Post-Behaviour of Concrete Structures Subjected to Fire

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State of Art Report - Topic I.4

POST-BEHAVIOUR OF CONCRETE STRUCTURES SUBJECTED TO FIRE

by

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Abstract

On the basis of the general functional requirements of fire exposed load-bearing structures, the characteristics of a differentiated fire engineering design are summarily reviewed in application to ordinary reinforced and prestressed concrete structures. For structures with a requirement on re-serviceability after a fire, a direct differentiated design constitutes the natural solution. On the presumption that realistic data are available on fire exposure and material properties, a differentiated fire engineering analysis also offers the best way for a determination of the residual state of a fire damaged structure with respect to stresses, deformations and load-carrying capacity. The present state of knowledge and the tendencies of development are briefly reported and discussed for the main steps of the differentiated design procedure. Finally, some characteristics are illustrated and exemplified for a more conventional estimation of the condition of a fire damaged structure on the basis of data on the properties of the concrete and reinforcement, determined in tests in situ or on test specimens drilled out or cut off of the structure.

1. General Functional Requirements of Fire Exposed Load-Bearing Structures

The primary objective of all active and passive fire protection measures for a building, a group of buildings or a community is to minimize the risk to life of long-term occupants, casual visitors, and fire fighting people. Occupants and visitors must be protected at a fire with respect to structural collapse of the building and intolerable levels of heat, smoke and toxic gases during an evacuation of the building or during a movement from fire affected areas to safe areas of refuge within the building and a subsequent stay there. Fire fighting people must be guaranteed an equivalent level of safety in connection to rescue and fire fighting operations.

On the basis of this primary objective, the load-bearing and separating structures of a building, which are the limited subject of this state of art report, ought to be designed as an integrated component of the overall fire protection system. Generally, in a fire engineering design it then is to be proved that these structures are able to fulfil the relevant functional requirements during the fire action. For a load-bearing structure that means a proof that the load-bearing capacity does not decrease below the design load or some other prescribed load, multiplied by a stipulated loadfactor, during a required duration of the fire exposure - the complete process of fire development or functionally grounded parts of it. For a separating structure, analogously, the fulfilment of specified functional requirements is to be proved with regard to insulation and integrity.

A further explanation of the philosophy behind the functional requirements within a fire engineering design of load-bearing and separating structures can be given according to Fig. 1 in the following way [1].

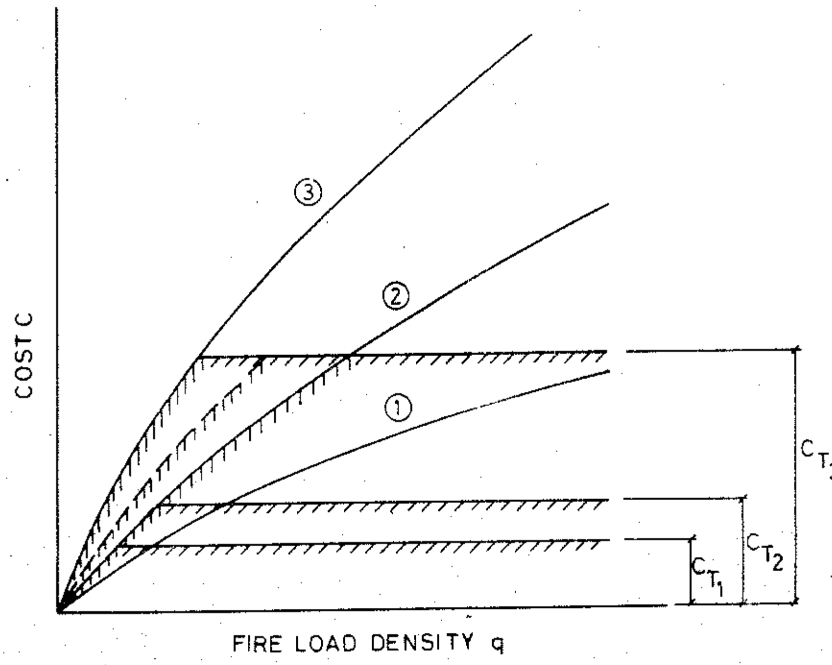

Figure 1. Relationship between cost C and fire load density q for a given type of structure and at given characteristics of fire compartment

In this figure three basic curves (1), (2), and (3) are shown for the relationship between the cost C of a load-bearing or a separating structure and the effective fire load density q. As an alternative, the y axis of the figure could have been referred to the requisite time of fire resistance of the structure, but using the cost C instead gives some simplifications of the discussion. The curves presuppose a given type of structure and a given fire compartment, specified by its geometrical, ventilation and thermal characteristics.

The curve (1) expresses the cost C connected to the shortest time of fire resistance, for which the structure must be designed in order to guarantee the fulfilment of the required function during the heating period of the process of fire development. The analogous curve 2 relates to an increased requirement of a fulfilled function of the fire exposed structure during a complete, undisturbed process of fire development, comprising the heating phase as well as the subsequent cooling phase. Applied to a load-bearing structure the curves 1 and 2 are characterized by the condition that the load-bearing function is to be fulfilled with respect to that level of loading which is representative to the structure from a probabilistic point of view in connection with a fire exposure.

For buildings containing activities, which are particularly important from, for instance, an economical point of view, there can be the motive for a further increase of the requirements on the fire protection measures to such a level that the building can be used again after a fire, almost immediately or very soon, for the current activities in a full extent. For a load-bearing structure then it must be required that the initial load-bearing capacity either will remain approximately unchanged after an exposure to a complete process of fire development or only will be reduced in such a limited extent that it can be restored to its initial value in a short time by a moderate amount of work. The curve 3 in Fig. 1 corresponds to a fire engineering design of a load-bearing structure which fulfils requirements of this level. The design with respect to the re-use of the building after fire then must be carried out for the same loading characteristics as applied to the initial, non-fire design. Fire engineering requirements as expressed by the curve 3 introduce for load-bearing structures and partitions re-service-ability criteria as a complement to the conventional fire resistance criteria [2] .

In ordinary applications, the absolute minimum standard of the fire prevention and the fire fighting measures will be determined by the requirement of a safe emergency evacuation of people at a fire or a safe personal movement from fire affected areas to areas of refuge. For most buildings a complete evacuation of the people will be the actual alternative. In such a case, the requirement of a safe emergency evacuation of the building means that a load-bearing structure or a partition has to fulfil its function during the necessary evacuation time T_1. In a presentation according to Fig. 1, this will lead to a minimum fire resistance and a corresponding cost C, determined by the curve 2 up to the level C_{T_1} and for larger values of the fire load density q by the horizontal line $C = C_{T_1}$.

For buildings with a content of vital and expensive equipment, there can be a financial motive for an increased minimum standard of the fire prevention and the fire fighting measures corresponding to a guaranteed combined evacuation of the people and parts of the equipment. To such a goal belongs a minimum evacuation time T_2 and a connected cost C_{T_2}, which are generally larger than T_1 and C_{T_1}, respectively. This gives a minimum fire resistance and a cost C, determined by the curve 2 up to the level C_{T_2} and for larger values of the fire load density q by the horizontal line $C = C_{T_2}$.

For some types of buildings, for instance tall buildings, the necessary occupant protection at a fire must be solved by a safe personal movement to areas of refuge and a safe subsequent stay there. As a consequence, the requirements of the load-bearing and separating structures must be increased to guarantee their functions at a prescribed safety level during either the complete process of fire development or the time T_3 necessary for the fire to be extinguished under the most severe conditions. With reference to Fig. 1, this will give a minimum fire resistance and a corresponding cost C which will be determined by either the curve 2 or the curve 3 or some other prescribed curve between them up to the level C_{T_3} and then for larger values of the fire load density q by the horizontal line $C = C_{T_3}$. The cause for such a requirement can also be dictated by the necessity of a safety against collapse of a fire exposed structure with regard to the fire brigade people engaged in fire fighting. If then a fire exposed structure has a larger residual load-bearing capacity after cooling than the smallest load-bearing capacity of the heated structure, the requirement also guarantees the safety for the people who have to clear the building after the fire. If, however, the load-bearing capacity of a fire exposed structure continues to decrease during the cooling phase of a fire, the minimum fire resistance of the structure must be higher than that corresponding to the level C_{T_3} in order to give the necessary safety for the clearing people.

In those applications, for which requirements must be put forward with respect to re-serviceability of the building after a fire, a determination of the residual capacity of the load-bearing structures and partitions must be included in the primary structural fire engineering design. For structures designed on the basis of very low requirements of fire resistance - according to the level C_{T_1} in Fig. 1 - it is to be expected that the structures ordinarily will be damaged too strongly at a fire for enabling a repair within a reasonable cost. For intermediate applications, the residual state and strength of the structures after a fire must be analysed in each specific case for a judging of the prerequisites for a re-use of the building and of the extent of the connected necessary repair. Such an analysis then can be made either in a theoretical way according to the procedure of a differentiated, structural fire engineering design or in a more conventional way, which implies an estimation of the condition of the fire damaged structure on the basis of data on the properties of the concrete and reinforcement, determined in tests in situ or on test specimens of the structure. For structures with a requirement on re-serviceability after a fire exposure, a direct differentiated fire engineering analysis constitutes the natural method of solution.

2. Differentiated, Structural Fire Engineering Design

For load-bearing structures or structural members, inside a fire compartment, a differentiated fire engineering design has the following characteristics [1], [3] - [6], Fig. 2.

