Reliability Based Design of Fire Exposed Concrete Structures

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LUND INSTITUTE OF TECHNOLOGY · LUND · SWEDEN
DIVISION OF BUILDING FIRE SAFETY AND TECHNOLOGY
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ABSTRACT

During the last ten years a rapid progress has been made in the development of analytical methods for a fire engineering design of load bearing structures and structural members. In a long-term perspective, the development then 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.

This progress is now followed up by a further development towards a reliability based structural fire engineering design. Internationally, the development includes contributions, related to a probabilistic approach on level I, based on a system of partial safety coefficients, as well as to a probabilistic approach on level II, based on the safety index concept.

The paper describes and comments on these parallel developments with special reference to fire exposed reinforced concrete structures.

INTRODUCTION

In a general sense, the fire engineering design problem is non-deterministic. Some level of risk - the probability of an adverse event - is virtually unavoidable and we have to recognize the impossibility of absolute compliance with a preset goal. Performance has to be described and measured in probabilistic terms.

Essential components of a reliability based design methodology include - in the ideal case (1)

* analytical modeling of relevant processes; verification of model validation and accuracy; determination of critical design parameters,

* formulation of functional requirements, independent of choice of design process, expressed either in deterministic or probabilistic terms,

* determination of design parameter values,

* verification by reliability analysis that the choice of safety factors leads to safety levels which are consistent with the expressed functional requirements.

Lack of knowledge concerning the structure of analytic models describing the physical processes has, up to recently, prevented all efforts to assess risk levels quantitatively within the field of fire engineering design. Gradually, with expanding modeling capabilities, the potential for a rational, reliability-based design will proportionally increase.

One of the applications, where fire safety analyses have been furtherst developed, applies to load bearing building structures and structural members. Mainly, then two schools can be distinguished, namely

(1) classical probability analyses as developed by Freudenthal and others (2), (3), (4), requiring that the probability density functions of the strength or resistance R and the load effect S are known or can be acceptably prescribed, and

(2) a more engineering directed approach, connected to the concept of safety index β (5),(6),(7),(8),(9).

In the latter approach, a design scheme can be based simply on the requirement that some minimum safety margin be maintained. In place of requiring that a calculated risk of failure must fall below a specified probability P_f, it may be required that the average safety margin $\overline{R-S}$ or $\overline{R}-\overline{S}$ must lie a specified number β of standard deviations above zero, i.e. - FIG 1

$$\overline{R-S} \geq \beta \cdot \sigma_{R-S} \text{ or } \overline{R} \geq \overline{S} + \beta\sqrt{\sigma_R^2 + \sigma_S^2} \qquad (1)$$

σ_{R-S} is the standard deviation of the safety margin R-S, σ_R and σ_S are the standard deviations of R and S, respectively.

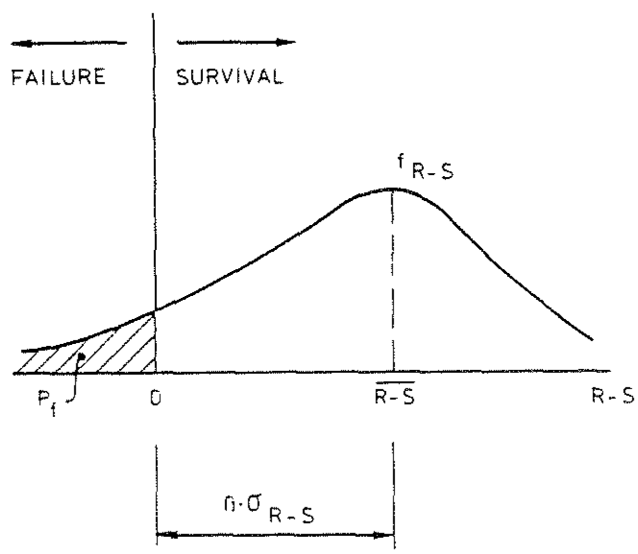


FIG 1. Probability density function f_{R-S} of safety margin R-S and definition of safety index β. Dashed area gives the failure probability P_f.

The method is distribution-free and employs only the first and second central moments of relevant stochastic variables, hence the name "second moment code formats".

The safety index β defines the reliability of, for instance, a design system and offers a quantitative basis for comparing the relative safety of two or more design alternatives. A greater value of β then corresponds to a higher level of safety. With this safety measure we can improve our design methods to be more consistent and assess the implications of assumptions and guesses.

The random variables R and S are invariable functions of other, more basic variables. The problem is to derive the means and variances of R and S from the first and second moments of the basic variables. Exact calculation is only possible when the functional relation between the two sets of variables is a linear transformation. In all other cases, approximate methods must be used. A convenient method is to make a Taylor expansion of R and S with the derivatives evaluated at the mean values and truncate the expansion at the linear terms. In more complicated cases, the required central moments must be derived by a Monte Carlo simulation.

DETERMINISTIC ANALYTICAL METHODS FOR A STRUCTURAL FIRE ENGINEERING DESIGN

The internationally predominant fire engineering design of load bearing structures and structural members is characterized by a schematic procedure, based on results of standard fire resistance tests and connected systems of classification. FIG 2 describes the procedure. The design comprises a proof that the structure has a fire resistance time t_{fr}, determined in a standard fire resistance test, which exceeds the required time of fire duration t_{fd}, specified in building codes and regulations for different applications.

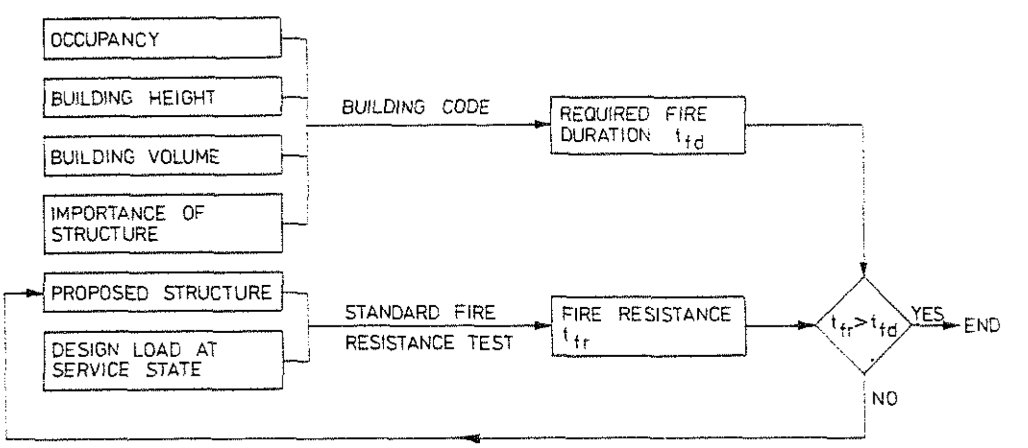


FIG 2. Conventional fire engineering design of load bearing structures and structural members, based on classification and results of standard fire resistance tests.

