Structural Fire Protection. Report at CIB W14 Meeting in Copenhagen, May 1978

Pettersson, Ove

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Report, Group Session 5.2, CIB W14 Commission Meeting in Copenhagen, May 1978

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The following report is focused on those papers, dealing with structural fire protection problems, which have been circulated within CIB W14 during the last two years, i.e. between the commission meetings 1976 and 1978. In the report, these contributions are placed within an overgrasping matrix, arranged on one hand with respect to different structural fire engineering design systems, on the other with respect to different structural materials. In order to improve the covering of the subject field, the report also includes some important contributions outside the CIB W14 circulation.

1. Matters Related to Standard Fire Testing

The internationally prevalent fire engineering design of buildings and elements of building construction is still characterized by a procedure, based on classification and a connected standardized fire test according to ISO 834 with fixed heating conditions. In the practical design, the results of such standard classification tests directly are to be compared with the corresponding requirements, specified in the building codes and regulations.

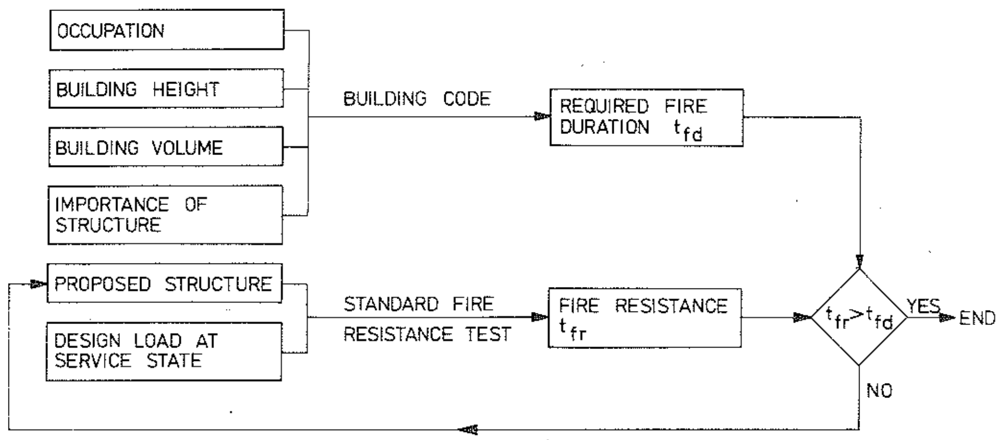


Figure 1. Conventional fire engineering design of load-bearing structures, based on classification and results of standard fire resistance tests

Fig. 1 illustrates the design procedure. For different applications, the codes and regulations are giving the required time of fire duration t_{fd}, for which the structure has to fulfil its load-bearing function. The fire duration time ordinarily depends on the occupation, the height and volume of the building, and the importance of the structure or the structural member. The design comprises a proof that the structure has a fire resistance time t_{fr}, determined in the standard fire resistance test, which exceeds the fire duration required. At the test, then the test load conventionally is put equal to the design load at service state.

Internationally, the standard fire resistance test according to ISO 834 is considered to be one of the fire test methods most thoroughly dealt with. In spite of this, the fire resistance test can be seriously criticized. In its present form, the test procedure is insufficiently specified in several respects, for instance concerning the heating and restraint characteristics, the environment of the furnace, and the thermocouples for measuring and regulating the furnace temperature. Consequently, a considerable variation can arise in fire resistance for one and the same structure or structural member, when tested in different fire engineering laboratories with varying furnace characteristics and varying practice, as concerns the support and restraint conditions of the test specimens - cf. for instance, [1]. Theoretically and practically possible difference ranges from unsatisfactorily specified furnace and thermocouple characteristics have been analyzed in detail by OTTO PAULSEN [2] on the basis of a model set up for the connected energy balance for a furnace and the thermocouples which control the temperature-time curve of the furnace at a fire test. The results of the analysis are exemplified in Fig. 2, which presents the concept "characteristic time" τ_k as an integrated measure of the thermal properties of a fire testing furnace. The characteristic time τ_k gives the point in time during an ideal furnace temperature-time process according to ISO 834, when the temperature for a defined concrete wall, fire exposed on one side, has increased by 350^{o}C in a section 15 mm in from the exposed side of the wall. The input parameters in the figure are:

$\lambda\rho c$, with λ = thermal conductivity, ρ = density and c = specific heat capacity for the interior material of the constructions enclosing the furnace,

$(p_{CO_2} + p_{H_2O})\ L_m$, with $p_{CO_2} + p_{H_2O}$ = the sum of the partial pressures for carbon dioxide and water vapour in the combustion gas and

$L_m = 3.5\ V/A$

= the mean distance for radiation from the gas mass to the surrounding furnace surfaces - V = the volume of the gas mass and A = the total limiting surface of the gas mass,

$f_v L_m$, with f_v = the volume concentration of soot in the gas mass.

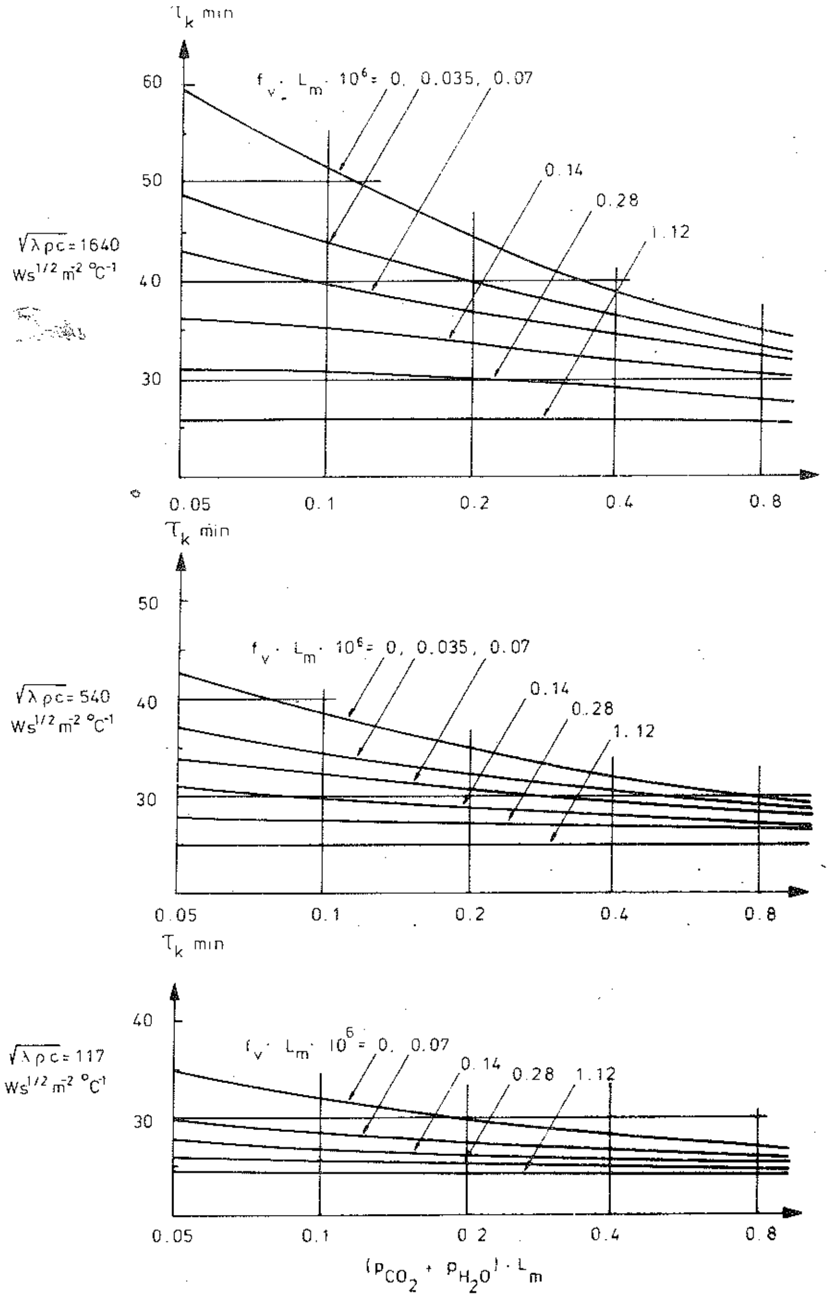


Figure 2. Characteristic time τ_k for a summarized description of the thermal properties of a fire test furnace [2]

