Structural fire behaviour - development trends

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Structural Fire Behaviour—Development Trends

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ABSTRACT

During the last two decades, a rapid progress has been made in the development of analytical methods for a fire engineering design of load bearing and separating structures and structural members. Consequently, more and more countries are now permitting a classification of structural members with respect to fire to be formulated analytically as an alternative to the internationally prevalent method of classification based on results of standard fire resistance tests. In a long-term perspective, the development goes towards an analytical design, directly based on a natural fire exposure, specified with regard to the combustion characteristics of the fire load and the geometrical, ventilation and thermal properties of the fire compartment.

Parallel to this progress, a further development is going on towards a reliability based structural fire engineering design. The development includes contributions related to a practical design format calculation, based on partial safety factors, as well as to an evaluation, based on first order reliability methods.

The paper describes and comments on these developments.

INTRODUCTION

During the last twenty years, important and rapid progress has been noted in the development of analytical and computation methods for the determination of the thermal and mechanical behaviour and the load bearing capacity of building structures and structural members exposed to fire. Consequently, an analytical design can be carried out today for most cases where steel structures are involved. Validated material models for the mechanical behaviour of concrete under transient high-temperature conditions and thermal models for a calculation of the charring rate in wood exposed to fire, derived during recent years, have significantly increased the area of application of analytical design. To aid this application, design diagrams and tables have been systematically computed and published, giving directly, on the one hand, the temperature state of the fire exposed structure, and on the other, a transfer of this information to the corresponding load bearing capacity of the structure [1-27].

METHODS OF STRUCTURAL FIRE DESIGN

The internationally applied methods for a fire design of load bearing structures and structural members may be described in outline with reference to the matrix set out in Figure 1 [25, 28]. The matrix is based on three models for thermal exposure (H_1, H_2 and H_3) in relation to three types of structural models (S_1, S_2 and S_3).

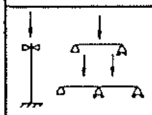
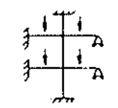
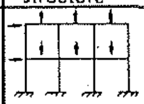
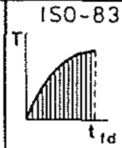
Model for structure / Model for thermal exposure		S_1 Element	S_2 Substructure	S_3 Complete structure
				
H_1	ISO-834 	test or calculation (deterministic)	calculation exceptionally testing (deterministic)	
H_2	ISO-834 	test or calculation (probabilistic)	calculation, exceptionally testing (probabilistic)	calculation (probabilistic) should be avoided
H_3	real fire 	calculation (probabilistic)	calculation (probabilistic)	calculation (probabilistic) in special cases and for research

FIGURE 1. Summary description of different methods for design of load bearing structures and structural elements under fire exposure conditions.

The thermal exposure conditions or models are defined as follows:

H_1 - A thermal exposure described by the standard temperature-time curve

$$T_t - T_o = 345 \log_{10}(8t + 1) \tag{1}$$

as specified in ISO Standard 834, "Fire Resistance Tests - Elements of Building Construction". T_t = furnace temperature at time t (°C), T_o = furnace temperature at time t = 0 (°C) and t = time in minutes. The time of exposure t_{fd} represents the time during which the structural element - or substructure - is required to fulfil its load bearing and/or separating function according to specifications in codes and regulations. The function may be verified either by test or by calculation.

H_2 - The same thermal exposure as for H_1 except that the *duration of exposure* t_e is determined in each case for the characteristics of a particular compartment fire. Accordingly, t_e represents an equivalent time of the standard fire exposure which produces the same effect upon the structural element or substructure with respect to the decisive limiting condition as the relevant natural fire. For protected and unprotected steel structures, the following approximate formula applies [29]:

$$t_e = 0.067 \frac{f}{\left(\frac{A\sqrt{h}}{A_{tot}}\right)^{\frac{1}{2}}} \qquad \text{(min)} \qquad (2)$$

where f = fire load density per unit area of the surfaces bounding the fire compartment ($MJ \cdot m^{-2}$), $A\sqrt{h}/A_{tot}$ = opening factor of the fire compartment ($m^{\frac{1}{2}}$), A = total area of the openings (m^2), h = mean value of the heights of the openings, weighted with respect to each individual opening area (m) and A_{tot} = total interior area of the structures enclosing the compartment, opening areas included (m^2).

Eq. (2) is verified to be appropriate for use also for those reinforced concrete beams where the critical concern is yielding of the reinforcement under bending conditions. For other types of load bearing and separating structural elements, there are very few studies reported on the applicability of the formula.