The basis is constituted by a fully developed compartment fire exposure. Decisive entrance quantities then are

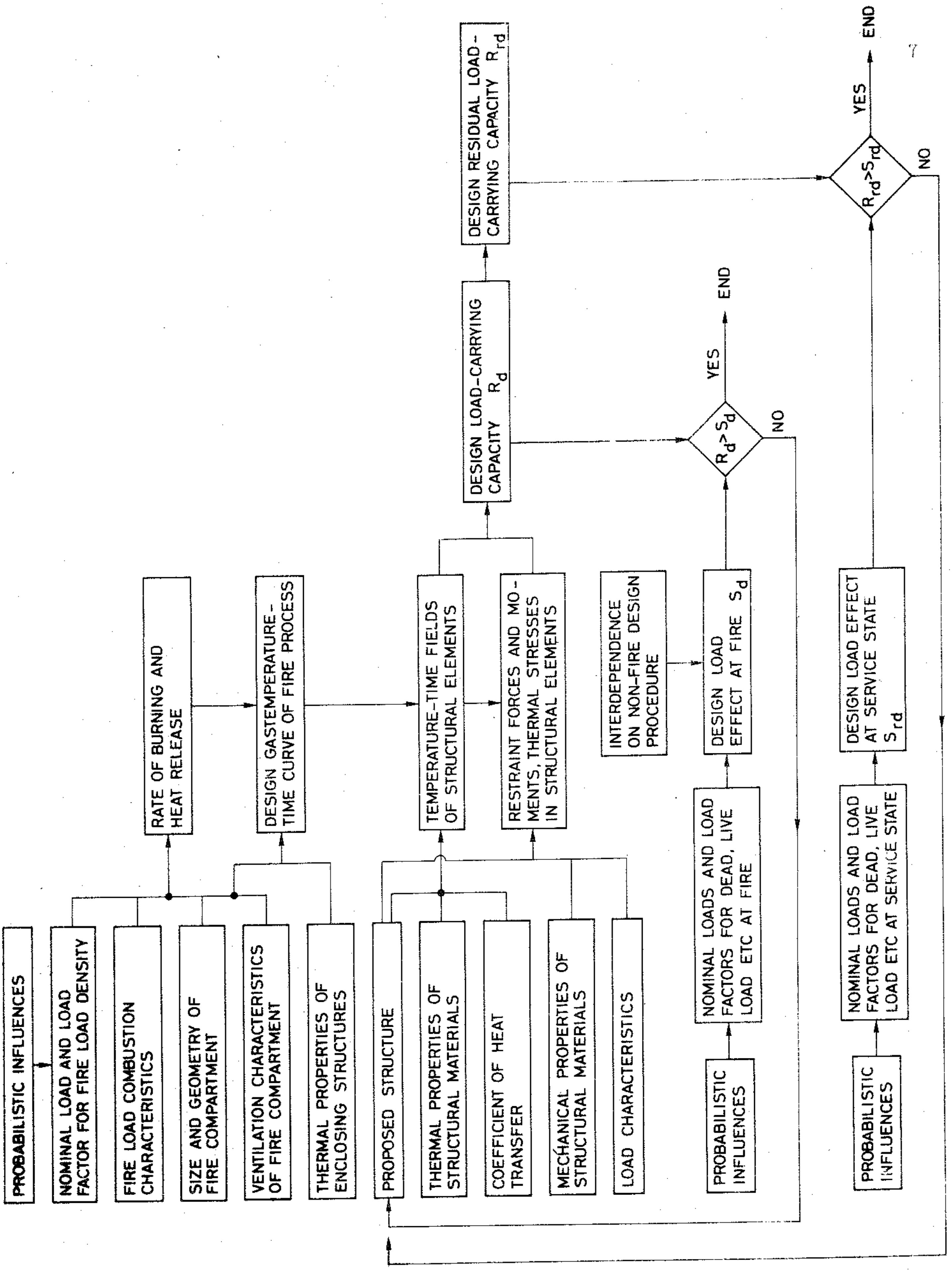

Figure 2. Procedure of a differentiated fire engineering design of load-bearing structures with additional requirement on re-serviceability after fire. Interior structures

the nominal load and load factor for the fire load density,
the combustion properties of this design fire load,
the size and geometry of the fire compartment,
the ventilation characteristics of the fire compartment, and
the thermal properties of the structures enclosing the fire compartment.

Jointly, these quantities are determining the rate of burning, the rate of heat release, and the design gastemperature-time curve of the complete fire process.

Together with

the structural data of the proposed structure,
the thermal properties of the structural materials, and
the coefficients of heat transfer for the various surfaces of the structure

this gastemperature-time curve of the fire compartment gives the requisite information for a determination of the temperature-time fields of the fire exposed structure or structural members. With

the mechanical properties of the structural materials, and
the load characteristics

as further entrance quantities, then a determination can be carried through of the time variation of the restraint forces and moments, thermal stresses, and load-carrying capacity R. The lowest value of this load-carrying capacity R of the structure or structural members during the complete process of fire development defines the design load-carrying capacity R_d.

Over nominal loads and load factors for dead load, live load, etc, statistically representative of a fire occasion, a design load effect at fire S_d is defined, interdependent on non-fire design procedure.

A direct comparison between the design load-carrying capacity R_d and the design load effect at fire S_d decides whether the structure or structural members investigated can fulfil their required function or not at a fire exposure.

If the structural fire engineering design also comprises a requirement on re-serviceability of the structure after fire, the design procedure is to be expanded as follows.

From the time curve of the load-carrying capacity R of the fire exposed structure or structural members, calculated on the basis of the temperature-time fields and the time variation of restraint forces and moments and thermal stresses, the design residual load-carrying capacity R_{rd} of the structure after fire is obtained as an end information. This quantity R_{rd} has to be compared with the design load effect at service, non-fire state for the structure S_{rd}, given by the corresponding nominal loads and load factors for dead load, live load, etc.

The same differentiated procedure as described above according to Fig. 2 can also be applied in those cases when the residual state and strength of a structure must be analysed, after a fire has taken place, for a judgement of the prerequisites for a re-use of the building and of the extent of the appurtenant necessary repair. Usually, then the calculation can be based on comparatively accurate, real values of the fire load density instead of design values, determined with respect to the probabilistic influences. As a consequence, the design residual load-carrying capacity will be replaced by a more distinct value.

For fire exposed, exterior, load-bearing structures, the procedure of a differentiated design will be modified according to Fig. 3. The temperature-time fields of such a structure or structural member is determined by a combined radiation and convection exposure from the flames and combustion gases outside the fire compartment as well as by radiation from the interior of the fire compartment through its window openings. The procedure, summarized in Fig. 3, includes the influence on fire exposure of burning parts of exterior walls of the fire compartment and the building.

Generally, as concerns the load factors applied to the nominal

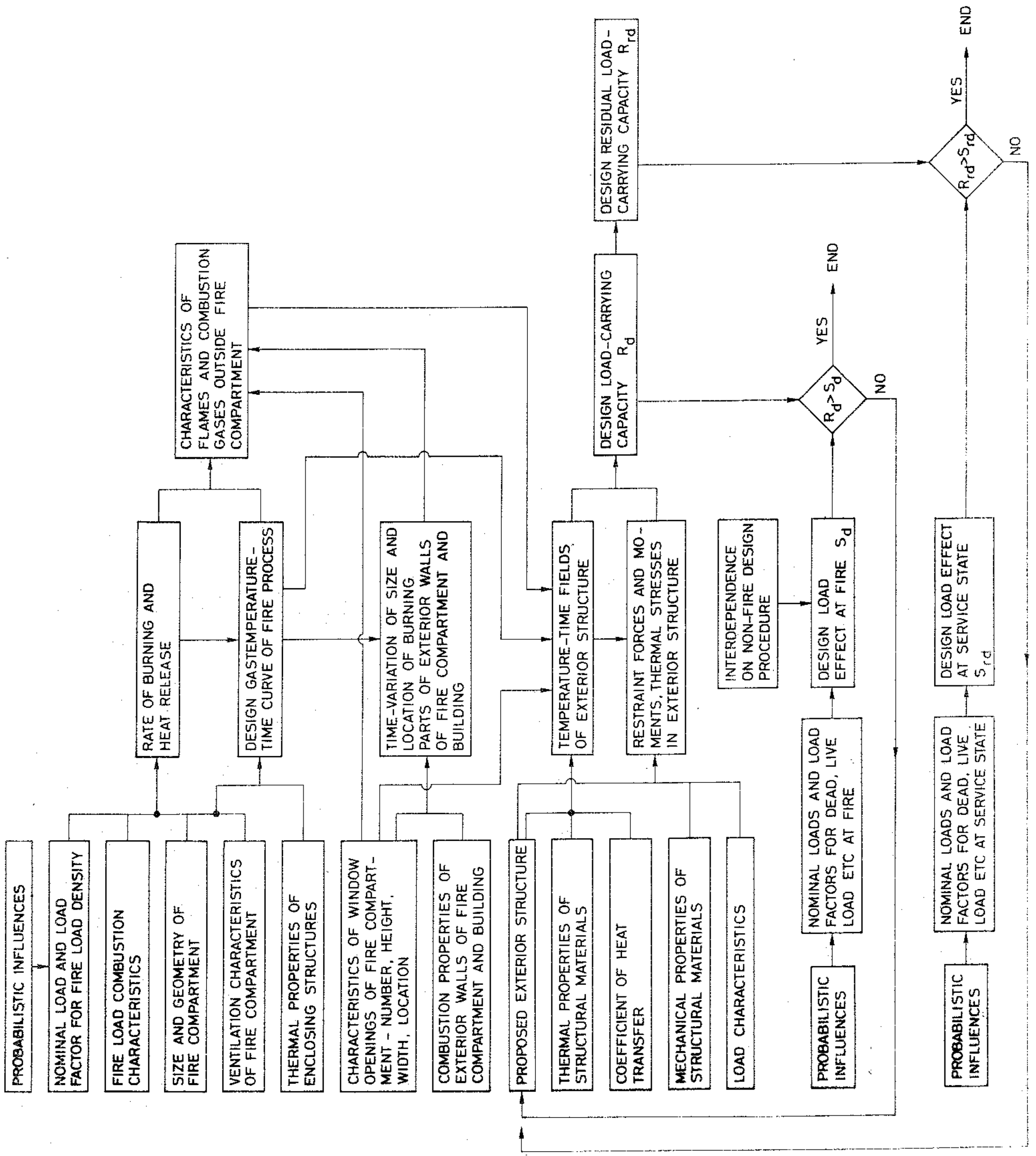

Figure 3. Procedure of a differentiated fire engineering design of load-bearing structures with additional requirement on re-serviceability after fire. Exterior structures

values of fire load density, live load and dead load, these ought to be derived in a statistically consistent way to match a given safety level, defined by, for instance, a safety index [6].