Fig. 2 gives a maximum possible variation in the characteristic time τ_k from 24 to 58 minutes. With decreasing thermal inertia $\sqrt{\lambda\rho c}$ of the surrounding constructions, the variation range of the characteristic time τ_k decreases. The circumstances exemplified apply for an unequivocally fixed gastemperature-time curve and an unequivocally fixed test specimen with regard to internal heat transfer. Another influence on the reproducibility of the thermal exposure in standard fire resistance tests is the difference between the real gas temperature and the temperature measured for the thermocouples which control the time variation of the thermal exposure in the test. This temperature difference can, theoretically, be shown to amount to more than 100^{o}C for a fire test in a furnace with a large characteristic time τ_k - Fig. 3 - and to approximately 50^{o}C for a fire test in a furnace with a small characteristic time τ_k. The influence of varying thermal characteristics of the test furnaces and the influence of varying energy balance conditions of the thermocouples counteract each other.

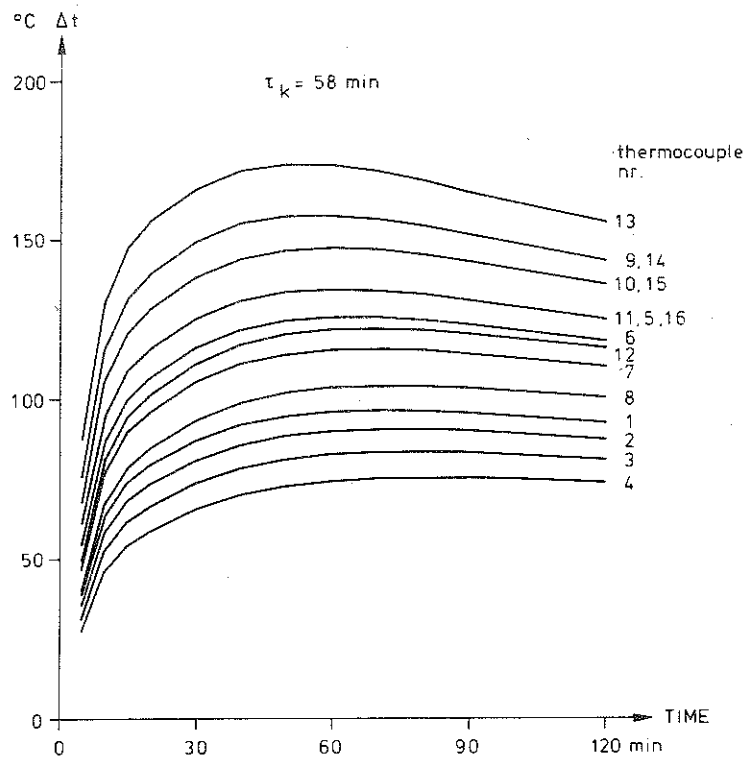


Figure 3. Calculated difference between real gas temperature and temperature measured for the thermocouples which control the time variation of the thermal exposure in a standard fire resistance test. The curves refer to 16 various thermocouples with a range of variation of 0.5 to 2.0 mm in diameter and of 0.5 to 4 ms^{-1}, as concerns the gas flow velocity around the thermocouples [2]

When testing load-bearing structures and partitions, which contain combustible material, the reproducibility of the test method is further impaired due to the fact that the thermocouples, which control the thermal exposure and which are specified to be unprotected, may be surrounded by flames from a burning test specimen.

Contributions, which may result in an improved reproducibility of the standard fire resistance test according to ISO 834, have been given the highest priority for the future activities of ISO/TC92 WG 11. A compulsory specification of the characteristic time τ_k of the furnace used in test reports would considerably improve the possibilities of making a comparative evaluation of test results from various fire laboratories. Radically improved reproducibility could be achieved if the present gas temperature control via thermocouples was replaced by a control of the radiation temperature via radiation meters or other equipment which register the incident heat flux on the exposed surface of the test specimen. Supplementing existing fire testing furnaces with an internal cladding of high insulating material with a low density - small $\lambda\rho c$ - comprises a further measure for improving the reproducibility of the test method. As a result of a CIB W14 proposal, a recommendation probably will be included in the commentaries of ISO 834, that primarily new furnaces and existing furnaces when repaired should be relined with material having a thermal inertia $\sqrt{\lambda\rho c}$ not greater than 600 W $s^{1/2}$ m^{-2} K^{-1} at 773 K.

Beside the task to modify ISO 834 for improving the reproducibility of the test method, activities are at present in progress within ISO/TC92 WG 11 primarily in connection with two other main tasks. One embraces the development of a test method for suspended ceilings which are forming part of an unventilated floor or roof assembly and which are fire exposed from below [3] - Fig. 4. In the proposal prepared, the test method has been designed so that the ceiling can be classified from a fire resistance point of view, using the test results as basis, with as general an application as possible with regard to the detailed design of the floor and roof assembly for the rest. Within the working group further a procedure has been developed for a theoretical extrapolation of the test results. The other main task of the working group embraces the development of a test method for determining the fire resistance of external structural elements, e.g. a column located in front of an external wall.

The activities within ISO/TC92 WG 3 have so far led to two internationally standardized test methods - ISO 3008 and 3009 - for assessing the fire resistance of doors, shutter assemblies and glazings [4], [5]. The work now continues with the development of methods for fire testing of smokeproof doors.

There is a considerable need for developing internationally co-ordinated fire test methods within the services sector. Development work of this

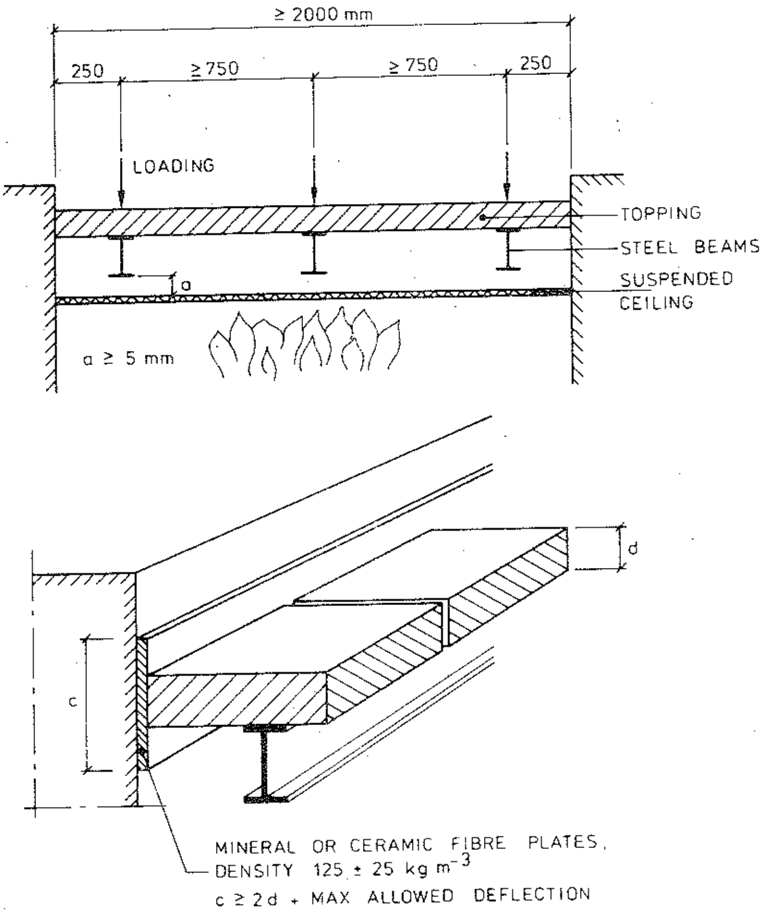


Figure 4. Test specimen according to a proposed ISO standard for a determination of the contribution of a suspended ceiling to the fire resistance of an unventilated floor or roof assembly with load-bearing steel beams [3]