H_3 - A thermal exposure determined by the conditions of a fully developed compartment fire with consideration given to: the combustion characteristics of the fire load, the ventilation of the fire compartment and the thermal properties of the structures enclosing the fire compartment. In the individual case, the exposure can either be calculated from the energy and mass balance equations for the compartment fire or be derived from curves or tables in manuals, giving the time variation of either the gas temperature within the compartment or the corresponding heat flux to the structure - cf., for instance, [2, 7, 8, 13, 17, 28] and further references given in these publications. Figure 2 gives an example of such data, taken from the Commentary 1976:1 to the Swedish Building Code.

The structural models are defined as follows:

S_1 - A simplification of a real structure by division into single elements such as beams and columns. The structural model may either be represented by a test specimen or dealt with analytically.

S_2 - A simplification of a real structure by division into substructures such as beam-column systems. The substructure thus derived is provided with well-defined, simplified conditions of support and/or restraint at its outer ends or edges. As with S_1, the structural model may either be dealt with analytically or - exceptionally - be represented by a test specimen.

S_3 - A complete real structure, e.g. a two or three dimensional frame, a beam-slab system or a column-beam-slab system. Such structural models are generally dealt with analytically, normally requiring the support of a computer.

The internationally most prevalent structural fire design is characterized by the combination H_1-S_1. The design is usually related to the results of the standard fire resistance test according to the ISO Standard 834 or some equivalent national standard. The fire resistance may also be derived analytically and this alternative is now officially being permitted in more and more countries. A few countries allow the application of the model combination H_1-S_2, normally by analytical methods. The combination H_1-S_3 involves a too great difference in the accuracy of simulation between the thermal exposure and structural models to be acceptable in practice.