A differentiated design according to Fig. 2 can be carried through in practice today in a general extent for fire exposed, interior, uninsulated and insulated steel structures. It is then also possible to calculate the residual state after a fire with respect to stresses, deformations, and load-bearing capacity. The practical application of the design procedure is facilitated by the availability of tables and diagrams which directly are giving the maximum steel temperature for a differertiated, complete process of fire development and the corresponding load-bearing capacity [7], [8].

In comparison with steel structures, fire exposed reinforced and prestressed concrete structures generally are characterized by an essentially more complicated thermal and mechanical behaviour. In consequence, the basis of a differentiated, structural fire engineering analysis and design is considerably more incomplete for concrete structures - cf, for instance, [9], [10], in which summary reports are given on the present state of knowledge.

2.1 Thermal Properties and Temperature-Time Fields at Fire Exposure

A theoretical determination of the transient temperature fields of a fire exposed concrete structure requires a thorough knowledge of the relevant thermal properties - the thermal conductivity λ and the specific heat c_p, alternatively the enthalpy I, connected to the specific heat c_p through the relation

$$I = \int_0^{\vartheta} c_p \, d\vartheta \qquad (2.1a)$$

ϑ is the temperature.

For normal weight concrete the thermal conductivity λ decreases with increasing temperature. This is illustrated for a granite aggregate concrete in Fig. 4 [11] which also shows the λ variation

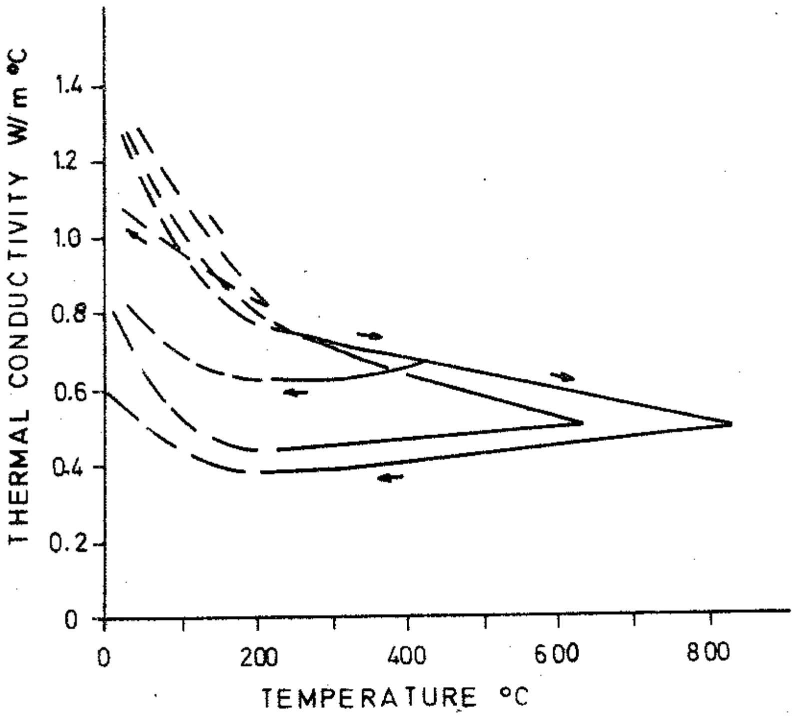

Figure 4. Thermal conductivity λ for concrete with granite aggregate as a function of temperature under heating and subsequent cooling. Cement: aggregate 1:6, w/c = 0.7 [11]

under cooling from different maximum temperature levels. The curves are demonstrating the difference in temperature dependance of the thermal conductivity for an initial heating process and a subsequent cooling process. This difference has to be taken into account in a theoretical fire engineering design, especially in calculating the residual state of a concrete structure after a fire exposure.

The effect of moisture on the thermal conductivity of concrete presents special difficulties. This is relevant for temperatures within the range up to 200°C. Well-defined measurements of λ for moist material in this temperature range are difficult to undertake due to the complicated interaction between moisture and heat flow.

As concerns the enthalpy of concrete, available methods of measurement only can give this quantity versus temperature under cooling. The latent heat of various reactions taking place under the initial heating then is not included. Curve 1 in Fig. 5 shows the

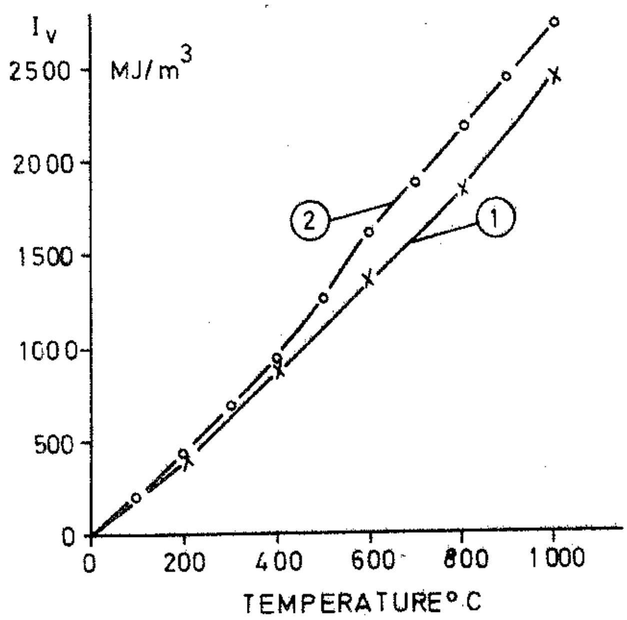

Figure 5. Enthalpy I_v per unit volume as a function of temperature for concrete with granite aggregate. 1 Measured curve under cooling [12], 2 theoretical curve [13]

enthalpy I_v per unit volume measured in this way [12]. Curve 2 gives that variation of the enthalpy which can be expected during heating of concrete without free moisture. The curve has been determined theoretically on the basis of stochiometric calculations and simplified assumptions on the chemical reactions [13]. A significant difference between the two curves exists for temperatures above 500°C.

The most important modification of the enthalpy curve measured under cooling, however, is due to the presence of evaporable water. As long as experimental evidence is lacking, the influence of moisture on the enthalpy has to be included in a simplified way in calculating the temperature-time fields at fire exposure. Usually, then it is assumed that all the moisture "boils" at the temperature 100°C with the required heat of evaporation giving a discontinuous step in the enthalpy curve at this temperature. Such a simplification also gives acceptable results for most practical purposes.

In reality, the evaporation of moisture in fire exposed concrete is not comparable to that of a free water surface. Capillary forces, adhesive forces, and interior steam pressure will increase the temperature, when the evaporation takes place. In a fire exposed concrete structure, the moisture distribution is changing continuously during the heating. Principally, it is then not correct to include the effect of free moisture into the thermal properties.

Available methods for a calculation of the transient heat flow within a fire exposed structure are based on the FOURIER equation of heat conduction in non transparent, non porous materials. In the general, three-dimensional case, this equation has the form

$$\frac{\partial}{\partial x}\left(\lambda_x \frac{\partial \vartheta}{\partial x}\right) + \frac{\partial}{\partial y}\left(\lambda_y \frac{\partial \vartheta}{\partial y}\right) + \frac{\partial}{\partial z}\left(\lambda_z \frac{\partial \vartheta}{\partial z}\right) + Q = \rho c_p \frac{\partial \vartheta}{\partial t} \qquad (2.1b)$$

where ϑ is the temperature; Q the rate of heat generation per unit volume; λ_x, λ_y, and λ_z the anisotropic thermal conductivities with respect to heat flow in the x, y, and z directions, respectively; ρ the density; c_p the specific heat; and t the time coordinate.

In application to concrete structures, Eq. (2.1b) constitutes an approximation of the problem. Concrete is classed as a porous material which implies that a heat transfer occurs also by convection and radiation in the pores of the material. Furthermore, the heat transfer is connected to a simultaneous moisture transport and from a strict thermodynamical point of view, these two transport mechanisms have to be analyzed parallelly over a system of partial differential equations.

For a practical solution of the problem of a theoretical determination of the temperature-time fields in fire exposed structures, numerical methods have been developed and arranged for computer calculations. Such numerical methods are based either on finite difference [10], [12], [14] - [16] or on finite element approximations [17] - [20], cf also [9]. In application to concrete structures, the methods have to start out from approximations of the thermal properties at elevated temperatures according to above. The methods are opening the possibilities for systematic determinations of the temperature-time fields for varying conditions of fire exposure and varying structural characteristics, giving a basis in the form of diagrams and tables for facilitating a differentiated structural fire engineering design in practice. The temperature in different points of the cross section of a fire exposed concrete structure, then can be calculated with sufficient accuracy without modeling the reinforcement of the cross section, if the percentage of the reinforcing steel is less than about 4 per cent [15], [20].