type is now going on in ISO/TC92 WG 14 for the sub-application ventilation ducts and fire dampers.

2. Standard Fire Classification of Load-Bearing Structures, Carried out Analytically

Some countries now are permitting a classification of load-bearing structures with respect to fire exposure to be carried out analytically. This leads to a design procedure according to Fig. 5. The theoretical determination of the fire resistance time of the structure t_{fr} then is to be based on the gastemperature-time curve, specified for the standard fire resistance test.

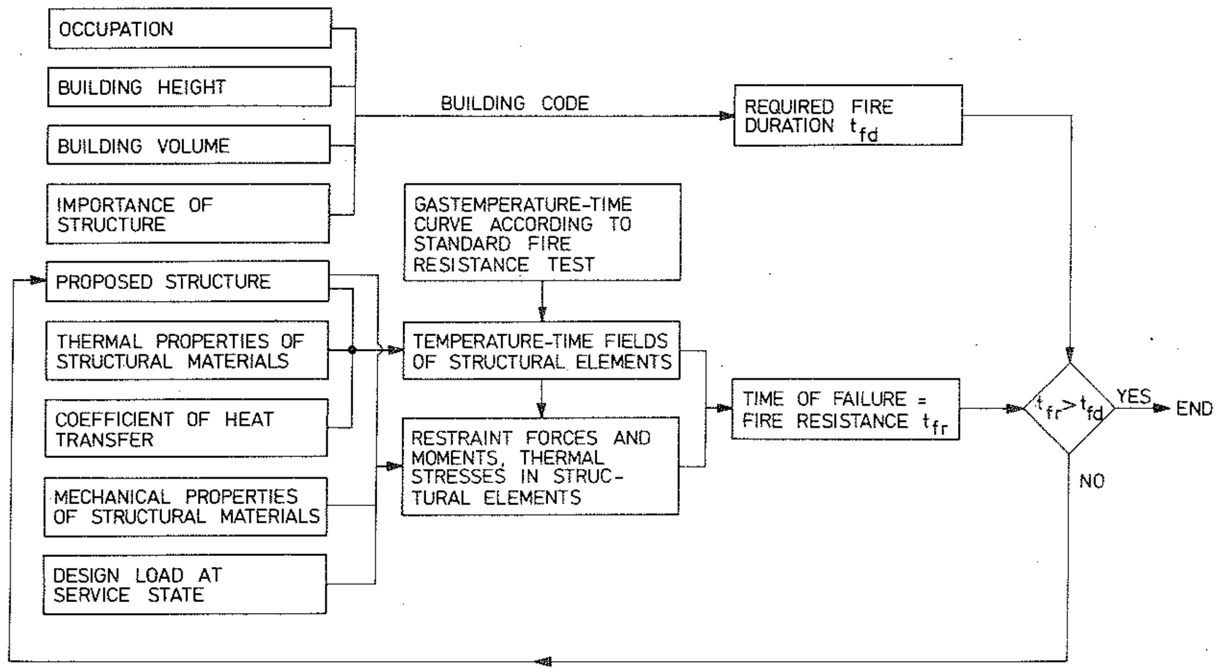


Figure 5. Theoretical procedure of a fire engineering design of load-bearing structures, based on classification and thermal exposure according to standard fire resistance tests

With this gastemperature-time curve as basic information, the temperature-time fields of the fire exposed structure or structural members can be calculated, using

(1) the structural characteristics of the proposed structure,
(2) the thermal properties of the structural materials, and
(3) the coefficients of heat transfer for the various surfaces of the structure

as further input data. Introducing

(4) the mechanical properties of the structural materials, and
(5) the load characteristics

then the time variation of the restraint forces and moments, thermal stresses and load-carrying capacity can be determined. The time, at which the load-carrying capacity has decreased to the level of the design load at service state, defines the time of failure or the fire resistance time t_{fr} and the design criterion to be satisfied is, that $t_{fr} > t_{fd}$.

In France, until the end of 1975, the only recognized legal justification with regard to a fire design of building structures was the standard fire resistance test. From 1975 the conceptual approach was revised, as concerns fire exposed concrete structures, by setting up a "Méthode de Prévision par le Calcul du Comportement au Feu des Structures en Béton [6], [7] - abbreviated "DTU Feu". Accordingly, now the following methods of justification are permitted:

(1) An application of simple rules of design of the type specified in the FIP-CEB Recommendations [8],
(2) an ultimate load analysis from observed test results on temperature,
(3) a complete analysis of the temperature state and the connected minimum load-bearing capacity, and
(4) a design, entirely based on test results.

With regard to the thermal exposure, all four alternatives are to be based on the gas temperature-time curve according to ISO 834. A basic, simplified guidance is given in the document "DTU Feu" [6] for the thermal and mechanical properties of concrete and reinforcing steels at elevated temperatures. A computer programme with instructions for a calculation of the temperature-time fields is included in the document. A generalized version of this computer programme has been developed by COIN [9], which enables that
the temperature dependence of the thermal material properties,
the presence of different materials in the cross section,
an initial content of free water,
the influence of chemical material transformations, and
the presence of voids
can be taken into account. The computer program is based on a finite difference method.

In order to improve the technical basis for the analytical methods, permitted in a fire design of concrete structures according to "DTU Feu", a comprehensive test programme now is running in France under the supervision of a group of engineers with Mr. COIN as chairman. The research programme is described summarily in [7] with some examples of test results, already received in a first test series, comprising 24 beams and devoted to a thorough study of the temperature distribution. The experimental results of this introductory study confirm the validity of the computer programme, presented in [9].

For a theoretical determination of the standard fire resistance of loadbearing steel structures a corresponding document "Prévision par le Calcul du Comportement au Feu des Structures en Acier" [10] has been drawn up by a group of engineers from CTICM. The document gives a very complete guidance for a practical design of unprotected and protected steel structures and enables, for instance, that the influence of a restraint with regard to the thermal axial elongation can be considered. The document also comprises a set of diagrams - based on Swedish theoretical studies; cf., for instance, [11] - for a transfer of the calculated standard fire resistance to real fire conditions.

An international document, aimed as European recommendations for the design of fire exposed steel structures, having about the same merits and quality as the French document, has been prepared by the ECCS Commission 3 [12].

Another international example of an approved method for a theoretical determination of the standard fire resistance is the US design procedure, developed by GUSTAFERRO and MARTIN for PCI with application to fire exposed concrete structures [13]. The design procedure starts from tables and diagrams, based on test results and giving the temperature in different points of structural members for varying standard fire test time. The transfer of this temperature information to a loadbearing capacity or a value of the fire resistance is then carried out analytically. The design enables that varying type and degree of structural restraint can be taken into account.