The fire resistance t_{fr} and the required fire duration t_{fd} then are connected to a thermal exposure, which shall vary with time within specified limits according to the relationship:

$$T-T_0 = 345 \log_{10} (8t+1) \qquad (2)$$

where

t = time, in minutes,
T = temperature at time t, in ^{o}C,
T_0 = temperature at time t=0, in ^{o}C.

During the last ten years important progress can be noted in the development of computation methods for an analytical structural fire engineering design. Internationally, the approach then can be categorized with reference to the following levels:

(1) A theoretical determination of the fire resistance of a load bearing structure, based on a thermal exposure according to the standard fire resistance test, Eq (2) - methods of <u>level 1</u>,

(2) an analytical design, directly based on the gastemperature-time curves of a natural compartment fire, specified with regard to the properties of the fire load and the fire compartment (10), (11) - methods of <u>level 3</u>,

(3) an analytical design, based on the gastemperature-time curves of a natural compartment fire, but taken into account indirectly over an equivalent time of fire duration, connected to the heating according to the standard fire resistance test, Eq (2), (10), (11), (12), (13) - methods of <u>level 2</u>.

(14) briefly describes the different approaches and demonstrates how useful input information for the analytical design methods can be derived from the results of standard fire resistance tests. The consistency between the various methods is critically reviewed and ways are indicated for improving this consistency. Although, the different analytical and experimental design methods have been developed mainly independently and rather frequently have been discussed as contradictory to each other, it is quite clear, that the methods in a long-term perspective will form a well coherent pattern.

Internationally, the development goes towards an increased practical use of design methods of level 3. In Sweden, an analytical procedure of this level is officially approved for a general practical application, as one alternative, since about ten years. Design methods of level 3 also constitute the natural deterministic basis for a further development of reliability based design methods.

STRUCTURAL FIRE ENGINEERING DESIGN ACCORDING TO LEVEL 3

A structural fire engineering design according to level 3 means a direct design based on temperature characteristics of the fully developed compartment fire as a function of the fire load density and the properties of the fire compartment. FIG 3 describes the design method in a summary way.

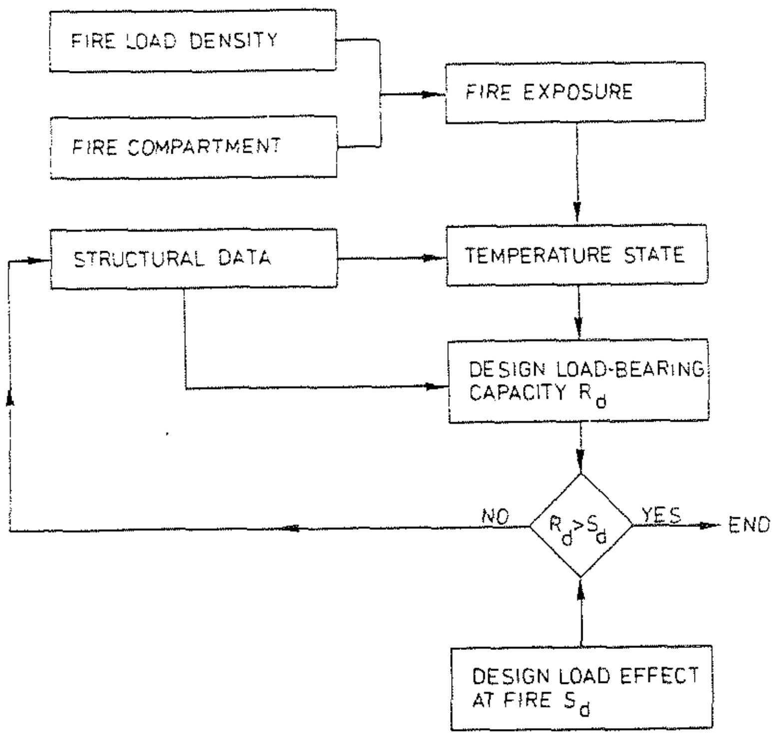


FIG 3. Summary description of a rational design method for fire exposed load bearing structures according to level 3.

The design starts by a determination of the fire exposure, given by the gastemperature-time curve of the natural compartment fire. The combustion characteristics of the fire load and the geometrical, ventilation and thermal properties of the fire compartment are the decisive influences.

For a given structural design, the fire exposure is transferred analytically to transient temperature fields of the structure. In the next step, a determination is carried out of the time variation of the load bearing capacity of the fire exposed structure. The lowest value of this load bearing capacity during the relevant fire process defines the design load bearing capacity R_d.

Nominal loads and load factors for dead load, live load, etc, statistically representative of a fire occasion, specify the design load effect at fire S_d.

A direct comparison between the design load bearing capacity R_d and the design load effect at fire S_d decides whether the structure can fulfil its required function or not at a fire exposure.

Fire load density and gastemperature-time curves of fully developed compartment fire

At known combustion characteristics of the fire load, the gas temperature-time curve of a fully developed compartment fire can be calculated in the individual practical application from the heat and mass balance equations of the fire compartment with regard taken to the size, geometry and ventilation of the compartment, and to the thermal properties of the structures enclosing the compartment - FIG 4 (10), (11), (15), (16), (17), (18).

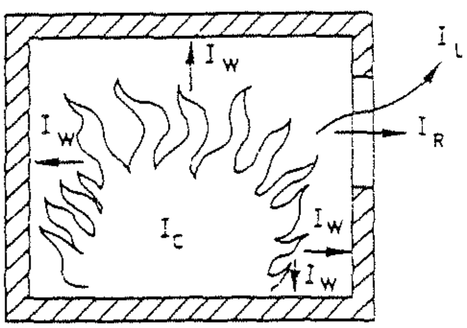


FIG 4. Energy balance equation $I_C = I_L + I_W + I_R$ of a fire compartment. I_C is the heat release per unit time from the combustion of the fuel, and I_L, I_W and I_R the quantities of energy removed per unit time by change of hot gases against cold air, by heat transfer to the surrounding structures, and by radiation through the openings of the compartment, respectively.