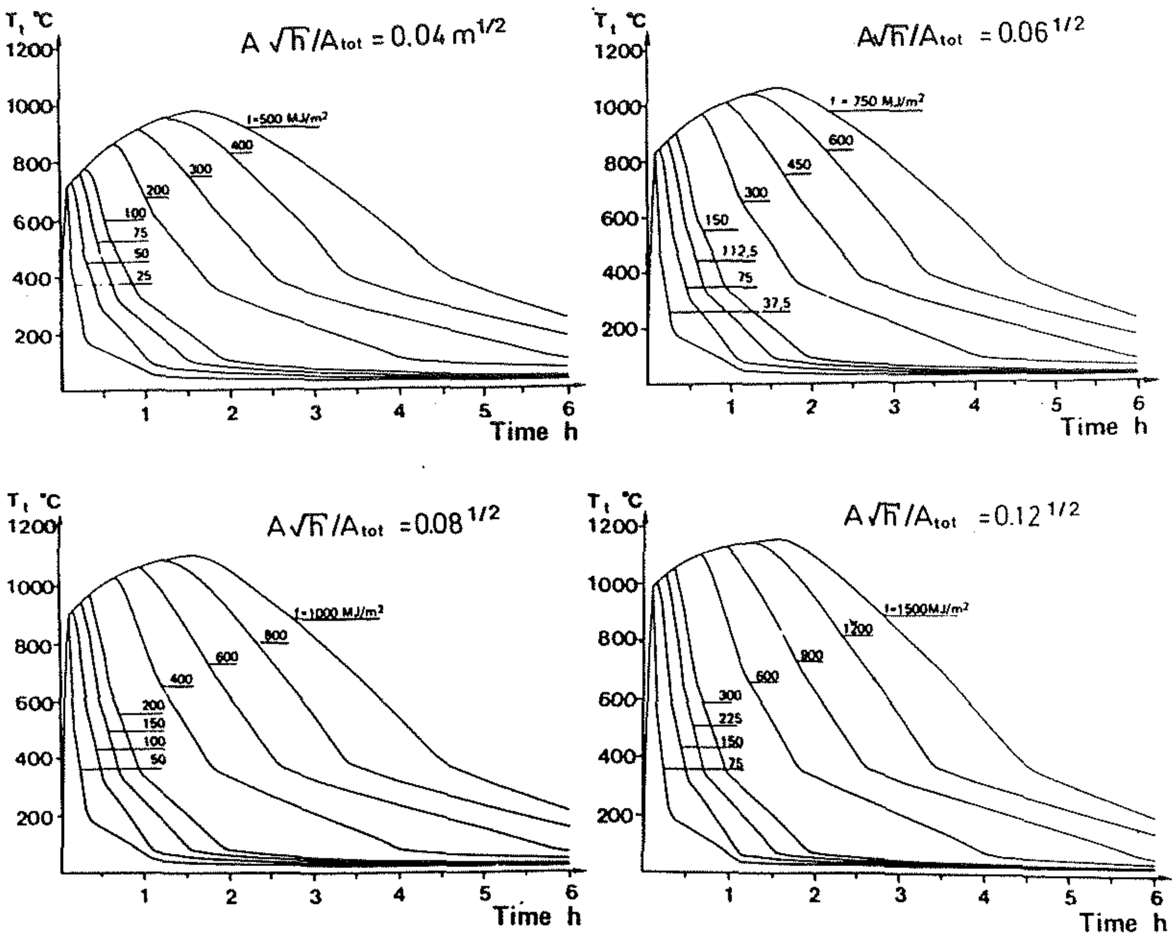


FIGURE 2. Examples of gas temperature-time curves for a natural fire as a function of fire load density f and opening factor $A\sqrt{h}/A_{tot}$ of the fire compartment. Enclosing structures, made of a material with a thermal conductivity $k = 0.81\ W\cdot m^{-1}\cdot {}^{o}C^{-1}$ and a heat capacity $\rho c_p = 1.67\ MJ\cdot m^{-3}\cdot {}^{o}C^{-1}$; fire compartment type A [7].

The substantial progress during the last twenty years in the development of analytical methods, referred to above, has considerably increased the possibility of performing a structural fire design, based upon the thermal exposure models H_2 and H_3 as an alternative to the conventional use, at present, of the thermal exposure model H_1.

A design directly based on a natural compartment fire exposure H_3 is generally characterized by an analytical treatment. For rapid practical application, it is necessary for systematized design data in the form of e.g. manuals to be available. Usually, then the model combination H_3-S_2 is used, and in certain cases the model combination H_3-S_1. Design according to the combination H_3-S_3 generally demands access to a computer. This combination is of central importance in the research context.

A structural design for thermal exposure of the H_2 type is based indirectly on a natural compartment fire, described by the temperature-time curve according to ISO 834, Eq. (1), with reference to the concept of equivalent time of fire exposure t_e. The structural behaviour, calculated for such an exposure, differs from the behaviour in a natural fire situation in cases where the heating history is of significance. In the combination H_2-S_1, the design can be performed either analytically or on the basis of a furnace test according to ISO 834. In the combination H_2-S_2, analytical design is the normal procedure, and experimental verification is an exception. The combination H_2-S_3 may be questioned from a practical standpoint since it does not provide for the simplifications of a design developed using the model combination H_3-S_3.

CHARACTERISTICS OF RELIABILITY BASED STRUCTURAL FIRE DESIGN

The most recent trend in the development of the structural fire design is to adopt modern loading and safety philosophy and include a probabilistic approach, based on either a system of partial safety factors (practical design format) or the safety index concept [25, 26, 28, 30-37]. For an everyday design, a direct application of the safety index concept then is too cumbersome and the more simplified practical design formats have to be used.

The fundamental components of such a reliability based structural fire design are

* the limit state conditions
* the physical model
* the practical design format
* deriving the safety elements.

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.

$$\min\{R(t)\} - S \geq 0 \qquad (3)$$

The criterion must be fulfilled for all relevant types of failure. The requirements with respect to insulation and integrity apply to separating structures. The design criterion regarding insulation implies that the highest temperature on the unexposed side of the structure - $\max\{T_s(t)\}$ - shall meet the temperature T_{cr}, acceptable with regard to the requirement to prevent a fire spread from the fire compartment to an adjacent compartment, i.e.

$$T_{cr} - \max\{T_s(t)\} \geq 0 \qquad (4)$$

For the integrity requirement, there is no analytically expressed design criterion available at present. Consequently, this limit state condition has to be proved experimentally, when required, in either a fire resistance test or a simplified small scale test.

The physical model comprises the deterministic model, describing the relevant physical processes of the thermal and mechanical behaviour of the structure at specified fire and loading conditions. Supplemented with relevant partial safety factors, the physical model is transferred to the practical design format.

Related to an analytical fire design of load bearing structures, directly based on the natural compartment fire exposure - thermal exposure type H_3 - the practical design format can summarily be described according to the flow chart in Figure 3.