The recently published FIP/CEB Report on methods of assessment of the fire resistance of concrete structural members [14] gives in its head sections basic information on fire severity, material properties and structural behaviour as well as detailed advice to the practising engineer on how to design structural concrete elements to withstand the thermal exposure of the standard fire resistance test for required periods of fire duration. The recommendations include a comprehensive set of tables, giving safe values - based on the results of research and testing - for the fire resistance of different types of structural members with varying dimensions made of dense aggregate concrete or lightweight aggregate concrete. As appendices, different rational methods for a theoretical design of fire exposed concrete structures are reviewed and discussed in order to encourage further development.

3. Analytical Structural Fire Engineering Design Methods, Based on Real Fire Exposure Characteristics

In the last ten years, several functionally based, differentiated design methods have been published, as concerns fire exposed loadbearing structures or structural members. Mainly, these methods can be referred to one of two different groups with respect to the use of the basic data of the process of fire development. The methods of the first group - Fig. 6 - are characterized by a design procedure, directly based on differentiated gastemperature-time curves of the complete process of a real fire development, specified in detail with regard to the influence of the fire load and the geometrical, ventilation and thermal properties of the fire compartment [11], [14], [15]. Characteristic for the methods of the second group is a design procedure with the varying properties of a real fire development taken into account over an equivalent time of fire duration, connected to the heating according to the standard temperature-time curve.

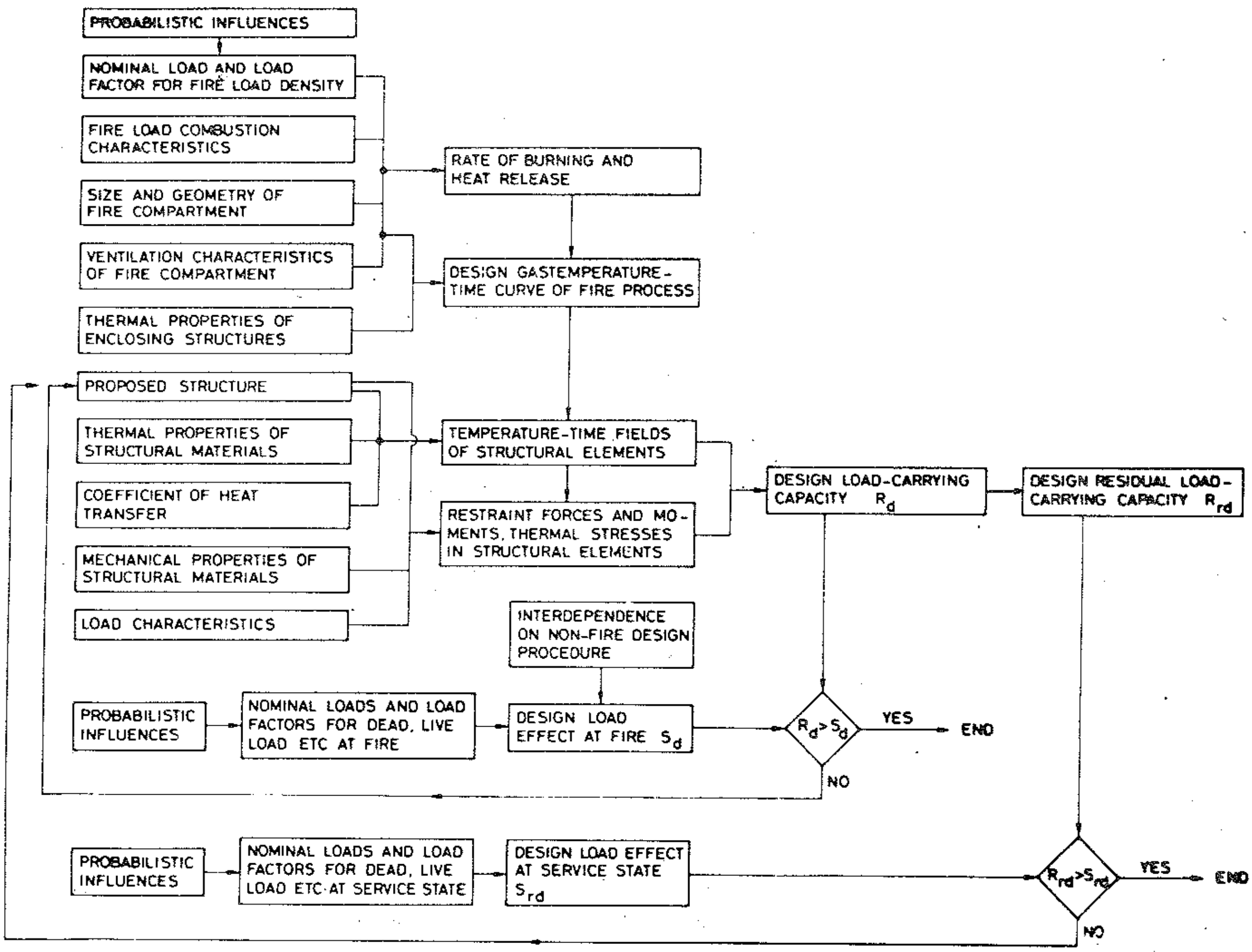


Figure 6. Procedure of a differentiated fire engineering design of load-bearing interior structures. The design scheme comprises an eventual, additional requirement on re-serviceability after fire [11]

As a provisional solution, the Swedish Standard Specifications permit a structural fire engineering design on the basis of gastemperature-time curves according to Fig. 7 [11], valid for a compartment with specified thermal properties of the surroundings - fire compartment type A. Entrance parameters of the diagrams are the fire load density and the opening factor of the compartment. A transfer between fire compartments of different thermal properties of the surrounding structures can be done according to simple rules, based on fictitious values of the fire load density and the opening factor. The gastemperature-time curves have