A systematized design basis of the described type now successively is produced. A fragmentary example is shown in Fig. 6, giving directly the maximum temperature in different points of a concrete beam of rectangular cross section, fire exposed from below on three surfaces.[1)] The fire exposure is characterized by complete gastemperature-time curves according to Fig. 7 [4], [7], [8], [21], [22], differentiated with respect to the fire load density q and the opening factor $A\sqrt{h}/A_t$ of the fire compartment. A is the total area of the openings, h the mean value of the heights of the openings, weighed with respect to each individual opening area, and A_t the total area of the interior surfaces bounding the fire compartment. The fire load density is referred to unit area of A_t.

2.2 Mechanical Properties and Structural Behaviour of Fire Exposure

A transfer of the temperature-time fields of a fire exposed concrete structure to data concerning the structural behaviour and loadbearing capacity requires an advanced knowledge on the strength and deformation properties of concrete and reinforcing steels in

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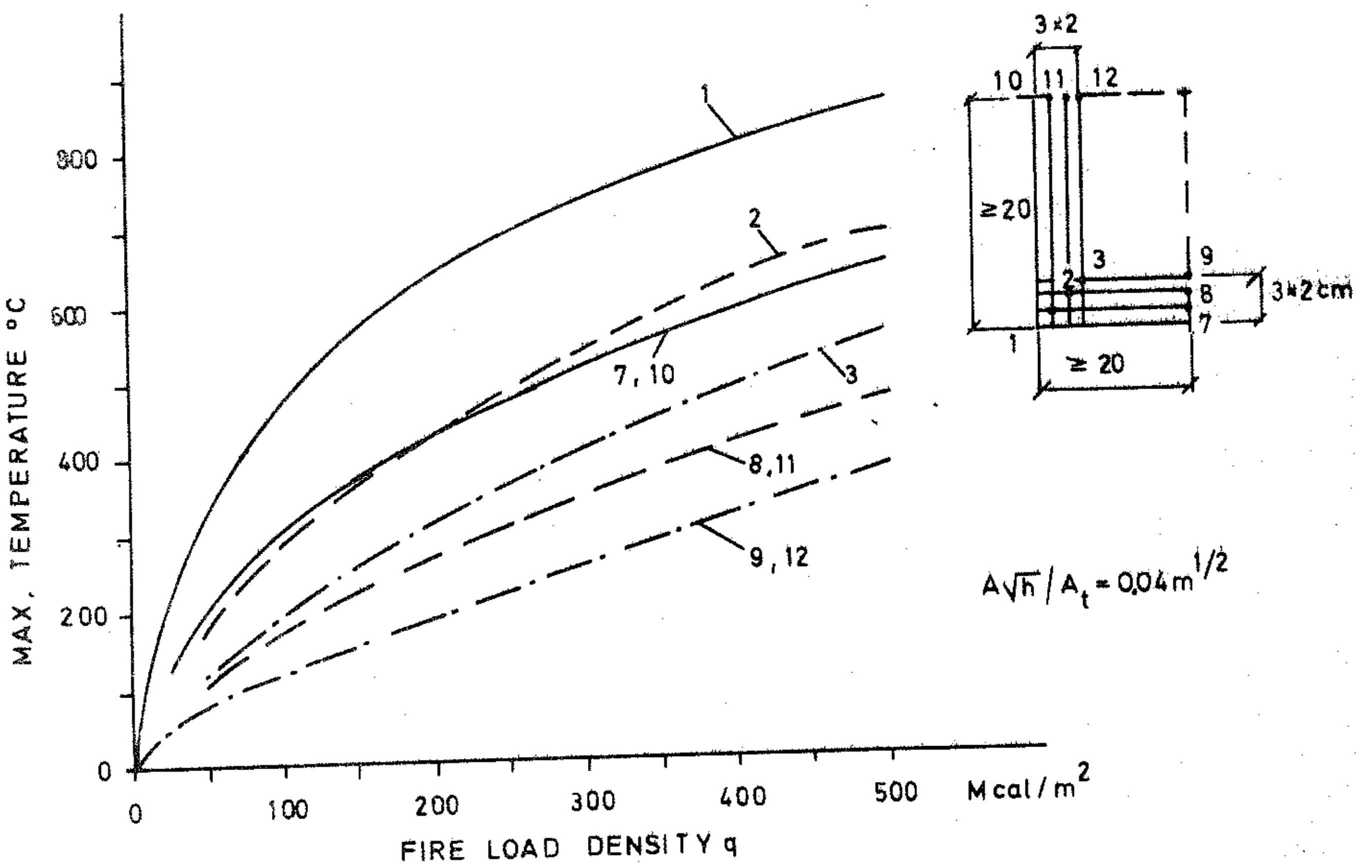
1) From a comprehensive design basis, computed by Ulf Wickström, Lund for a manual to be issued by the National Board of Urban Planning in Sweden



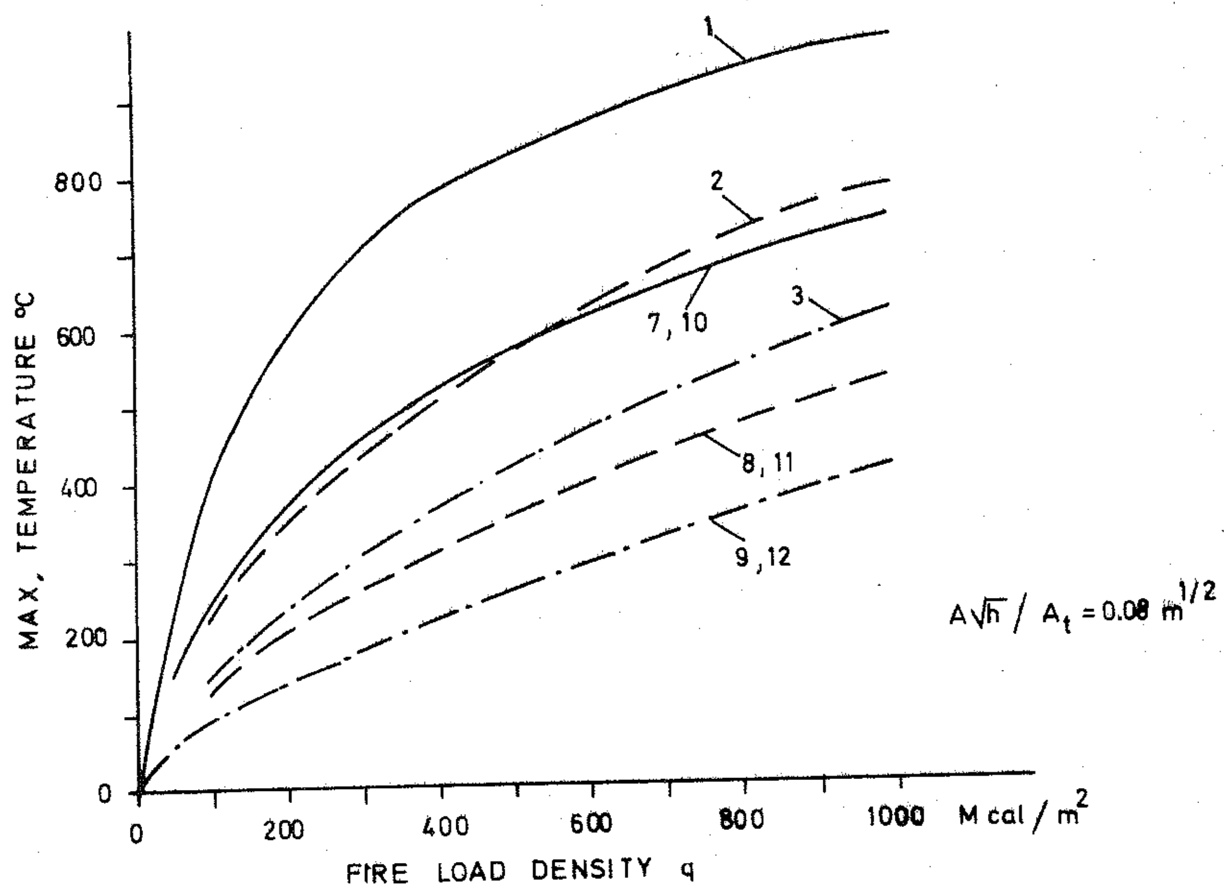
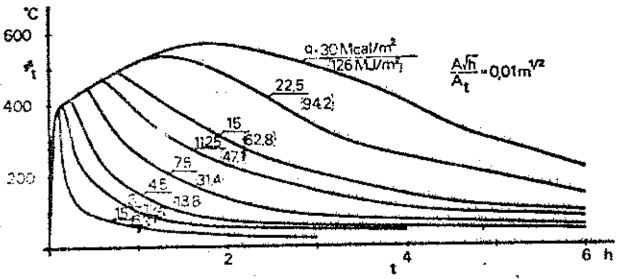
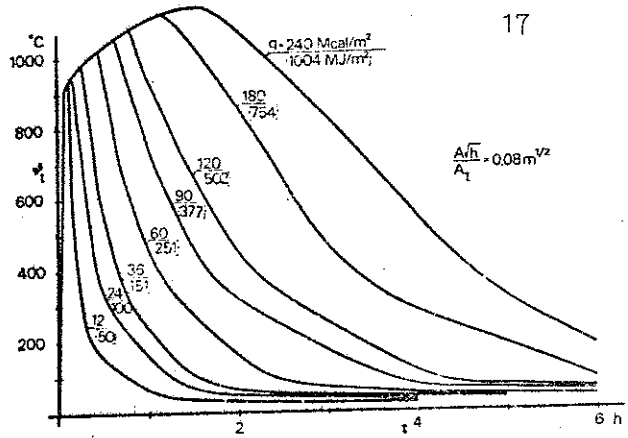
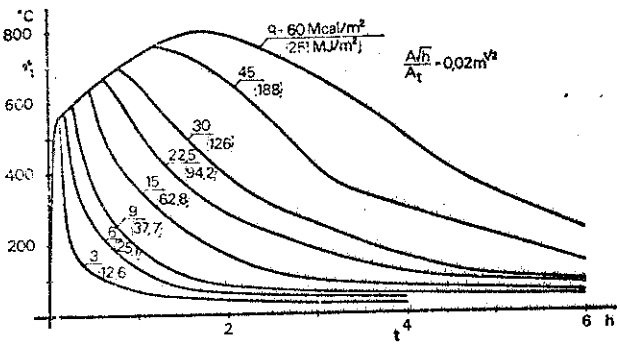
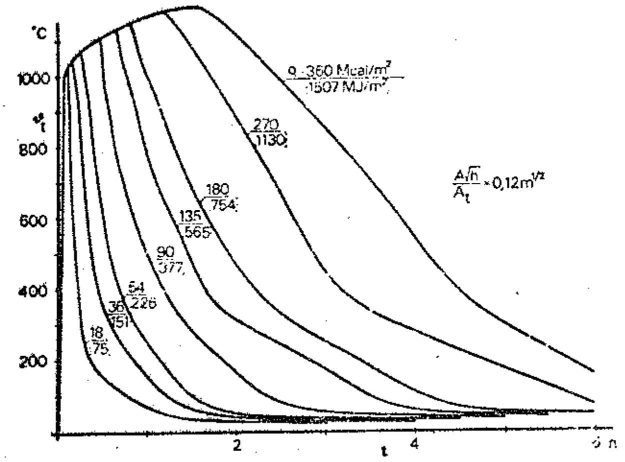