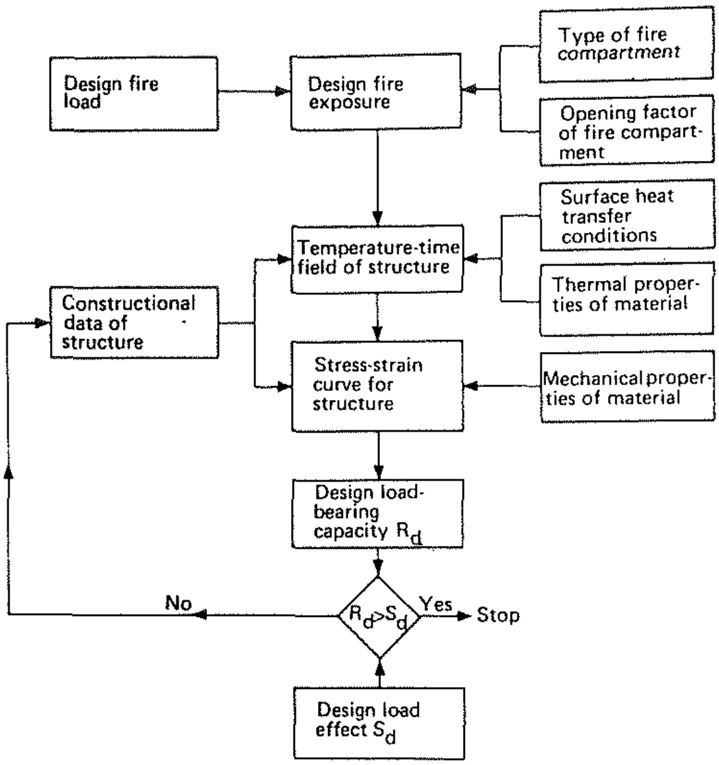


FIGURE 3. Flow chart for an analytical fire design of load bearing structures on the basis of a natural compartment fire exposure.

From the design fire load and the geometrical, ventilation and thermal characteristics of the fire compartment (opening factor and type of fire compartment), the design fire exposure is determined either by energy and mass balance calculations or from a systematized design basis. Together with design values for the constructional data of the structure and the thermal and mechanical properties 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.

The design format condition to be proved is

$$R_d - S_d \geq 0 \tag{5}$$

where S_d is the design load effect at fire. Depending on the type of practical application, the condition has to be verified for either the complete fire process or a limited part of it, determined by, for instance, the design evacuation time for the building.

The probabilistic influences are considered by specifying characteristic values and related partial safety factors for the fire load, such structural design data as imperfections, the thermal properties, the mechanical strength and the loading. The partial safety factors then are to be derived by a probabilistic analysis, based on a first order reliability method.

The procedure of deriving the safety elements is further outlined in Figure 4, as exemplified for a timber structure [26].

Expressed in terms of a safety index β - defined as the ratio of the mean value of the safety margin to its standard deviation - the design criterion then has the form