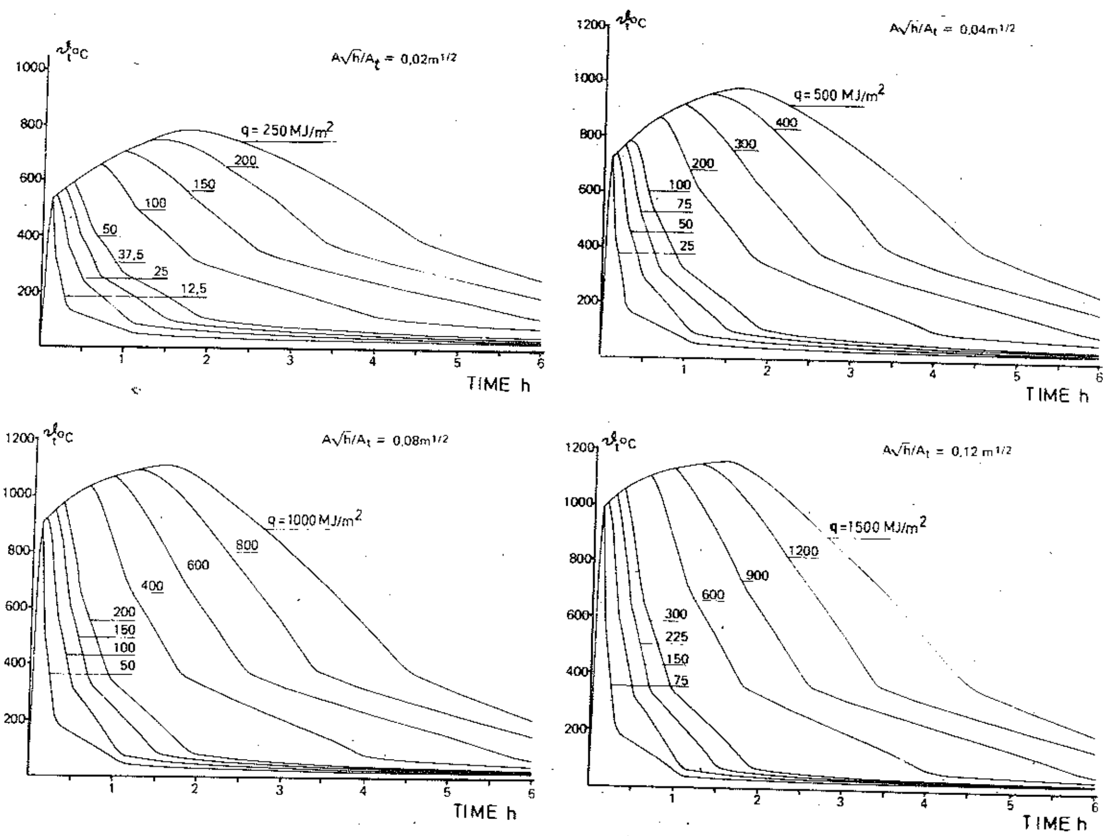


Figure 7. Gas temperature-time curves for a complete, fully-developed, compartment fire with varying values for the fire load density q and the opening factor $A\sqrt{h}/A_t$. A is the total opening area of the fire compartment, h is a weighted mean value of the height of the openings based on their size and A_t is the total internal surrounding area of the fire compartment, including openings. Fire compartment, type A [11]

generally been determined on the assumption of ventilation controlled fires, which implies that they may give a thermal exposure not inconsiderably on the safe side for compartment fires which are fuel-controlled. Similar, somewhat more simplified gas temperature-time curves also have been derived by LIE, cf., for instance, [16]. The gastemperature-time curves for ventilation controlled fires constitute the basis for the design tables, presented in the French document "Prêvision par le Calcul du Comportement au Feu des Structures en Acier" [10] for a transfer from standard heating conditions to real fire exposure characteristics.

In a paper by BRESLER [17], [18] it is proposed that four categories of compartment fires according to Fig. 8 would provide the necessary criteria for a broad range of building types and uses.

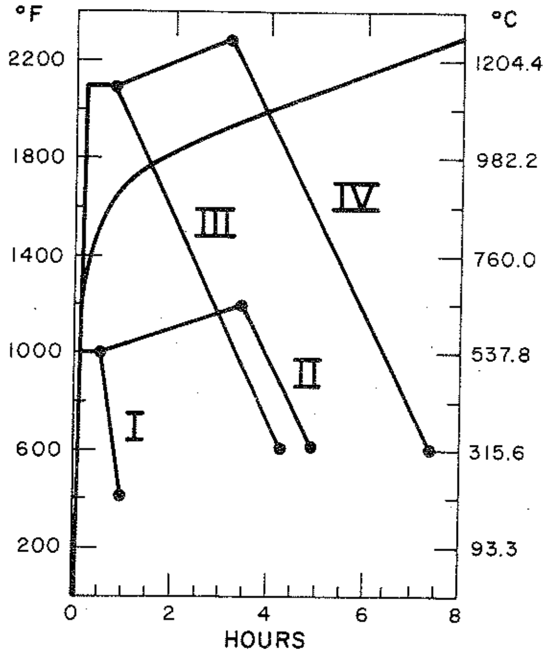


Figure 8. Four categories of design fires, proposed in [17], [18]

In a long-term perspective, the differentiated design basis with respect to a real fire exposure should be developed in the direction of a full consideration to whether a compartment fire is ventilation controlled or fuel bed controlled. This has strongly been stressed in a series of contributions by HARMATHY, cf., for instance, [19], [20].

In [20], [21] HARMATHY analyzes and discusses the real fire behaviour of partitions on a philosophy, based on
(1) real characteristics of fully-developed compartment fires, specified as depending on the three "fire severity" parameters duration, average temperature of the compartment gases and the effective heat flux penetrating the compartment boundaries, and
(2) a partition does often become exposed to fire on both sides, although not necessarily simultaneously.

A numerical technique is presented for a calculation of the temperature history of a partition. The technique is a finite difference method with the cross section divided into elements arranged on a diagonal mesh. Illustrative examples are given, showing the temperature histories of reinforced concrete slabs, exposed on both sides - with a time lapse between the fire in compartment I and the fire in compartment II - to a moderately severe or a very severe fire. Fig. 9 exemplifies the influence of the alternative of a very severe fire.