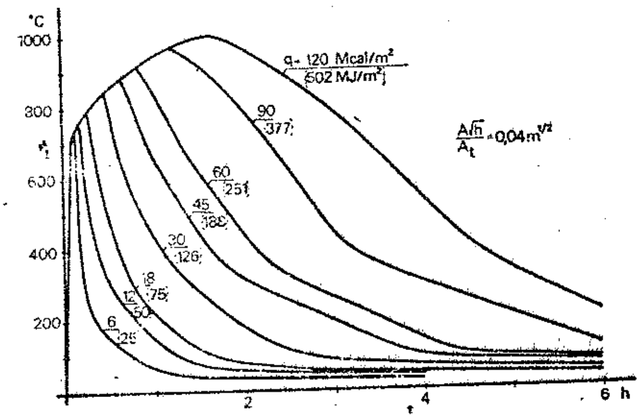
Figure 6. Maximum temperature in different points of a concrete beam of rectangular cross section, fire exposed from below on three sides. Fire exposure according to Fig. 7, differentiated with respect to the fire load density q and the opening factor $A\sqrt{h}/A_t$ of the fire compartment

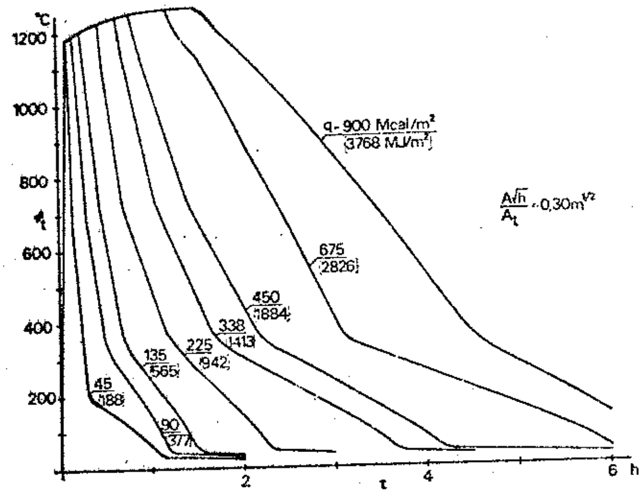












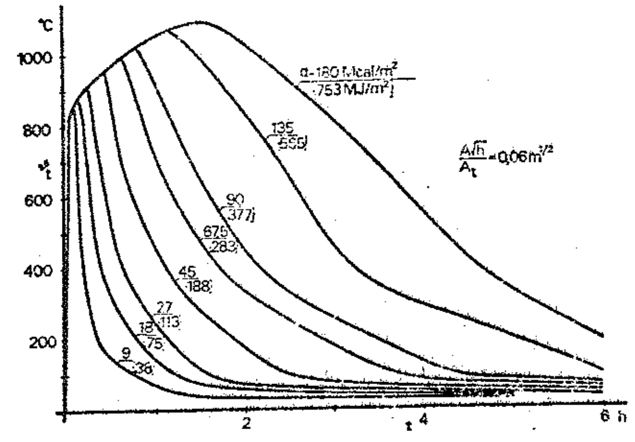


Figure 7. Gastemperature-time curves ϑ_t-t of the complete process of fire development for different values of the opening factor $A\sqrt{h}/A_t$ and the fire load density q [21]. Fire compartment, type A, characterized by surrounding structures of a material with a thermal conductivity λ = = 0.81 W/m°C and a heat capacity ρc_p = 1.67 MJ/m³ °C

the temperature range associated with fires.

Comparatively detailed informations then are available for some types of reinforcing steels, as concerns stress-strain relation, short-time creep, and residual strength [23] - [27].

For concrete, the deformation behaviour at elevated temperatures is much more complicated and far from sufficiently clarified [9], [10]. The various sources of deformation are controlled by a large number of variables and the different types of deformation are not independent of each other. The strain increment in a certain moment depends on the preceding stress and temperature histories.

At elevated temperatures, the material structure of concrete passes through alterations which have a direct influence on the mechanical properties. These alterations are partly due to physical and chemical changes of cement paste and aggregate and partly to interior stresses and crack formations caused by differences in the thermal dilatation of the cement paste and aggregate particles. Important factors of the first group of influences are the vapourization of the nonevaporable water, the dehydration of calciumhydroxide and the quartz inversion, weakening the material structure of concrete.

The possibility of applying an ultimate load approach on these types of fire exposed concrete structures, for which the concrete component has a decisive influence, depends on whether the deformability of heated concrete is sufficient for the redistribution of stresses to take place. Another essential aspect in this connection is the definition of the ultimate stress, since this quantity depends on the previous stress history. In [28] then it is suggested that for ordinary applications the ultimate stress might be determined from tests, where the specimens are first loaded to certain stress levels and then heated until failure occurs.

An accurate analysis of the complete 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.

The first formulation of a realistic constitutive equation for concrete under transient, high-temperature conditions recently has been published by THELANDERSSON [29] in connection with a combined experimental and theoretical study of concrete in pure torsion. The constitutive equation has been derived in terms of the strain components: elastic strain, constant temperature creep strain, and transient strain. The elastic strain is determined by the shear modulus, which is a function of the temperature. The constant temperature creep is the time-dependent strain measured under constant stress and temperature. The third component, the transient strain, is developed only if the temperature increases in the concrete under load. Ordinarily, then the transient strain constitutes the major part of the total deformation.

In [29] the different components of the constitutive equation are formulated mathematically and applied, together with the equilibrium and compatibility equations, in a theoretical analysis of stresses and strains in a concrete prism of circular cross section under pure torsion at transient, high-temperature conditions. A remarkably good agreement between experimental and theoretical results for a wide range of loading and temperature conditions proves the reliability of the mathematical model - Fig. 8 [29].

A development of qualitatively equivalent and realistic constitutive models for concrete under other types of stresses, primarily compression and tension, at transient, high-temperature conditions is an important task for future research. Then the thermal expansion and shrinkage have to be added as further strain components.

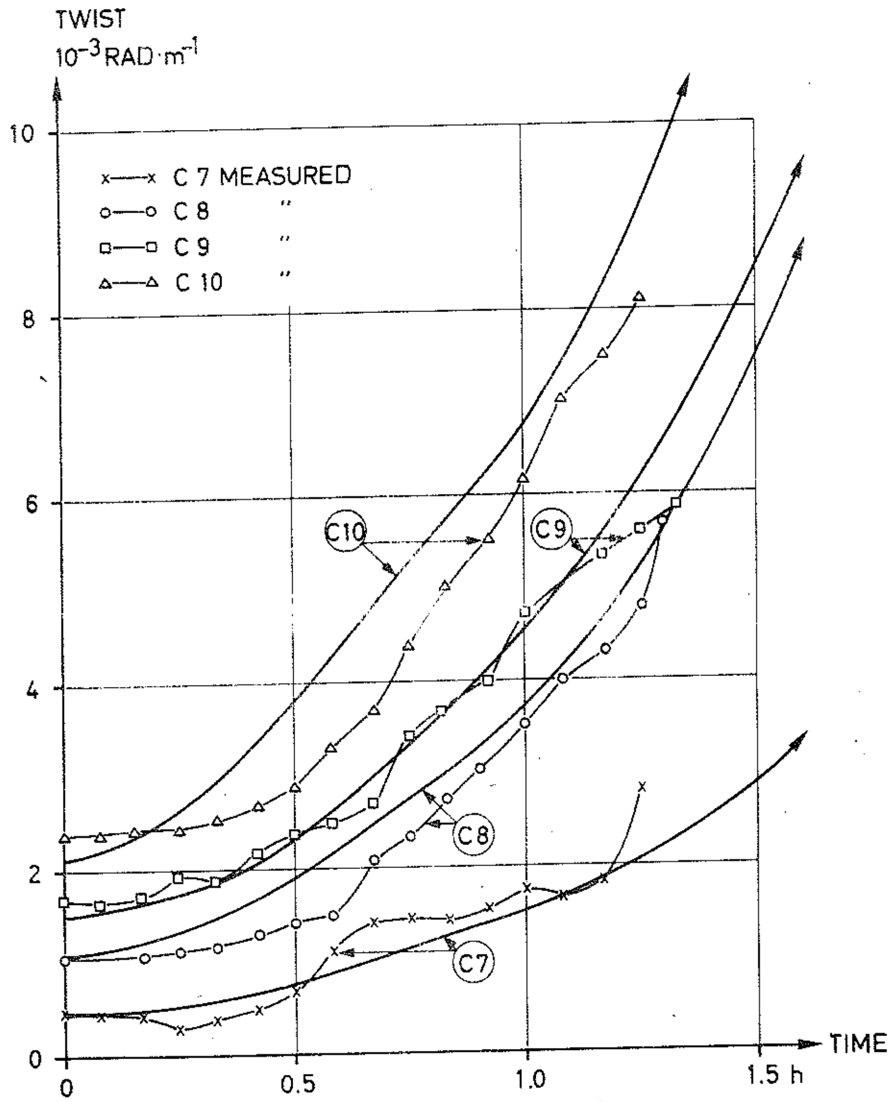


Figure 8. Theoretical and experimental twist-time curves for concrete prisms under pure torsion. Load level: 15 (C7), 30 (C8), 45 (C9), and 60 (C10) per cent of ultimate load at 20°C, respectively. Rate of heating 4°C/min [29]