Provisionally, the Swedish building code permits the structural fire design to be based on gastemperature-time curves $T_t - t$ according to FIG 5, which applies to a fire compartment with surrounding structures made of a material with a thermal conductivity $\lambda = 0.81\ W \cdot m^{-1} \cdot {}^{o}C^{-1}$ and a heat capacity $\rho c_p = 1.67\ MJ \cdot m^{-3} \cdot {}^{o}C^{-1}$ (fire compartment, type A). Entrance parameters of the diagrams are the fire load density q, defined by the formula

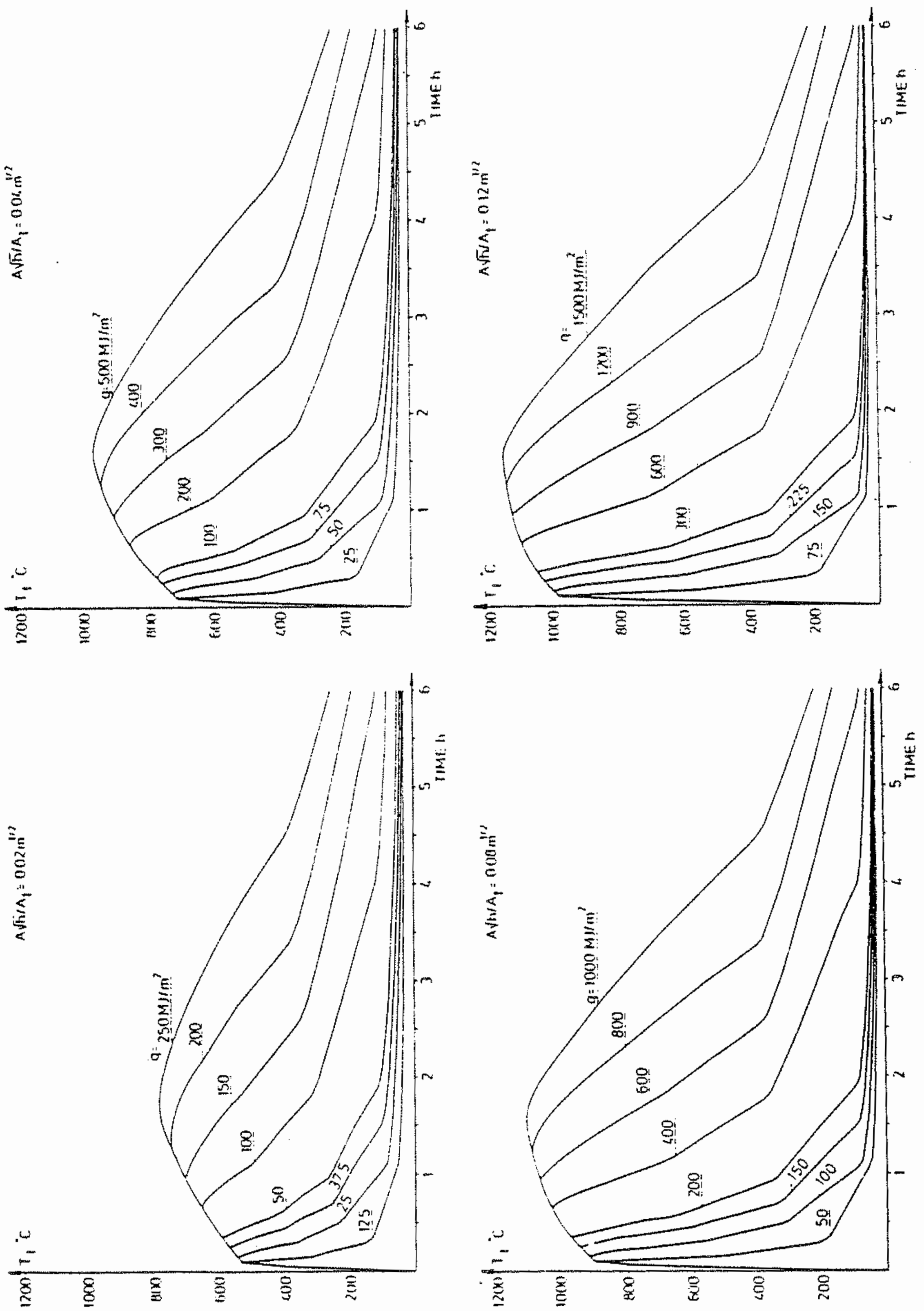


FIG 5. Gas temperature-time curves T_t-t of the complete process of fire development for different values of the fire load density q and the opening factor $A\sqrt{h}/A_t$. Fire compartment, type A.

$$q = \frac{1}{A_t} \Sigma \mu_\nu m_\nu H_\nu \qquad (MJ \cdot m^{-2}) \tag{3}$$

and the ventilation characteristics of the fire compartment, expressed by the opening factor $A\sqrt{h}/A_t$ $(m^{1/2})$,

where

A = total area of window and door openings (m^2),
h = mean value of the heights of window and door openings, weighed with respect to each individual opening area (m),
A_t = total interior area of the surfaces bounding the compartment, opening areas included (m^2),
m_ν = total weight of combustible material ν (kg),
H_ν = effective heat value of combustible material ν of the fire load $(MJ \cdot kg^{-1})$, and
μ_ν = a fraction between 0 and 1, giving the real degree of combustion for each individual component of the fire load.

As a rule, the design fire load density is to be determined on the basis of statistical investigations for the type of building or premises in question. Such statistical investigations have been carried out for dwellings, offices, administration buildings, schools, stores, and hospitals (10), (11). As a temporary regulation, the Swedish building code authorizes the 80 percent level of the statistical distribution curve to be applied as the design fire load density.

The gas temperature-time curves in FIG 5 have generally been determined on the assumption of ventilation controlled fires. For fires, which are fuel bed controlled in reality, this assumption leads to a structural fire engineering design on the safe side in practically every case, giving an overestimation of the maximum gastemperature and a simultaneous, partly balancing underestimation of the fire duration. For the minimum load bearing capacity, which thermally can be seen as an integrated effect, the gas temperature-time curves in FIG 5 give reasonably correct results, verified in (10), (16).

As pointed out, the gas temperature-time curves in FIG 5 apply to a certain fire compartment, type A, specified with respect to the thermal properties of its surrounding structures. Fire compartments with surrounding structures of deviating thermal properties can be transferred to fire compartment, type A, via effective values of the fire load density q_f and the opening factor $(A\sqrt{h}/A_t)_f$ - (10), (11).