$$\beta_{fm} - \beta_r \geq 0 \qquad (6)$$

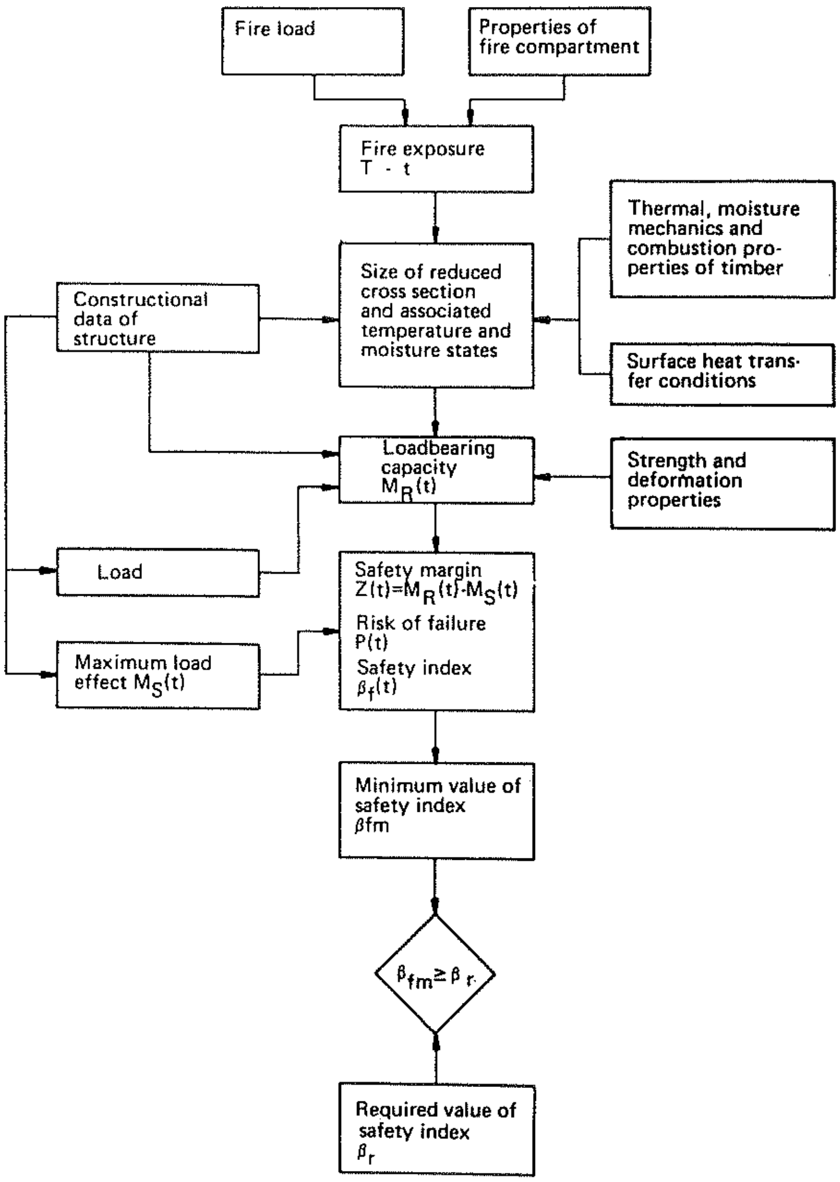


FIGURE 4. Derivation of partial safety factors for a fire exposed timber structure by a probabilistic analysis, based on a first order reliability method.

where β_{fm} is the least value of the safety index for the structure during the relevant fire process and β_r is the required value of the safety index.

The safety margin is defined by the formula

$$Z(t) = M_R(t) - M_S(t) \tag{7}$$

where $M_R(t)$ is the load bearing capacity at time t, expressed in terms of e.g. the bending moment at a critical section of the structure, and $M_S(t)$ is the corresponding bending moment related to the maximum load effect. The corresponding probability of failure P(t) and safety index $\beta_f(t)$ are given by the formulae

$$P(t) = \int_{-\infty}^{0} f_Z\{Z(t)\}dZ \tag{8}$$

$$\beta_f(t) = \phi^{-1}\{1 - P(t)\} \tag{9}$$

where $f_Z\{Z(t)\}$ = the probability density function of the safety margin Z and ϕ^{-1} = = the inverse of the standardized normal distribution.

In determining Z(t), P(t) and $\beta_f(t)$, the following probabilistic effects must be taken into account:

* The uncertainty in specifying the loads and of the model, describing the load effect on the structure
* the uncertainty in specifying the fire load and the characteristics of the fire compartment
* the uncertainty in specifying the design data of the structure and the thermal and mechanical properties of the structural materials
* the uncertainty of the analytical models for the calculation of the compartment fire, the heat transfer to and within the structure and its ultimate load bearing capacity.

The required value of the safety index β_r depends on

* the probability of occurrence of a fully developed compartment fire
* the efficiency of the fire brigade actions
* the effect of an installed extinction system, if any
* the consequences of a structural failure.

In the design procedure according to Figure 3, the latter four influences can be accounted for by the partial safety factors allocated to either the design mechanical strength or the design fire load and design fire exposure.

For a structural fire design, based on the thermal exposure model H_2, the practical design format can be given in the following form [28, 36]:

$$\frac{t_f}{\gamma_f} \geq \gamma_n \gamma_e t_e \tag{10}$$

where t_f is the fire resistance of the structural element, t_e equivalent time of fire exposure - Eq. (2) - and γ_f, γ_n 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 and the fire compartment characteristics, including the uncertainties of the analytical model for a determination of the fire exposure. The partial safety factor γ_f

considers the uncertainties of the mechanical load and the thermal and mechanical 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 factor (differentiation factor) γ_n takes into consideration the effects related to the required safety index β_r, as listed above.

THERMAL AND MECHANICAL BEHAVIOUR OF BUILDING STRUCTURES AT FIRE EXPOSURE

As stated in the introduction, the important progress in modelling the thermal and mechanical behaviour of fire exposed structures and structural elements during the last twenty years has considerably enlarged the area of application of an analytical fire design. Access to validated material behaviour models for transient high-temperature conditions then is a necessary prerequisite for a successful simulation of the real structural fire behaviour.

Thermal Properties and Transient Temperature State

The transient heat flow within a fire exposed structure is governed by the heat balance equilibrium equation, based on the Fourier law

$$\underline{\nabla}^T(\underline{\lambda}\underline{\nabla}T) - \dot{e} + Q = 0 \tag{11}$$

where T = temperature, $\underline{\lambda}$ = symmetric positive definite thermal conductivity matrix, $\dot{e} = \partial e/\partial t$ = rate of specific volumetric enthalpy change, Q = rate of internally generated heat per volume, and t = time. The gradient operator $\underline{\nabla}$ is defined as

$$\underline{\nabla} = \begin{bmatrix} \frac{\partial}{\partial x} \\ \frac{\partial}{\partial y} \\ \frac{\partial}{\partial z} \end{bmatrix} \tag{12}$$

where x, y and z are Cartesian coordinates.