From the present state of knowledge, as concerns the mechanical properties of concrete and reinforcing steels at elevated temperatures, it follows, that such phenomena easily can be predicted for fire exposed concrete structures, for which the strength and deformation properties of the reinforcement are decisive. This applies to the ultimate moment capacity of simply supported beams and slabs of ordinarily reinforced and prestressed concrete. Then the load-carrying capacity can be determined for the hot state as well as for the residual state after fire exposure. Other types of failure - as shear, bond and anchorage failures - have not been the subject of any systematic studies in connection with fire and little is known about them at present.

For fire exposed, continous beams and slabs it seems justified to assume that the limit state theory can be applied in many cases [30]. It should be noted, however, that the rotations induced by thermal gradients are considerable and the rotation capacity required for a complete redistribution of moments therefore can be greater than at ambient conditions. The influence of thermal exposure on the rotation capacity of concrete structures has not yet been studied. In continuous beams, exposed to fire from below, portions with negative moments will be affected by the fire mainly in the compression zone. Here the possibility, that concrete failure occurs before the reinforcement yields, must be considered.

For non-slender, centrically loaded columns and walls, the failure occurs when the compressive strength of the concrete is exceeded. If sufficient plastic deformations can develop at fire exposure, then the ultimate state can be analyzed according to the plastic theory. At the present state of knowledge, it is difficult to say whether such an assumption always is justified or not. Studies, made by BENGTSSON [31], indicate the validity of the assumption, as concerns a theoretical determination of the residual, load-bearing capacity of concrete columns after fire, Fig. 9.

Also for more complicated applications, for instance a theoretical analysis of the structural behaviour of fire exposed concrete frames, mathematical models and connected computer programs are available [10], [32], [33]. The most comprehensive program is that presented in [33], which 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

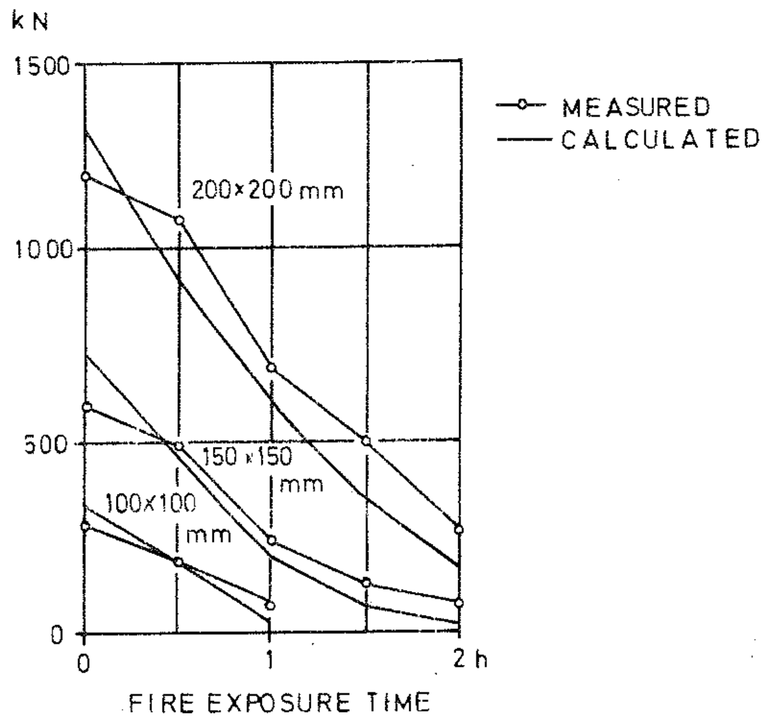


Figure 9. Measured and calculated values of the residual load-bearing capacity of non-slender, concrete columns as function of fire exposure time at standard heating conditions [31]

steel reinforcement, as well as the current states of concrete with respect to cracking or crushing and steel reinforcement with respect to yielding. Instability phenomena and second order effects are not included in the program. An example of results, calculated by using the program in [33], is given in Fig. 10.

The output of a mathematical model and a computer program according to [33] depends on the reliability of the applied data on the material properties under transient, high-temperature conditions. The creep model, used in the program at present, is correlated with creep data obtained at constant temperatures, i.e. the transient strain component for stressed concrete under heating is not included in the model. At the same time, such a model and computer program can be seen as a framework, which can be successively improved as new material data are obtained.

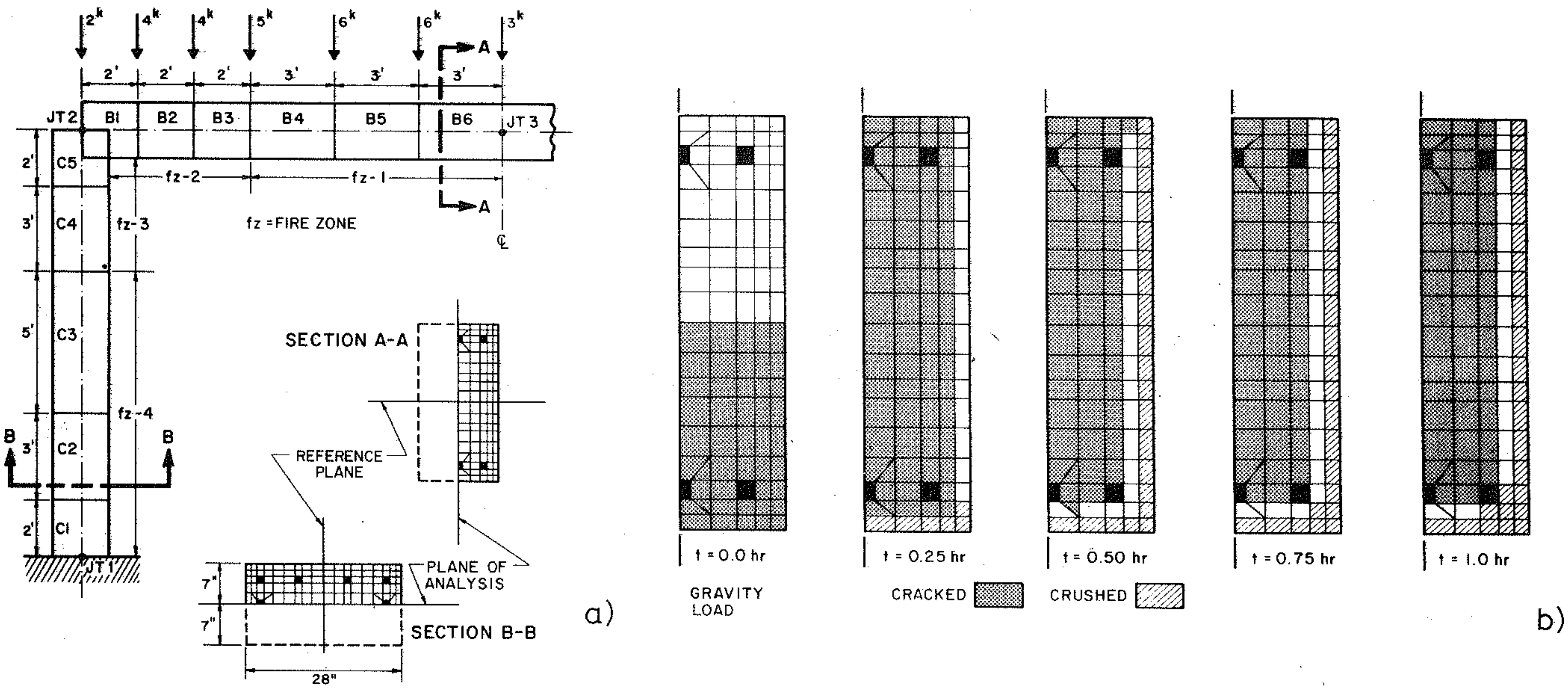


Figure 10. a) Idealization of a reinforced concrete frame, divided into substructural members, segments from substructure, concrete subslices and reinforcing bars
b) Degradation profiles for beam segment B6 at fire exposure according to standard heating conditions [33]