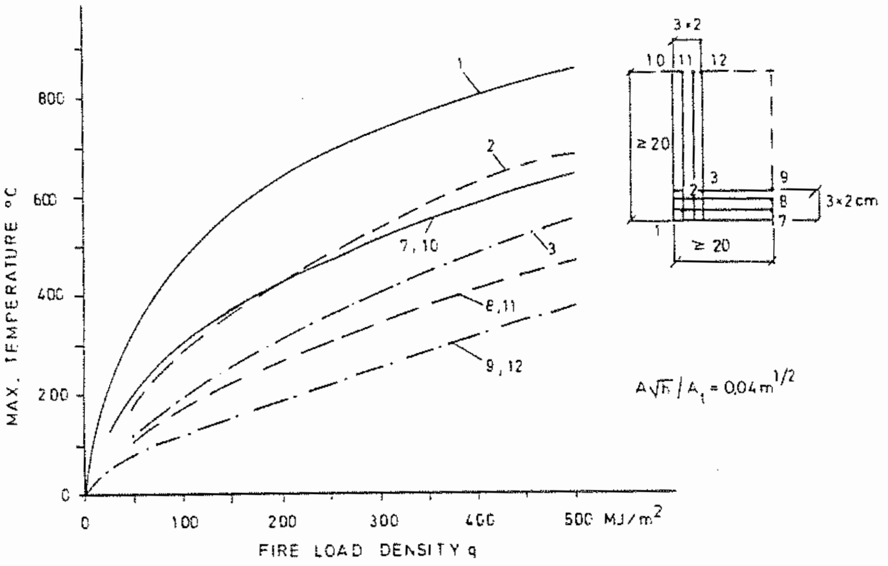
An analytical design according to the described procedure can be carried through in practice today in a comparatively general extent for fire exposed steel structures. Validated material models for the mechanical behaviour of concrete under transient high-temperature conditions and thermal models for a calculation of the time variation of the charring rate in wood at a fire exposure, derived during the last years, now are enabling an essential enlargement of the area of application. To aid this application, design diagrams and tables are systematically produced giving, directly, on the one hand, the design temperature state of the fire exposed structure, and on the other, a transfer of this information to the corresponding design load bearing capacity of the structure; cf., for instance (10), (11).

Temperature state and design load bearing capacity of fire exposed concrete structures

For a practical determination of the transient temperature state in fire exposed structures, numerical methods have been developed and arranged for computer calculations. The methods are based either on finite difference (19), (20) or on finite element approximations (21), (22). In application to concrete structures, the methods have to start out from approximations of the thermal properties at elevated temperatures and of the moisture transport and evaporation. The methods are enabling a systematic determination of a design basis in the form of diagrams and tables, giving directly, for instance, the maximum temperature in different points of a concrete beam during a complete fire process at varying values of the fire load density q and the opening factor $A\sqrt{h}/A_t$ - FIG 6.

A transfer of the temperature-time fields of a fire exposed concrete structure to data on the structural behaviour and load bearing capacity requires in the general case a qualified knowledge on the strength and deformation properties of the concrete and the reinforcing steels in the temperature range associated with fires. Comparatively detailed information then is available for some types of reinforcing steels, as concerns stress-strain relation and short-time creep at elevated temperatures and residual strength. For concrete, the deformation behaviour at elevated temperature is much more complicated than for steel and consequently the present state of knowledge is more incomplete.

An accurate analysis of the stress and deformation behaviour of a fire exposed concrete structure implies that the constitutive relations between stresses and strains are known, the time-dependent behaviour included. In comparison with metallic or ceramic materials, stressed concrete then presents special difficulties in that respect that during the first heating considerable deformations develop which do not occur at stabilized temperature. This effect has been confirmed in flexural, torsional and compressive tests and for moderate as well as high temperatures.



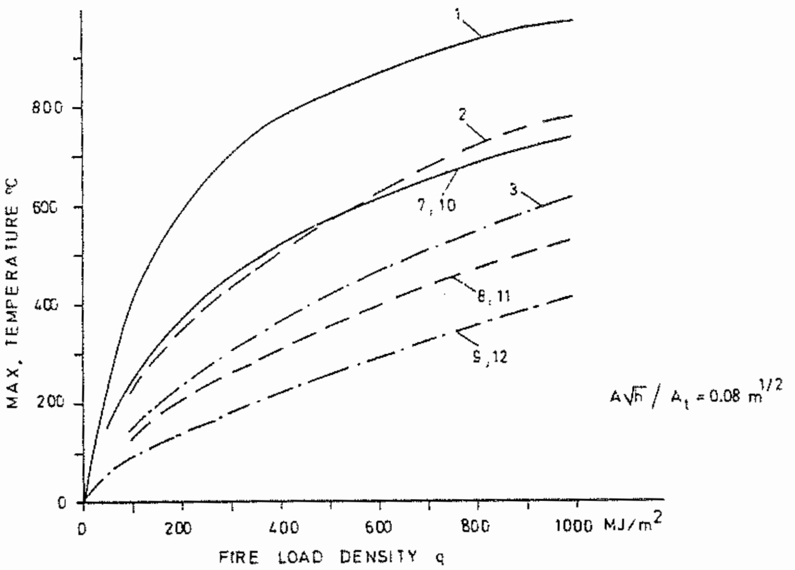


FIG 6. Maximum temperature in different points of a concrete beam of rectangular cross section, fire exposed from below on three sides. Fire exposure according to FIG 5, differentiated with respect to the fire load density q and the opening factor $A\sqrt{h}/A_t$ of the fire compartment.

For practical applications, the total strain ε can adequately be given as the sum of a number of strain components, phenomenologically defined with reference to specified types of test and depending on the temperature T, the stress σ, the stress history $\tilde{\sigma}$ and the time t. For concrete stressed in compression this leads to the relation (23)

$$\varepsilon = \varepsilon_{th}(T) + \varepsilon_{\sigma}(\tilde{\sigma},\sigma,T) + \varepsilon_{cr}(\sigma,T,t) + \varepsilon_{tr}(\sigma,T) \quad (4)$$

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} ordinarily plays a predominant part. This is illustrated in FIG 7, showing how the total strain ε is composed of the thermal strain ε_{th}, the instantaneous stress-related strain ε_{σ}, the creep strain ε_{cr} and the transient strain ε_{tr} for a concrete specimen, initially stressed in compression and heated to failure.

A validated model for the mechanical behaviour of concrete under transient, high-temperature conditions of the type described constitutes a prerequisite for getting reliable results by applying the mathematical models and connected computer programs available for an analysis of fire exposed concrete beams and frames. The most comprehensive program probably then is Fires-RC, developed by Becker and Bresler (24) for a calculation of the fire response of reinforced concrete frames. With the frame divided into substructural members, segments from substructure, concrete subslices and reinforcing bars, the computer program is capable of providing a broad spectrum of response data, including the time history of displacements, internal forces and moments, stresses and strains in concrete and in steel reinforcement, as well as the current states of concrete with respect to cracking or crushing and steel reinforcement with respect to yielding.