For isotropic materials

$$\underline{\lambda} = \lambda\underline{I} \tag{13}$$

where λ = thermal conductivity, and $\underline{I}$ = identity matrix.

A solution of Eq. (11) requires the initial and boundary conditions to be specified. The initial condition is given by the distribution of temperature within the structure at a reference time zero. The boundary conditions are prescribed as temperature $T = T(x,y,z,t)$ or heat flow q on parts of the boundary ∂V_T and ∂V_q, respectively. The total boundary is then

$$\partial V = \partial V_T + \partial V_q \tag{14}$$

The heat flow normal to the surface on the boundary ∂V_q must satisfy the heat balance equation

$$q_n = -\underline{n}\lambda\nabla T \tag{15}$$

where $\underline{n}$ = outward normal to the surface.

At free surfaces, the heat flow q_n is caused by convection q_{nc} and radiation q_{nr} and follows the formula

$$q_n = q_{nc} + q_{nr} = \alpha(T_s - T_t)^m + \varepsilon_r \sigma(\bar{T}_s^4 - \bar{T}_t^4) \tag{16}$$

where α, m = convection factor and convection power, respectively - see, for instance, [38], ε_r = resulting emissivity, varying with gas or flame emissivity, surface properties and geometric configuration, σ = Stefan-Boltzmann constant, T_s = surface temperature, $\bar{T}_s$ = absolute surface temperature, T_t = surrounding gas temperature, and $\bar{T}_t$ = absolute surrounding gas temperature.

The solution of Eq. (11) is complicated by the fact that the thermal conductivity matrix $\underline{\lambda}$ and the rate of specific volumetric enthalpy change $\dot{e}$ depend on the temperature T to an extent that cannot be diregarded. Further complications arise when the material undergoes phase changes during the heating and when the material has an initial moisture content.

Well-defined measurements of the thermal conductivity λ for moist materials are difficult to undertake within the temperature range relevant at fire exposure, due to the complicated interaction between moisture and heat flow. As concerns the enthalpy e, the way evaporable water reacts to pressure has not been experimentally clarified and consequently, this influence has to be included in a simplified manner in calculating the transient temperature state of a fire exposed structure. Usually, all moisture is assumed to evaporate, without any moisture transfer, at the temperature 100°C or within a narrow temperature range - ref. 38 applies a range of 100 to 115°C - with the heat of evaporation giving a corresponding discontinuous step in the enthalpy curve. This simplification has proved to give acceptable results for most practical situations.

In reality, the evaporation of moisture in a fire exposed material is not comparable to that of a free water surface. Capillary forces, adhesive forces, and interior steam pressure will allow the temperature to increase during evaporation. During the heating of the structure, the moisture distribution changes continuously. Hence, it is not principally correct to include the effect of moisture content in the thermal properties. For a moist material, the heat transfer is combined with moisture transport and, from a strict thermodynamical point of view, these two transport mechanisms should be analysed simultaneously by a system of related partial differential equations. Consequently, Eq. (11) constitutes an approximation when applied to fire exposed structures made of materials that contain moisture.

For materials used, for instance, for fire protection of steel structures or in suspended ceilings, there are test methods developed for a determination of derived values, characterizing the fire behaviour of the product in an integrated way. Normally, the values are derived from test results by use of some analytical simulation model. As a consequence, the derived values do not represent any well-defined material or product properties but are influenced also by the characteristics of the analytical model, adopted for the evaluation. This leads to limitations with respect to a generalized application of the derived values.

Analytical solutions of the heat balance equilibrium equation (11) are feasible only for linear applications with simple geometries and boundary conditions. For a practical determination of the transient temperature state of fire exposed structures, numerical methods have been developed and arranged for computer calculations. The methods are based either on finite difference or finite element approximations. For the first group of numerical methods, reference can be made to [12, 39-44], and for the group employing finite element methods to [12, 38, 45-48].

The computer programmes can be used either directly as an advanced component in the fire design procedure or as a tool for calculations of diagrams and tables, facilitating a practical determination of the design temperature state for varying conditions of fire exposure and varying structural characteristics. For a thermal exposure according to the standard temperature-time curve, Eq. (1), such design aids are given in [5, 6, 9, 10, 13, 20, 23, 27] for steel structures and in [3-5, 11, 14-16, 19, 21] for concrete structures. A corresponding design aid is presented in [2, 7, 13, 17, 29, 49] for steel structures and in [7, 49] for concrete structures when exposed to a natural compartment fire with gas temperature-time curves according to Figure 2.