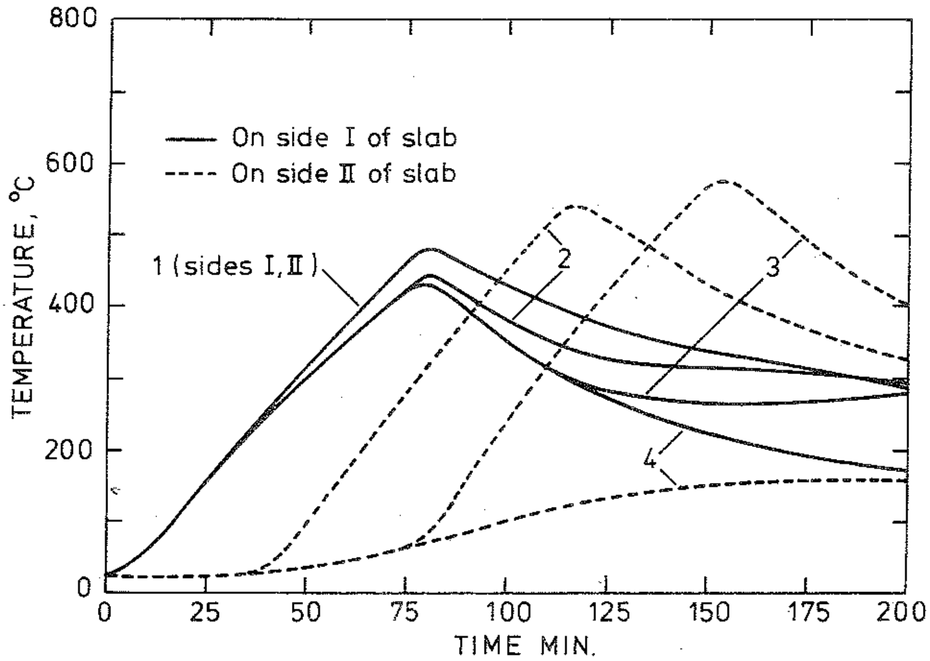
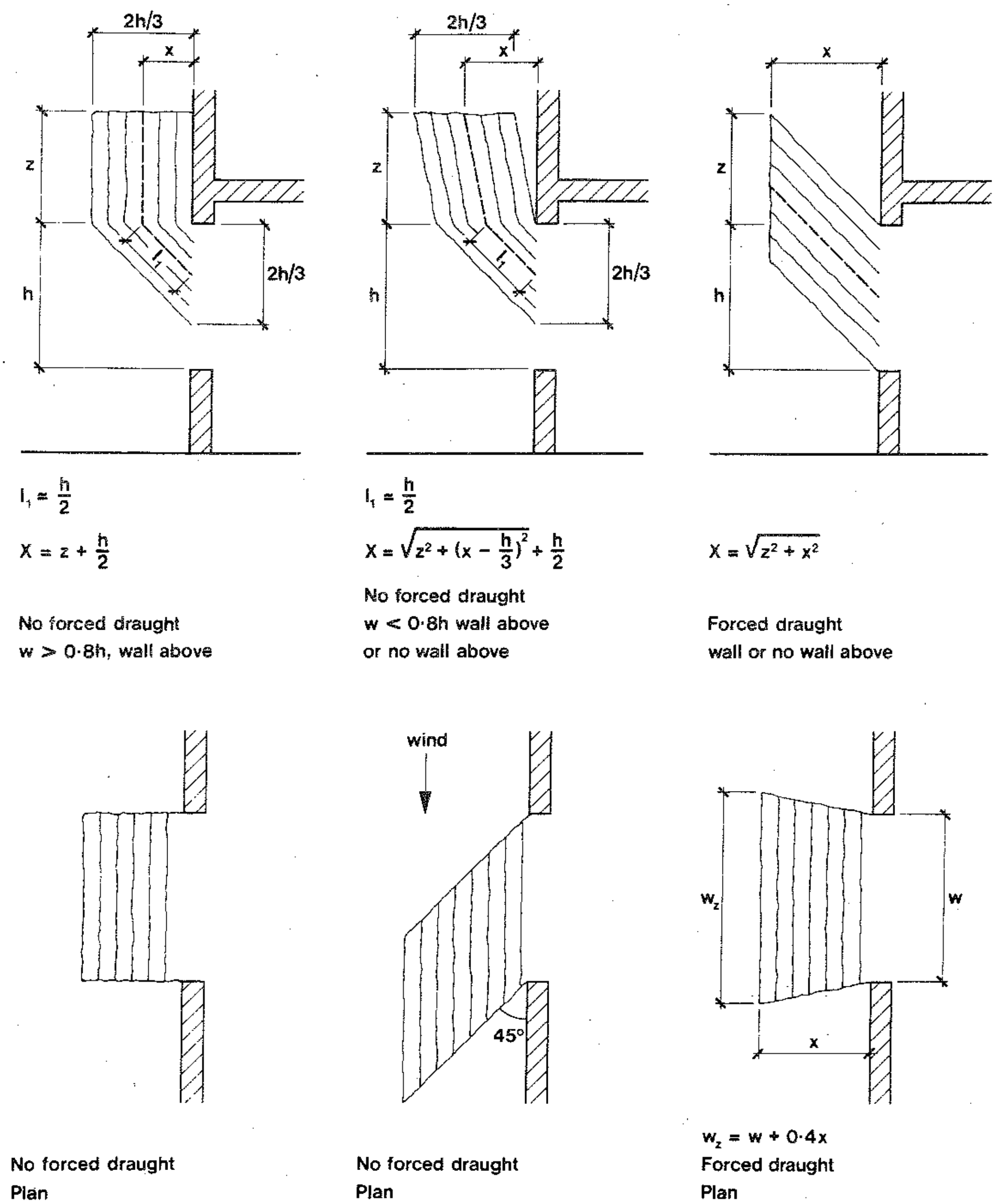


Figure 9. Temperature of the steel reinforcing bars of a concrete slab, exposed to a very severe fire. Thickness of slab 152.4 mm. Reinforcing bars located at 30.5 mm from surfaces. Time lapse from the beginning of fire in compartment I to the beginning of fire in compartment II t_0 = 0 for curve 1, 37.5 min for curves 2, 75 min for curves 3 and ∞ min for curves 4 [19], [20]

A fire exposure as described by the gas temperature-time curves according to Fig. 7 refers to interior load-bearing structural members. As concerns a differentiated design basis with regard to the thermal exposure of exterior load-bearing columns and beams, two important contributions recently have been given by LAW [22] and BECHTOLD [23].

In [22], the "State of the Art" is reviewed concerning relevant research programs carried out over the past twenty years. The tests reported are found to give a consistent pattern on the length and position of flames emerging from the windows of a fire compartment. Based on the research reviewed, a general theory is developed for the effects of fire behaviour - temperature and duration - and emerging flames on the heat transfer to exterior steel. The heat transfer depends on the flame trajectory and temperature, the fire temperature, the position of the exterior steel element and the cooling of the element to the surroundings. Relationships are given for a calculation of the flame and fire behaviour as depending on the amount and type of the fire load, the dimensions of the fire compartment and the dimensions of the windows. The effects of a



<u>Figure 10</u>. Simplified trajectories of emerging flames according to [22]

forced draught, such as a through wind, then can be taken into account. The presented heat transfer model employs simplified flame shapes according to Fig. 10. The calculation model developed assumes compartments with a size normally found in buildings and with interior wall and roof-surfaces having no linings of combustible materials.

In [23], a comprehensive test series is reported and analyzed for a full-scale study of the thermal exposure on exterior columns at a fire in a compartment of a multi-storey building. In most of the tests, the fire load was of wood crib type but also some tests were performed with a fire load of furniture, representative to dwellings or offices. The measurements made at the tests comprised the temperature fields of the flames and combustion gases, the temperature in characteristic structural members, the velocity and composition of the combustion gases, and some characteristic pressure differences. On the basis of the test results, a theoretical model is developed for a calculation of the fire exposure on exterior columns at varying conditions and useful design diagrams are presented - exemplified in Fig. 11.

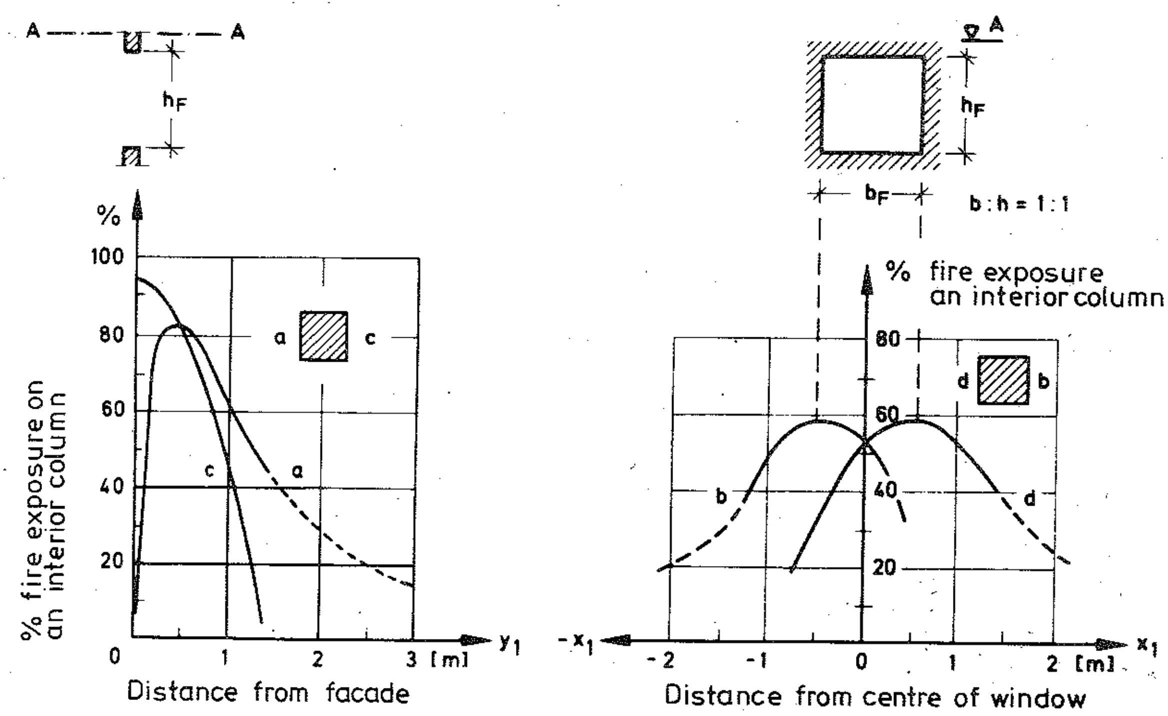


Figure 11. Thermal exposure on different surfaces of an exterior column, level A-A, in percent of the connected thermal exposure on an interior column. Window openings with a width-height ratio 1:1 [23]

In [24], HARMATHY illustrates the role of circumspect design in fire safety with regard to the subject areas: the growth of fire, the smoke problem and the fully developed fire.