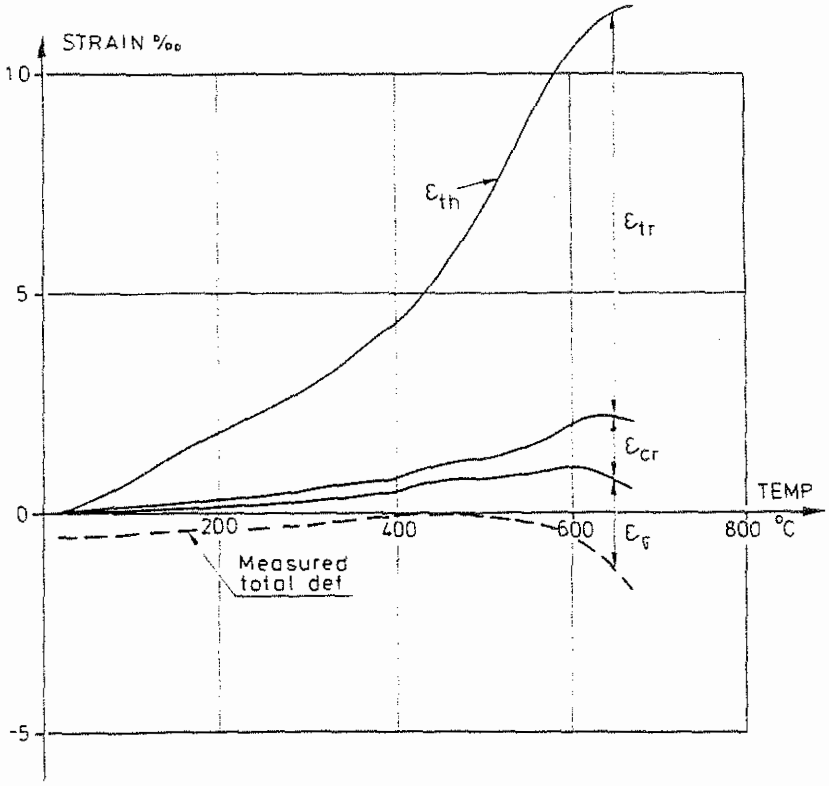


FIG 7. Relation between different strain components for a concrete specimen, initially stressed in compression to a stress level of 35% of the strength at 20°C, and heated to failure (23).

Fires-RC has been modified by Anderberg for a theoretical analysis of hyperstatic concrete beams, fire exposed from below (25). FIG 8 gives a fragmentary result from this study, showing the calculated structural behaviour of an unloaded, respectively a loaded plate strip restrained against rotation as well as longitudinal movement at both ends. The full-line curves give the time history of the bending restraint moment and the dashed curves the time history of the axial force (FIG a). FIG b shows the corresponding midpoint deflection.

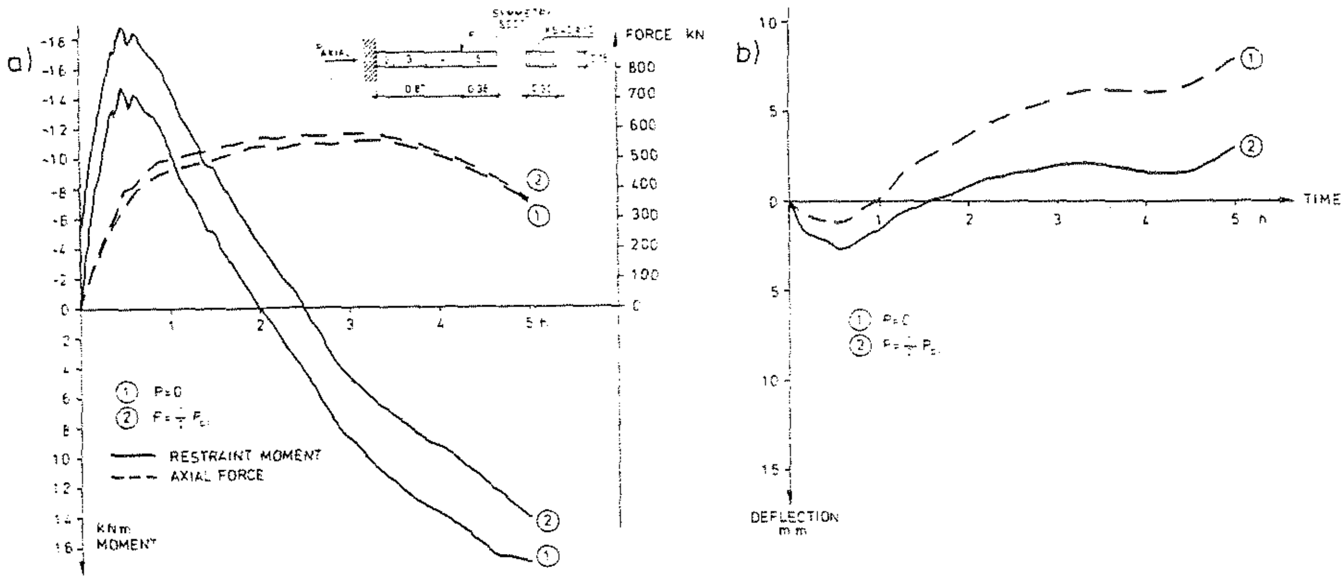


FIG 8. Structural fire behaviour of an unloaded, respectively loaded plate strip restrained against rotation as well as longitudinal movement at both ends. a) Bending restraint moment and axial restraint force, b) midpoint deflection. Fire process characteristics: $q = 500\ MJ \cdot m^{-2}$, $A\sqrt{h}/A_t = 0.04\ m^{1/2}$ according to FIG 5 (25).

RELIABILITY BASED STRUCTURAL FIRE ENGINEERING DESIGN METHODS UNDER DEVELOPMENT

Up to now, there are only two reliability based structural fire engineering design methods reported in the literature, as concerns methods of design level 3 (9), (26), (27). The methods are still under further improvement.