Fire exposed timber structures present special problems due to the continuous decrease of the effective cross section by combustion of the material. For a thermal exposure according to the standard fire resistance test, Eq. (1), a large number of tests, made in different fire engineering laboratories, verify an approximately constant rate of charring of 3.5 $cm \cdot h^{-1}$ for glued laminated timber beams and columns. The value is roughly applicable up to a charring depth equal to one quarter of the cross section dimension in the direction of charring. For a larger charring depth, the rate of charring increases.

Analytical models for a calculation of the charring rate and depth of wood at varying thermal exposure are presented in, for instance, [26, 50-54]. The refs. [51, 52, 54] also include a model for a determination of the temperature distribution within the uncharred part of the cross section. Ref. [53] includes diagrams giving the charring depth of a cross section at a natural compartment fire exposure, defined by the gas temperature-time curves according to Figure 2.

Mechanical Properties and Structural Behaviour

A reliable calculation of the mechanical behaviour and load bearing capacity of a fire exposed structure or structural element on the basis of the transient temperature state requires validated models for the mechanical behaviour of the materials involved within the temperature range associated with fires. It is important that the material behaviour models are phenomenologically correct with input information received from well-defined tests.

Available tests for a determination of the mechanical properties of materials at elevated temperatures can mainly be divided into two groups: steady state tests and transient state tests - Figure 5 [55]. Fundamental parameters are the heating process, application and control of load, and control of strain. These can have constant values or be varied during testing.

Figure 5 defines six practical regimes with mechanical properties as follows:

* steady state tests
  - stress-strain relationship (stress rate control, $\dot{\sigma} = const$)
  - stress-strain relationship (strain rate control, $\dot{\varepsilon} = const$)
  - creep (stress control, $\sigma = const$)

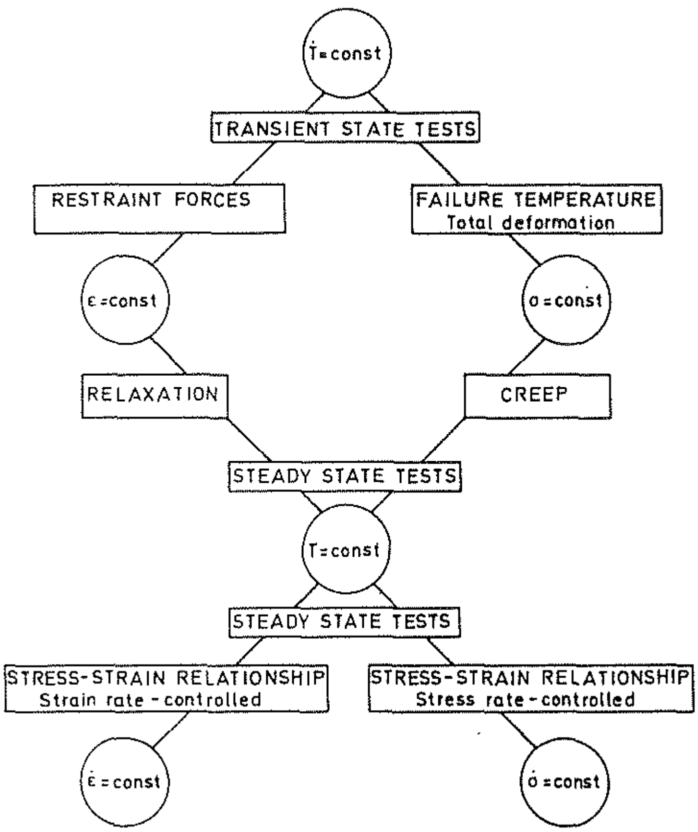


FIGURE 5. Different testing regimes for determining mechanical properties of materials at elevated temperatures [55].

- relaxation (strain control, $\varepsilon = \text{const}$)

* transient state tests
  - failure temperature, total deformation (stress control, $\sigma = \text{const}$)
  - restraint forces, total forces (strain control, $\varepsilon = \text{const}$).

The material properties measured are closely related to the test method used. Consequently, it is extremely important that reported test results always are accompanied by an accurate specification of the test conditions applied. For steels, there is analytical modelling technique available enabling a coupling of steady state and transient state tests [55].

For steel, validated mechanical behaviour models for transient, high-temperature conditions have been available for many years - cf., for instance, [55-59]. The models divide the total strain into thermal strain, instantaneous stress-related strain and creep strain or time dependent strain. Some of the models operate with temperature compensated time according to Dorn [56].