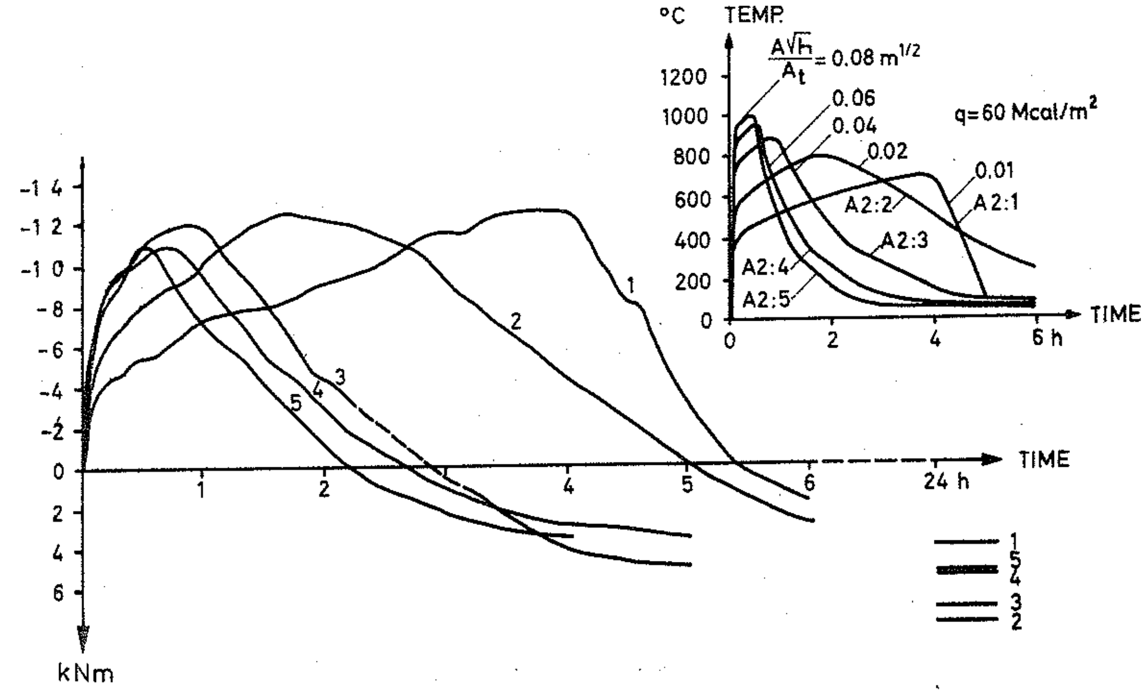


Figure 11. Variation in end bending moments with time for a one-way slab of reinforced concrete, fixed against rotation at both ends, but free to expand longitudinally, and exposed to fire on the lower surface. No exterior loading. Differentiated fire exposure, characterized by a constant fire load density $q = 60\ Mcal/m^2$ with varying opening factor $A\sqrt{h}/A_t$ within the range 0.01 – 0.08 $m^{1/2}$ [34]

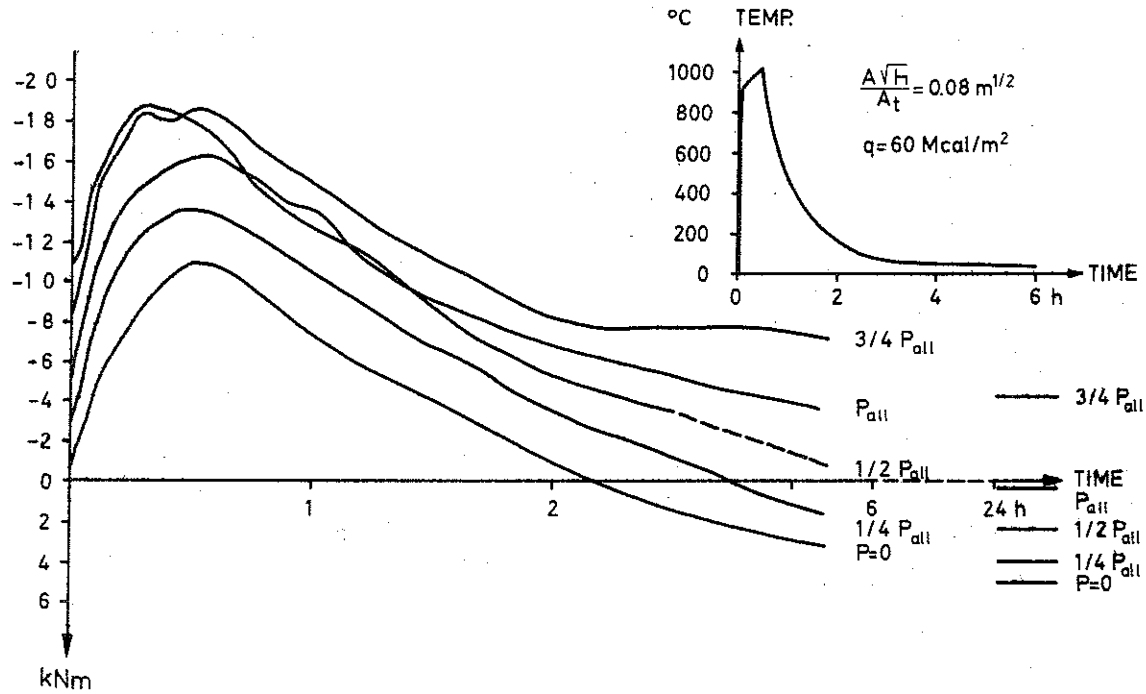


Figure 12. Variation in end bending moments with time for a one-way slab of reinforced concrete, fixed against rotation at both ends but free to expand longitudinally, and exposed to fire on the lower surface. Fire exposure according to the gastemperature-time curve shown. Exterior loading P in form of two symmetrical, transverse point loads, defined in relation to the allowable design loading P_{all} [34]

As a further illustration of the differentiated behaviour of fire exposed, hyperstatic, concrete structures, some results are reproduced in Fig. 11 and 12 from tests on one-way slabs, fixed against rotation at both ends, but free to expand longitudinally, and exposed to fire on the lower surface [34]. The figures are giving the variation of the end bending moments with time for fire exposures according to the gastemperature-time curves shown. Fig. 11 applies to slabs without any exterior loading at a differentiated fire exposure, characterized by a constant fire load density $q = 60\ \mathrm{Mcal/m^2}$ with varying opening factor $A\sqrt{h}/A_t$. Fig. 12 refers to slabs subjected to a transverse load P, defined in relation to the allowable design load P_{all}, and to a fire exposure corresponding to the fire load density $q = 60\ \mathrm{Mcal/m^2}$ and the opening factor $A\sqrt{h}/A_t = 0.08\ \mathrm{m^{1/2}}$.

In the figures also are indicated the residual end restraint moments after cooling down to ordinary room temperature. The results demonstrate that at low values of the exterior load level, positive end restraint moments of considerable magnitude can remain after a fire exposure.

An additional factor of uncertainty in an analysis of a fire exposed concrete structure is the spalling phenomenon. When the spalling occurs, the geometry of the structure is changed and the temperatures will increase more than expected from the calculations, based on the original geometry. The spalling may also directly influence the structural behaviour. Hence, a special estimate must be made, as regards the risk of spalling, which constitutes an additional problem in the application of a differentiated design. It should be noted, however, that the same problem also is inherent in the conventional schematic design procedure, related to classification systems.

By experience it is known that the disposition to spalling increases

at high moisture content,

at presence of compressive stresses from exterior loading or prestress,

at high rate of temperature increase,
at highly unsymmetrical temperature distribution,
at thin walled cross sections, and
at high percentage of reinforcement.

The risk of spalling is greater for concrete with quartz aggregate than with, for instance, limestone aggregate. An increase of the air content of the concrete gives an improved resistance against spalling.

Primarily, spalling is caused by one or several of the following mechanisms [35] - [40]:
(1) Vapour pressure due to vaporisation of moisture in the material.
(2) Thermal stresses due to restrained temperature deformations, including restraint stresses from differences in thermal elongation of concrete and reinforcement.
(3) Structural disintegration of the aggregate.

Further studies which can contribute to a better functional understanding of the spalling of concrete and its causes have high degree of priority. A central concept in such studies is that concerning the joined mechanisms of transient heat and moisture transfer.

In order to prevent the occurence of spalling, the diagram in Fig. 13 can be used as a simple guidance in the fire engineering design of concrete structures [39]. The diagram is based on extensive experimental studies covering a wide region of variations with respect to concrete quality and temperature exposure. The diagram gives a borderline, determined by the maximum stress σ_o from exterior loading and prestress and by the cross section width b or web thickness t. Above this borderline a destruction by spalling probably will occur at a fire exposure, and below, the structure will be safe with regard to spalling. The results are directly valid for concrete structure with a low percentage of reinforcement. An increase of the percentage of reinforcement results in an increased risk of spalling.