Both methods relate to a design for the ultimate limit state. One of them - a German model code draft (26), (27) - belongs to the probabilistic approach on level II, based on the "second moment code formats". The functional requirement implies that the minimum value of the safety index during the relevant fire exposure $\beta_{fe,min}$ has to meet the required value of the safety index β_r, derived by a probabilistic analysis, i.e.,

$$\beta_{fe,min} - \beta_r \geq 0 \quad (5)$$

The other method - a development of the Swedish design method according to FIG 3 (9) - is related to a semi-probabilistic approach on level I, based on a system of partial safety coefficients. The functional requirement then implies that the design value of the minimum load bearing capacity of the structure during the fire exposure R_d shall meet the design load effect on the structure S_d, i.e.

$$R_d - S_d \geq 0 \qquad (6)$$

The functional requirements apply to all relevant types of failure - bending failure, shear failure, instability failure, etc.

In the design, the following probabilistic influences should be taken into consideration:

* the uncertainty in specifying the statical loading,

* the uncertainty in specifying the fire load and the characteristics of the fire compartment,

* the uncertainty in specifying the thermal and mechanical properties of the structural materials,

* the uncertainty of the models for calculation of the compartment fire, the heat transfer to and within the structure and the ultimate load bearing capacity of the structure,

* the consequences of a structural failure.

In the German model code draft, the consequences of a structural failure are included in the required value of the safety index β_r. In the Swedish approach, these consequences are related indirectly to the design load bearing capacity R_d via a differentiation of the design fire load density and the length of the fire process, to be considered in the design.

Swedish reliability based design

In a summary way, the Swedish design procedure under development can be described as follows - FIG 9.

The design fire load density, the fire compartment characteristics, and the fire extinguishment and fire fighting characteristics constitute the basis for the determination of the design fire exposure, given as the gas temperature-time curve T-t of the fully developed compartment fire. Depending on the type of practical application, the load bearing function of the structure or structural member will be required to comply with either the complete fire process or a limited part of the fire process t_d, determined from the time necessary for the fire to be extinguished under the most severe conditions, or from the design evacuation time for the building.

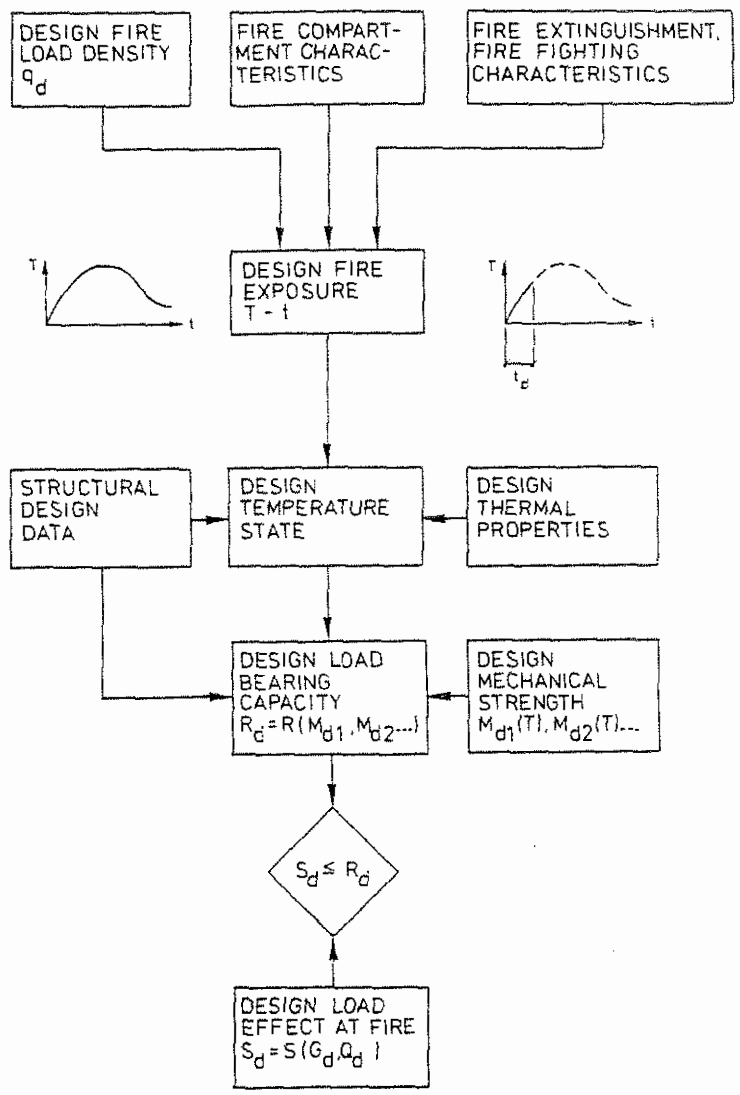


FIG 9. Reliability based, structural fire engineering design procedure under development.

Together with the structural design data, the design thermal properties and the design mechanical strength of the structural material, 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 bearing capacity R_d and the design load effect at fire S_d finally decides whether or not the structure or structural member can fulfil its required function on exposure to fire - Eq (6).

Design loads and design load effect

The design consists of an analysis of simultaneous exposure to static loading and fire, dealt with as an accidental case. The determination of the static loading and the associated design load effect S_d then follows the procedure according to FIG 10. The determination begins with characteristic permanent and variable loads G_k and Q_k. The characteristic value of the permanent load G_k will be chosen as the average, and the characteristic value of a variable load Q_k as that corresponding to a probability of excess at least once a year. The characteristic Q_k values may be differentiated with respect to whether a complete evacuation of people can be assumed or not in the event of fire.

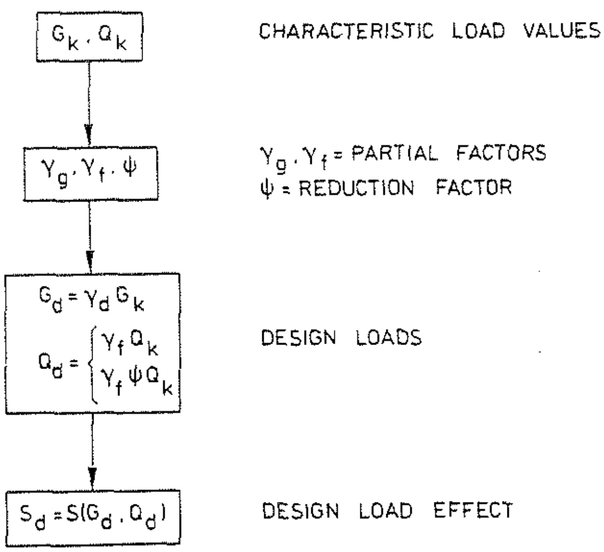


FIG 10. Procedure of determination of design load effect S_d.