Analytical models for determination of the mechanical behaviour and load bearing capacity of steel beams, columns and frames exposed to fire are presented in, for instance, [12, 57-66]. The most general models are those put forward in [59, 63-66]. A simplified design aid, giving directly the load bearing capacity for a design temperature state or the critical temperature state for a design load effect, can be found in [2, 5-7, 9, 10, 13, 17, 20, 23, 27, 49, 57].

Simple formulae for the fire resistance of unprotected and protected steel columns, derived by Lie and Stanzak, are quoted in [8].

For concrete, the deformation behaviour at elevated temperatures is much more complicated than for steel. Stressed concrete involves special difficulties since considerable deformations develop during the first heating which do not occur when the temperature is stable. This effect has been confirmed by flexural, torsional and compressive tests and for moderate as well as high temperatures.

For practical applications, the total strain ε can adequately be given as the sum of various strain components, phenomenologically defined with reference to specified tests and depending on the temperature T, the stress σ, the stress history $\tilde{\sigma}$, and the time t. For concrete, stressed in compression, then the following constitutive equation applies [67]

$$\varepsilon = \varepsilon_{th}(T) + \varepsilon_{\sigma}(\tilde{\sigma},\sigma,T) + \varepsilon_{cr}(\sigma,T,t) + \varepsilon_{tr}(\sigma,T) \qquad (17)$$

where ε_{th} = thermal strain, including shrinkage, measured on unstressed specimens under variable temperature; ε_{σ} = instantaneous, stress-related strain, based on stress-strain relations, obtained at a rapid rate of loading under constant, stabilized temperature; ε_{cr} = creep strain or time-dependent strain, measured under a constant stress at constant, stabilized temperature; and ε_{tr} = = transient strain, accounting for the effect of temperature increase under stress, derived from tests under constant stress and variable temperature.

For stressed concrete in a transient high-temperature state, the transient strain component ε_{tr} usually plays a predominant role. Parameter formulations for each of the strain components and a practical guidance on the application of the material behaviour model at a time varying stress and temperature state are given in [67]. An alternative model formulation of the mechanical behaviour of concrete at transient elevated temperatures is given in [68].

In [69] an attempt is made to formulate a multiaxial constitutive model for concrete in the temperature range up to 800°C. The model can be characterized as isotropic, elastic-viscoplastic-plastic in the compression region. Brittle failure is assumed in the tensile region.

As stressed above, validated material behaviour models of the type described is a condition for getting reliable results from the calculation models and corresponding computer programmes for determination of the mechanical behaviour and load bearing capacity of fire exposed reinforced concrete structures. Comprehensive computer programmes for such a determination are presented in [12, 22, 65, 70-76]. The methods dealt with in [12, 22, 65, 72, 73, 75, 76] include secondary order effects. A computer programme for evaluating the fire response of reinforced concrete slabs is published in [77]. The programme is based on a non-linear finite element method coupled with a time-step integration and includes a combined bending and membrane action of the slab.

Simplified methods, facilitating the practical design of fire exposed, reinforced concrete beams and columns exposed to fire can be found in [3-5, 7, 8, 11, 14, 16, 19, 21, 39, 40, 78-80]. In [8], simple formulae are given for the fire resistance of concrete beams, columns, walls and slabs, based on an international survey - cf. also [14, 19]. The design aid has to be applied with due consideration of the fact that the analytical tool for a determination of the ultimate load bearing capacity of fire exposed concrete structures mainly covers the failure in bending. For other kinds of failure - shear, bond, anchorage and spalling - the present state of knowledge is still unsatisfactory. In a practical fire engineering design, it is therefore important to detail the structure in such a way that these types of failure will have a lower probability of occurrence than failure by bending.

For load bearing timber structures, the possibilities for an analytical modeling of the mechanical behaviour during fire exposure are essentially more limited than for steel and concrete structures. As mentioned earlier, validated analytical models are available for a calculation of the charring rate of wood under varying thermal exposure and approximate models also exist for an evaluation of the temperature distribution within the uncharred part of the cross section under the simplified assumption of no moisture content in the wood [51, 52, 54]. It is highly desirable that analytical models should be developed for the mass transfer of moisture and for the mechanical behaviour of wood under conditions of transient temperature and moisture content.

In [81], approximate formulae are derived for the fire resistance of laminated timber beams and columns, exposed to the standard temperature-time curve. A detailed structural design guide for the fire resistance of beams, columns, joints, floors, roofs and walls, based on classification and results of standard fire resistance tests is given in [24] which is a very comprehensive manual. A simplified design aid for laminated timber beams and columns, exposed to a natural compartment fire, is presented in [7, 26], and [82] supplements this design aid for beams with respect to the risk of lateral buckling during the fire exposure.

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