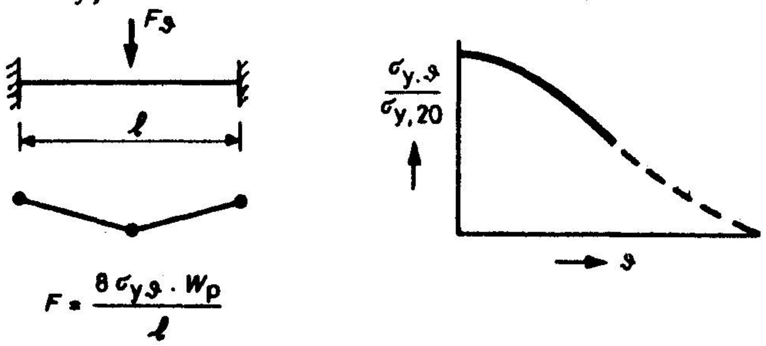


Fig. 7 Structural design at elevated temperatures according to the elementary theory of plasticity

In the European Recommendations for the fire safety of steel structures [13] numerous diagrams and tables are given for the determination of the load-bearing capacity at elevated temperatures of beams, columns and braced frames. Fig. 8 gives an example of such a diagram for unrestrained axially loaded columns, based on tests performed recently in Belgium and some other countries [23].

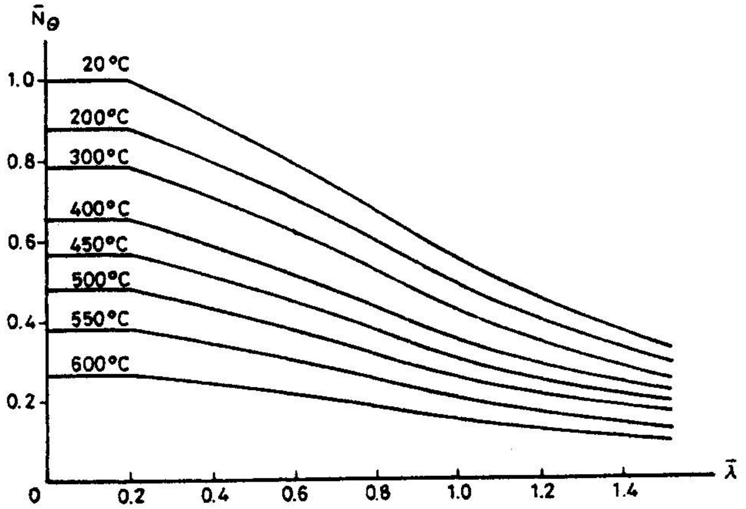


Fig. 8 Relationship between non-dimensional buckling load $\bar{N}_\theta$ and slenderness factor $\bar{\lambda}$ at varying steel temperature T_s for unrestrained axially loaded steel columns [23]

The second method has to be used when the limit state at elevated temperatures is defined by a criterion based on deflections or a rate of deflection. This method must also be applied when geometrically non-linear effects have a significant bearing on the structural behaviour i.e. columns and unbraced frames. At a given temperature, the load-bearing capacity can be determined with the associated stress-strain relationship (Fig. 5), by computing the deflection curve. Fig. 9 gives an illustration. Application of this method usually requires a computer [21].

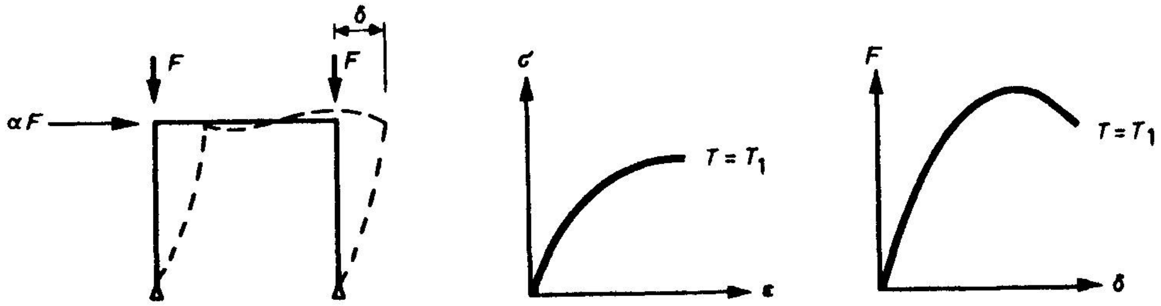


Fig. 9 Structural design at elevated temperatures with an incremental elasto-plastic analysis

This method is particularly used for research purposes, from which simplified design rules can be obtained and implemented in codes for structural fire safety.

ACKNOWLEDGEMENT

International cooperation on the development of new concepts for structural fire engineering design takes place in the Fire Committee of the Conseil International du Bâtiment (CIB/W14) [2]. Design methods for structural steel exposed to fire are coordinated in the Fire Committee of the European Convention for Constructional Steelwork (ECCS-TC3) [5, 13, 24, 25]. The author is grateful for the stimulating discussions and contributions in these committees, which certainly have influenced the contents of this paper.

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