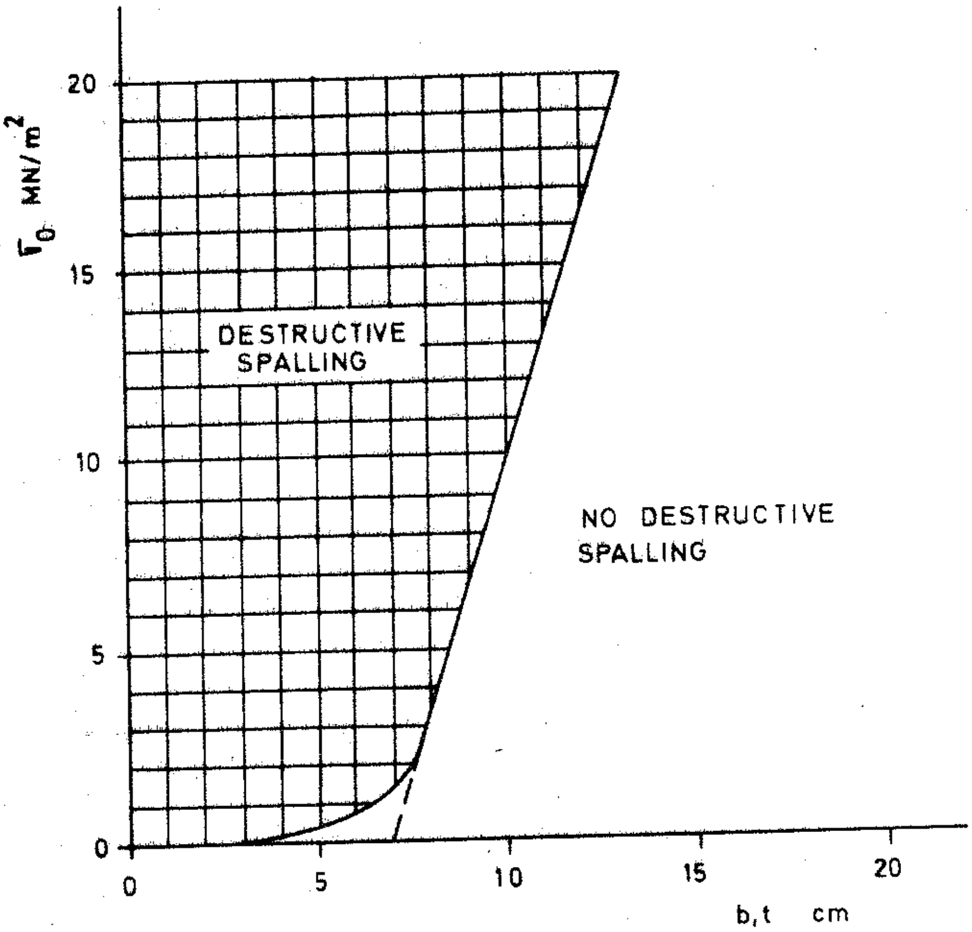


Figure 13. Borderline between destructive and no destructive spalling of fire exposed concrete structures with a low percentage of reinforcement. σ_0 is the maximum compressive stress from exterior loading and prestress, b width of cross section, and t web thickness [39]

3. Estimation of Condition of Fire Damaged, Concrete Structures [40]

In the present state of art report, the primary importance is attached to a summary review of the characteristics of a differentiated fire engineering design of ordinary reinforced and prestressed concrete structures. This way has been chosen in the light of judging the aspect of the future as the most essential one. For structures with a requirement on re-serviceability after a fire exposure, a direct differentiated design constitutes the natural solution. If realistic data concerning fire exposure and material properties are available, a differentiated fire engineering analysis also offers the best way for a determination of the residual state and strength of a fire damaged structure and for a judging of the prerequisites for a re-use of the building and of the extent of the necessary repair.

The conventional way at present, as concerns the latter case of application, implies an estimation of the condition of the fire damaged structure on the basis of data on the properties of the concrete and reinforcement, determined in tests in situ or on test specimens of the structure.

A thermal exposure on a concrete structure can give rise to the following main types of damages:
(1) A permanent deterioration of the mechanical properties of the concrete and reinforcement.
(2) Falling off of material by spalling etc.
(3) Remaining deformations and crack formation.
(4) Loss of prestress in prestressed concrete structures.
(5) Changes in appearance.

One way of an experimental reconstruction of the temperature conditions for a fire exposed concrete structure has been developed by HARMATHY [41]. He shows, that the maximum temperatures attained at various locations in a concrete structure during a fire can be determined with the use of thermogravimetric and dilatometric analyses. A thermogravimetric test then implies that small test samples of crushed concrete are heated at a constant rate with a continuous recording of the weight decrease. By comparing the curves of weight decrease for temperature exposed and unaffected concrete, it is possible to find out the maximum temperature level, which the former has been exposed to. The principal of the determination is illustrated in Fig. 14. A condition for the use of the method is that the test samples can be procured within 1 or 2 days of the fire in order to avoid the concrete receiving moisture of the air in any decisive extent. The taking of the test specimens must be done without supplying any moisture or water, too.

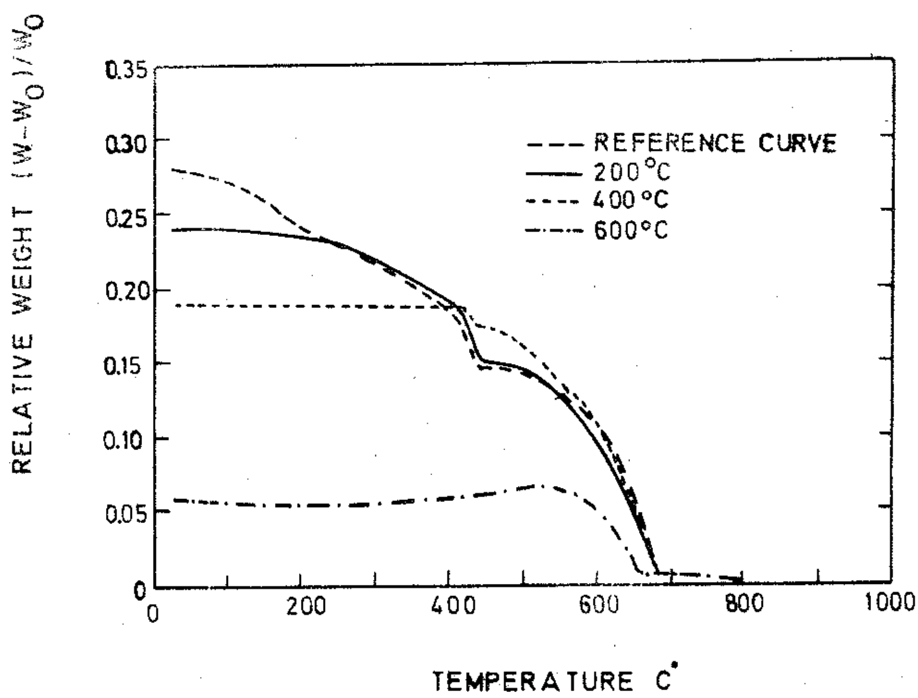


Figure 14. Thermogravimetric curves of a portland cement paste and some of its "modifications". Rate of heating 0.5°C/min [41]

If the distribution of the maximum temperature is known for a fire exposed concrete structure, the mechanical properties of the concrete and reinforcement can be estimated as well as the residual load-carrying capacity of the structure. The mechanical properties also can be determined on concrete test specimens drilled out of the fire damaged structure and analogously for the reinforcement. As concerns the test specimens of concrete, the influence of an eventual post-conditioning must be observed. If, for instance, the specimens are drilled out under adding water, a favourable effect can be introduced by a rehydration of the concrete before testing.

For concrete, there is a marked deterioration in strength at temperatures between about 450 and 600°C, as a result of the dehydration of calcium hydroxide and the change in the structure of quartz. Parts of a concrete structure, exposed to temperatures above this level, as a consequence, will be seriously damaged. Such sections easily can be cut away and by this directly identified.

For hot-rolled reinforcing steels, the residual strength is not appreciably reduced in comparison with the initial strength when heated in the temperature range from 0 to 600°C. For hardened and cold-streched prestressing steels, the residual strength begins to diminish to a noticeable degree when the temperature exceeds about 400 and 300°C, respectively. A non-destructive determination in situ of the residual strength of, for instance, a prestressing steel in a fire damaged concrete structure can be made by metallographic tests for microhardness [42]. Test results of the method are exemplified in Fig. 15, showing the relationship between

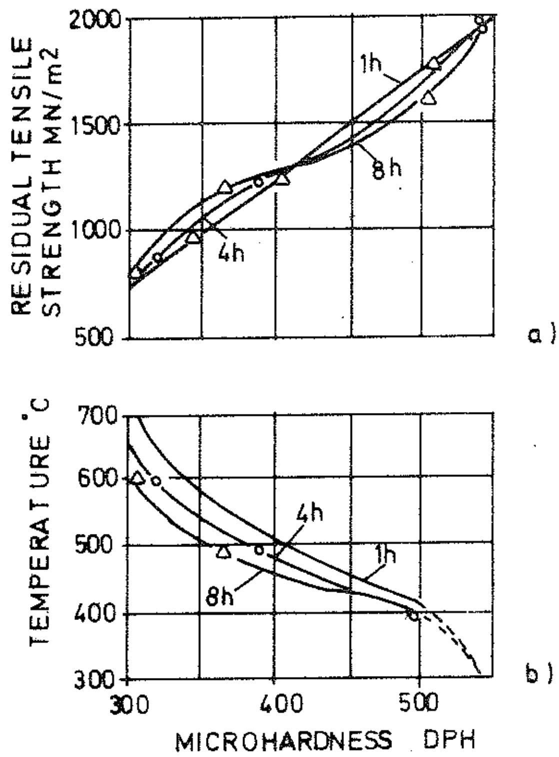


Figure 15. Data for estimating the residual tensile strength and exposure temperature of a prestressing steel from microhardness (DPH) at varying exposure time [42]

residual strength-microhardness and exposure temperature-microhardness, respectively [42].

When the mechanical properties of a fire damaged concrete structure are determined, the structure usually can be analyzed according to the design procedure, presented in chapter 2. Alternatively, experimental means for analyzing the residual state and load-bearing capacity are constituted by a test loading of the structure, giving information on the residual stiffness, and by a loading to failure of structural members or sections of them, cut out of the fire exposed structure.

Generally, it is important to ascertain whether the bond of the reinforcement is sufficient at splices and in anchorage zones for a concrete structure during and after a fire exposure. For prestressed concrete structures, even comparatively moderate temperatures can give rise to a considerable residual reduction in the prestressing forces. This reduction may be very difficult to estimate only on the basis of a known temperature history. As a rule, a thorough determination of the reduced prestressing forces requires a test on a representative section, cut out of the fire damaged structure, or an investigation in situ on prestressing bars and wires, made easily accessible within limited lengths [43].

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