As concerns fire safety related to the fully developed fire, the following items, among others, are stressed:
(1) All realistic possibilities of simultaneous and delayed fire exposure of "key structural elements" on two sides should be examined,
(2) a door, i.e. a self-closing sliding door, that remains closed during the fire is an effective barrier against the spread of fire as well as the spread of smoke,
(3) horizontal projections, i.e. flame deflectors wider than 3 to 4 ft are effective in keeping the flames away from the face of the building and in reducing the radiation to the story above to an acceptable level - Fig. 12,
(4) a high degree of fire safety can be arrived at by a technique referred to as "fire drainage". This is achieved by
(a) drawing air into the fire cell in quantities that ensure fuel-surface-controlled conditions,
(b) keeping the pressure in the fire cell below the pressure levels prevailing in the neighbouring spaces,
(c) removing the smoke and flames from the fire cell in a safe and organized manner.

A design basis for such a "fire drainage" system is given in [19].

In those cases, at which the present state of knowledge does not enable a complete analytical, differentiated structural fire engineering design to be carried out, the concept equivalent time of fire duration can be a useful implement. Generally, the concept has been introduced as a means for a direct translation for a real fire exposure to a corresponding heating according to the temperature-time curve of the standard fire resistance test, and vice versa. Depending on the type of design problem to be dealt with and the level of accuracy intended, the character of the concept will vary.

If the available design basis permits a theoretical determination to be performed, as concerns the transient temperature fields but not the design load-carrying capacity of a fire exposed structure, it can be motivated to use a differentiated form of the equivalent time of fire

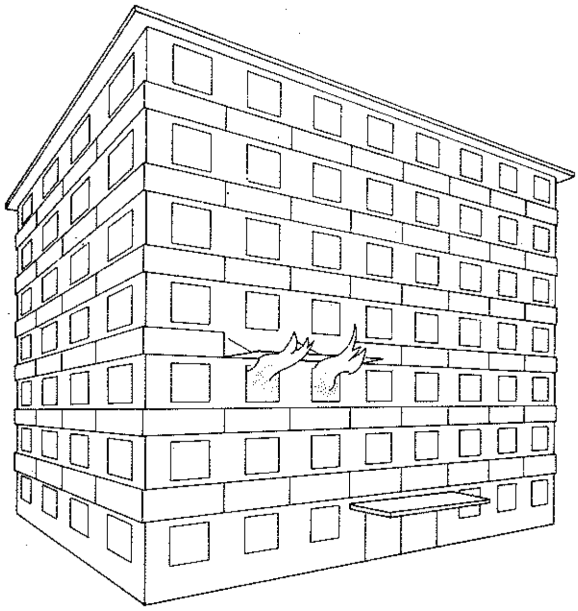


Figure 12. Flame deflectors in operation [19], [24]

duration concept - Fig. 13. Determined in this way, the equivalent time of fire duration depends on the parameters, influencing the process of fully developed fires, as well as on a number of structural parameters - for a nonprotected steel structure: exposed surface, steel volume, resultant emissivity.

A more rough form of the concept equivalent time of fire duration is necessary to use, when the structural fire engineering design is to be based on real fire exposure characteristics without the prerequisites existing for an analytical determination of the transient temperature fields and the connected load-carrying capacity of the fire exposed structure. Under such circumstances, the equivalent time of fire duration t_e can be differentiated only with respect to the detail properties of the fire process but not with respect to the characteristics of the structural design - Fig. 14. This way of applying the equivalent time of fire duration t_e has been introduced by LAW (1971), and THOMAS-HESELDEN (1972) for fire exposed, insulated steel structures. A similar approach is presented by PETTERSSON (1974), leading to the following generalized, approximate formula

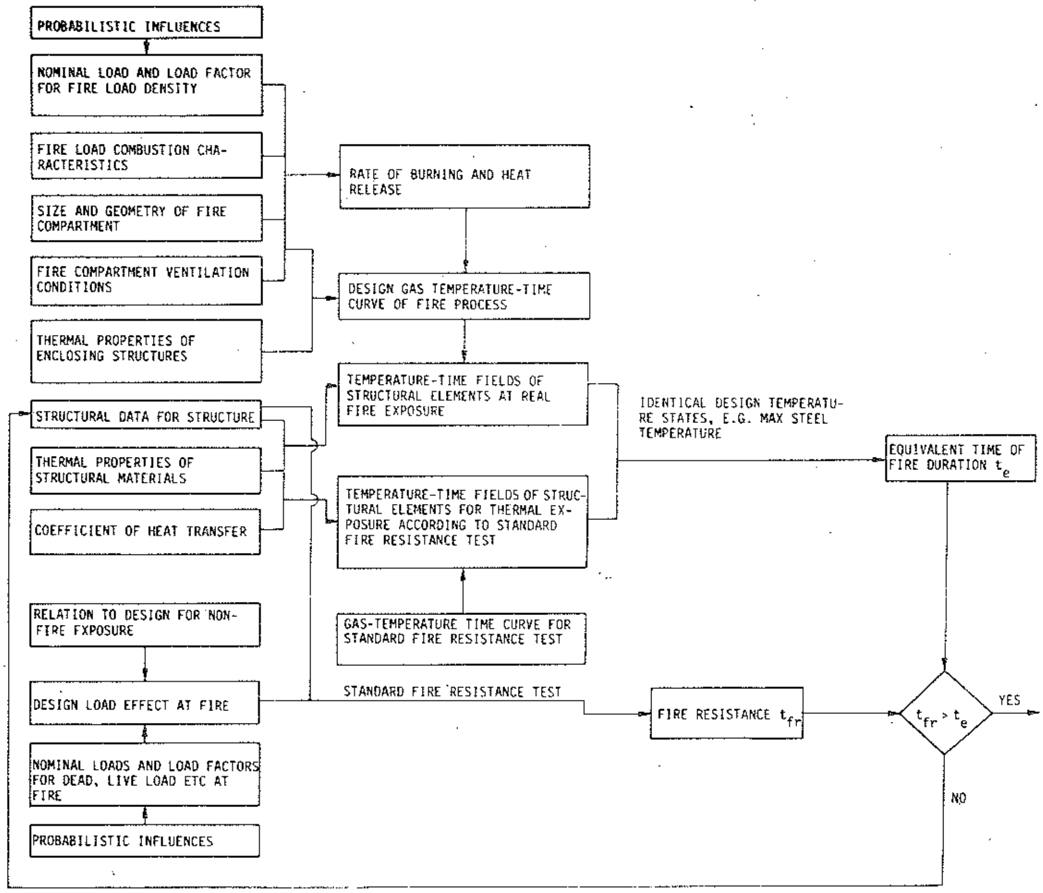


Figure 13. Procedure for a differentiated design of a structure exposed to fire, based on a theoretical determination of the equivalent time of fire duration t_e and an experimental determination in accordance with ISO 834 of the fire resistance t_{fr} [1]

$$t_e = 0.067 \frac{q_f}{\left(\frac{A\sqrt{h}}{A_t}\right)_f^{1/2}} \qquad \text{(min)}$$

in which q_f is the fictitious value of the fire load density ($MJ \cdot m^{-2}$) and $(A\sqrt{h}/A_t)_f$ the fictitious value of the opening factor of the fire compartment ($m^{1/2}$). Written under this form, the formula enables that the influence of varying thermal properties of the constructions surrounding the fire compartment can be taken into account at the determination of the equivalent time of fire duration.