A multiplication by partial factors γ and reduction factors ψ transfers the characteristic load values to design loads G_d and Q_d. By using the partial factors γ, the following effects are taken into consideration:

* the probability that the load differs unfavourably from the characteristic value,

* the uncertainty of the model, describing the load - for instance with regard to the distribution of the load over the structure,

* such uncertainties of the design model which are independent of material.

The partial factors γ, furthermore, depend on the type of loading and of the appropriate load combination.

The reduction factors ψ give expression to the relative duration of a variable load.

For values of G_k, Q_k, γ and ψ to be applied in the design, reference is made to the Swedish building code (28).

Categories of structures. Design fire exposure

The functional requirements to be laid down for a fire engineering design should be differentiated with respect to such effects as the occupancy, the height and volume of the building, and the importance of the structure or structural member to the overall stability of the building. This can be done by dividing the structures or structural members into categories, with a related differentiation of the design fire load density q_d, and the length of the fire process, to be considered in the design.

In the version of the design procedure under development, four categories K0, K1, K2 and K3 have been introduced and defined according to TABLE 1. The table relates the different categories and the fire endurance in minutes - F30, F60 and F90 - required in the current design, based on classification and results of standard fire endurance tests, which is to be seen as a procedure of a relative calibration.

Fire endurance in minutes, required in current design, based on classification	Category
-	K 0
F 30	K 1
F 60	K 2
F 90	K 3

TABLE 1. Definition of categories of structures or structural members.

For the different categories, the design fire exposure will be chosen according to TABLE 2, specifying the design fire load density q_d, in relation to the characteristic fire load density q_k, and the duration of the fire process. The characteristic fire load density q_k then is defined as that value corresponding to a probability in excess of 20%. The related gas temperature-time curves of the fire exposure are specified in accordance to FIG 5, with due consideration taken to the influence of the thermal properties of the structures, enclosing the fire compartment.

Category of structural member	Design fire load density q_d	Duration of fire exposure
K 1	1.0 q_k	$\leq$ 30 min
K 2	1.0 q_k	complete fire
K 3	1.5 q_k	process

TABLE 2. Design fire exposure, expressed by the design fire load density q_d.

By specifying the design fire exposure as described, consideration is taken of:

* the probability that the fire load density differs unfavourably from the characteristic value,

* the uncertainty of the analytical model for the determination of the compartment fire and its thermal exposure on the load bearing structure or structural member,

* the uncertainty in specifying the geometry and thermal properties of actual fire compartment materials,

* the safety level required for the respective categories of structure or structural member.

The probability and the consequences of a fire outbreak are strongly influenced by various types of active fire protection measures such as fire detection systems, sprinkler systems, smoke control systems, roof venting systems, fire alarm systems, and the fire fighting facilities of the fire brigade. The present version of the method does not allow for such influences to be included in any sophisticated way in the specification of the design fire exposure. Discussions are in progress concerning whether the presence of an approved sprinkler system could be taken into account in a very rough way by transferring a structure or structural member to the next lower category.

Design load bearing capacity

The calculation of the ultimate design load bearing capacity R_d of a structure or structural member will be based on the design strength values M_d of the actual structural materials. These strength values are given by the corresponding characteristic strength values M_k, divided by a resulting partial factor γ_{mn}. Normally, the characteristic value is put equal to the lower 5 percent fractile, as concerns strength.

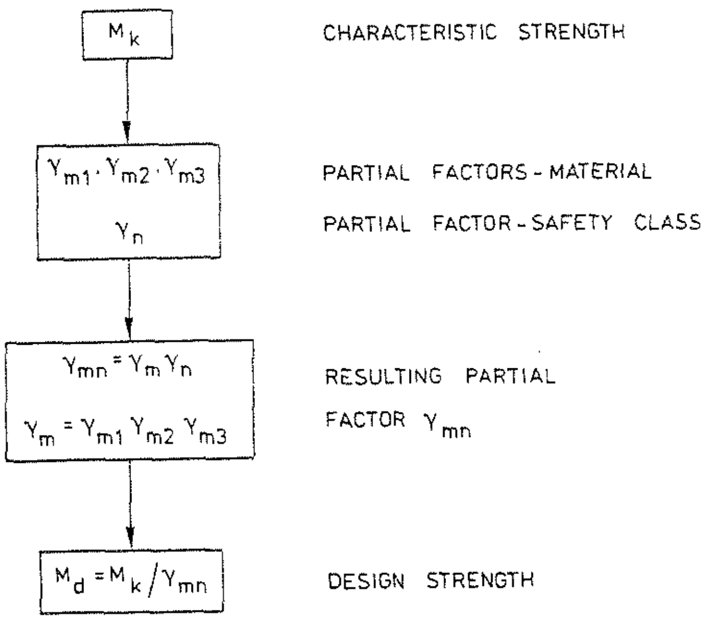


FIG 11. Procedure of determination of design strength M_d at non-fire ultimate limit state.

In a non-fire design for the ultimate limit state, the determination of the design strength follows the procedure according to FIG 11. The different partial factors γ_{m1}, γ_{m2}, γ_{m3} and γ_n are expressing the influence of:

* the probability that the value of the material property differs unfavourably from the characteristic value - γ_{m1},

* the uncertainty of the model for calculation of the ultimate load bearing capacity, including the influence of such deviations of measurements which are not to be considered separately - γ_{m2},

* the uncertainty of the relation between the properties of the material in the structure and the corresponding material properties, determined in the test - γ_{m3},

* the safety class - γ_n.

By introducing various categories of structure and structural members when specifying the design fire load density and the design fire exposure, the influence of different safety classes is already covered. Consequently, the partial factor γ_n is to be made equal to 1 in the fire design.

Values of M_k and γ_{mn}, to be applied in a fire design of reinforced concrete structures, are given in the Swedish regulations (29).

German reliability based design

The German model code draft for a reliability based structural fire design includes methods for design level 3 as well as design level 2 (26). As concerns design level 3, the German model code draft and the Swedish design procedure are in good principal agreement. With respect to their detailed structure, the two methods differ primarily in the way to consider the consequences of a structural failure. In the German method, this influence is included in a more advanced and detailed way in the required value of the safety index β_r by introducing three safety classes with the following definition:

* safety class SK_b3 - members of the main load bearing structure and components bounding the fire compartments,

* safety class SK_b2 - other important structural members,

* safety class SK_b1 - structural members of secondary significance.

To the safety classes, the following failure probabilities per year p_{fi} are allocated, referred to average sizes of fire compartments:

* $p_{f3} = 10^{-6}$ for SK_b3,

* $p_{f2} = 10^{-5}$ for SK_b2,

* $p_{f1} = 10^{-4}$ for SK_b1.

The failure probabilities p_{fi} apply independently of the risk of occurance of a fully developed compartment fire. This risk λ_b may be estimated from the formula

$$\lambda_b \simeq pA \tag{7}$$

where p is the unit area probability and A the area of the fire compartment. The unit area probability p may be described as

$$p = p_1 p_2 p_3 \tag{8}$$

where

p_1 = mean probability of fire occurance per m^2 fire compartment area and year,

p_2 = factor to assess the efficiency of the fire brigade actions,

p_3 = factor to consider the effect of an installed extinguishment system, if any.

In a given design situation, the failure probability p_{fi}, the unit area probability p, and the area of the fire compartment A can be calculated or estimated. This information then can be transferred to a required value of the safety index β_r. FIG 12 exemplifies β_r values, derived for structural members, related to safety class SK_b3.

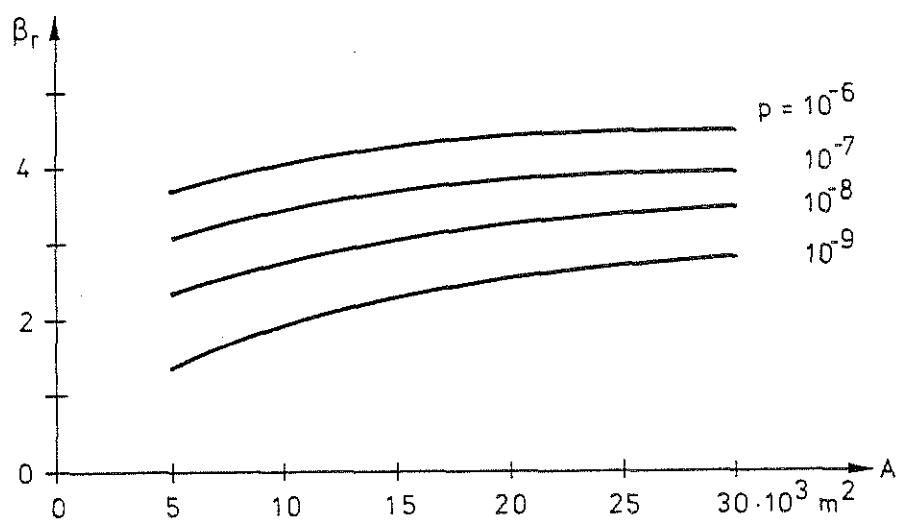


FIG 12. Required values of safety index β_r as function of unit area probability p and area of fire compartment A for structural members of safety class SK_b3 $(p_{fi}=10^{-6})$. The values are representative to German industrial buildings.

In the design, it finally must be proved that the minimum value of the safety index during the fire exposure $\beta_{fe,min}$ meets the required value β_r - Eq (5). The determination of $\beta_{fe,min}$ includes a calculation of

* the gas temperature-time curve of the fire compartment by solving the heat and mass balance equations,

* the transient temperature fields of the load bearing structure or structural member, and

* the corresponding time curve of the load bearing capacity.

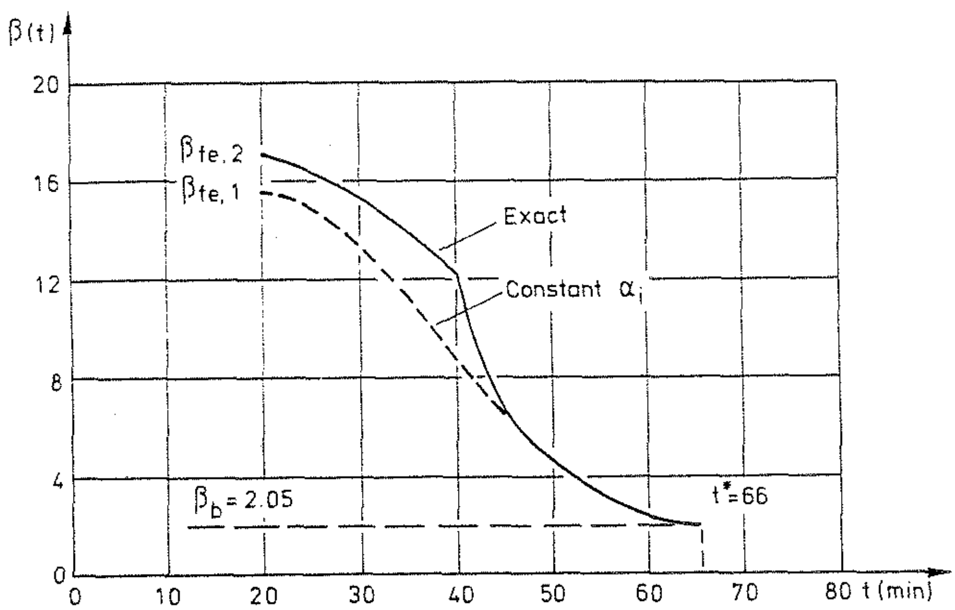


FIG 13. Calculated time curves of safety index β_{fe} for a reinforced concrete column, exposed to a natural compartment fire. The two curves correspond to one accurate and one approximate solution (26).

FIG 13 shows for a specific application - a fire exposed reinforced concrete column in an industrial building - how the calculated safety index β_{fe} decreases with time at a natural compartment fire with an increasing gas temperature during the first 60 minutes, then followed by a cooling down period. The column is functionally reliable as long as $\beta_{fe} \geq \beta_r = 2.05$ which is fulfilled for $t \leq 66$ min.

Summary comparison between German and Swedish design methods

Compared with each other, the German and Swedish reliability based design methods can be generally commented on as follows:

(1) The Swedish method relates to a probabilistic approach on level I, based on a system of partial safety coefficients, and the German method to a probabilistic approach on level II, based on a verification with respect to safety index,

(2) the German method applies to industrial buildings, i.e. large fire compartments, while the Swedish method is restricted to comparatively small fire compartments as in dwellings, schools, offices and hospitals,

(3) the German method requires generally a structural survival, whereas the Swedish method includes two categories of structures and structural members - K0 and K1 - without such a requirement,

(4) the German method considers the influence of the probability of occurance of a fully developed compartment fire and the related efficiency of an installed extinguishment system and of the fire brigade in a more complete and functionally correct way than the Swedish method. However, the merits of the German method in this respect can easily be incorporated in the Swedish method,

(5) in their present forms, the Swedish method enables a relatively quick structural fire design, while the German method is rather time-consuming or needs a computer.

But generally, the two methods are in good principal agreement and both methods have such a pattern that they can be successively improved as knowledge increases.

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