The concept equivalent time of fire duration is dealt with by SCHNEIDER-HAKSEVER in [25] with special application to reinforced concrete beams.

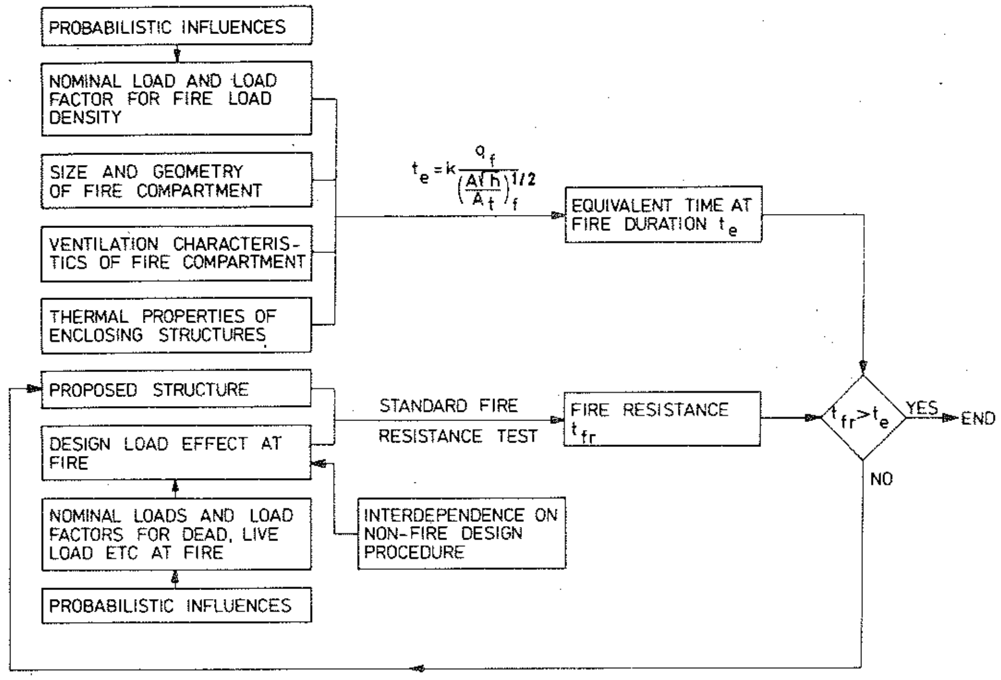


Figure 14. Procedure for a structural fire engineering design, based on a simplified approximate expression for the equivalent time of fire duration t_e and on an experimentally determined fire resistance for the structure t_{fr} [1]

The paper presents an analytical study based on the theory of heat transfer and structural fire behaviour and on the results of full-scale tests, performed at the research station at Maizières-lès-Metz within the activities of ECCS Commission 3. Comparisons are made and discussed of the equivalent time of fire duration, determined on the basis of different design criteria,

(1) temperature of the reinforcement,
(2) deflection in the centre of the beam span,
(3) rate of deflection in the centre of the beam span,
(4) horizontal movement of a support,
(5) inclination of the beam at a support.

The study then indicates that the temperature of the reinforcement ordinarily represents the decisive criterion. The investigation is summarized by the diagram, referred in Fig. 15, in which the shadow area represents the values of t_e found in the study. On the basis of these results, the authors are stating that earlier published formula for t_e are giving values on the unsafe side. As the formula according to PETTERSSON concerns - quoted above - this statement, however, is connected to those values of the formula valid for a fire compartment, type A. Applied to a fire compartment, type C, which is representative to the type of fire compartment used in the full-scale tests at Maizières-lès-Metz, the formula according to PETTERSSON gives values in close agreement with the values calculated in the investigation of SCHNEIDER-HAKSEVER.

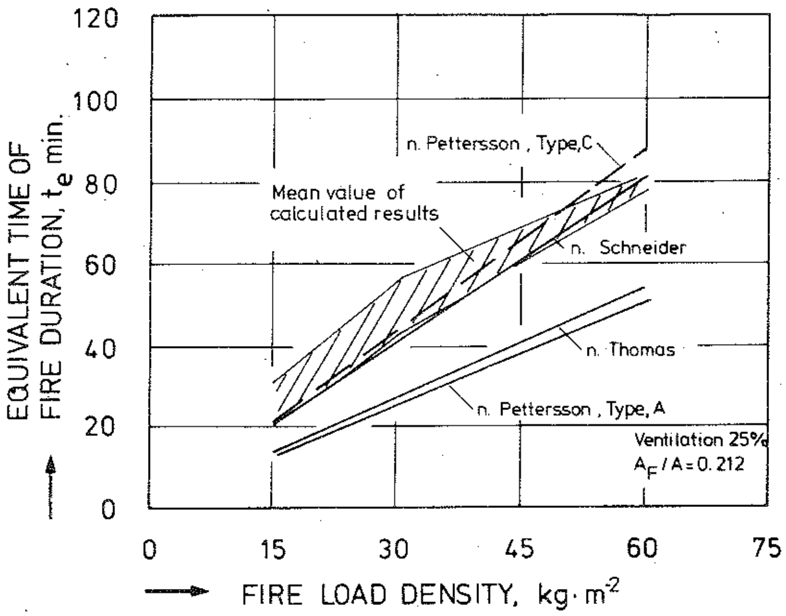


Figure 15. Equivalent time of fire duration, found by SCHNEIDER-HAKSEVER [25] for reinforced concrete beams - shadow area. For comparisons, the diagram also includes the values according to formulas by THOMAS-HESELDEN (1972) and PETTERSSON (1974) - the curve according to PETTERSSON, type C, however is not included in the report by SCHNEIDER-HAKSEVER

In reporting essential contributions, recently published or in progress, concerning different structural fire engineering design methods, it finally should be mentioned, that LAW-PETTERSSON-THOMAS at present are prepairing a CIB/W14 document, intended to get the status of internationally agreed recommendations as regards the basis of design for the fire protection of building structures [26]. Hopefully, such a document - in limited parts or as a whole - could stimulate and give some guidance for future drafting of national codes and regulations and also serve as a fundamental basis for future international recommendations, devoted to the fire protection of steel structures, reinforced concrete structures, timber structures, masonry structures, etc. In a long-term perspective, the document could contribute in a development towards internationally harmonized codes and regulations within the field of structural fire engineering design.

4. Temperature-Time Fields of Fire Exposed Structures

In [27], LIE presents a calculation procedure, based on a finite difference method, for a determination of the time variation of the temperature distribution in a rectangular shaped insulation of a steel column. The steel temperature is assumed to be uniform over the entire steel volume. The cross section is divided into elements which are square inside the insulation and triangular at its boundaries. For the steel elements of the inner surface of the insulation is assumed that a fraction α of each elementary mass is in direct contact with the adjacent elementary insulation surface - receiving heat from the insulation by conduction - while a fraction $(1-\alpha)$ of its mass is at some distance from the insulation surface - receiving heat only by radiation.

A very accurate theory, together with computer routines, is presented by WICKSTRÖM [28] for a determination of the temperature distribution within fire exposed structures or structural assemblies, designed with

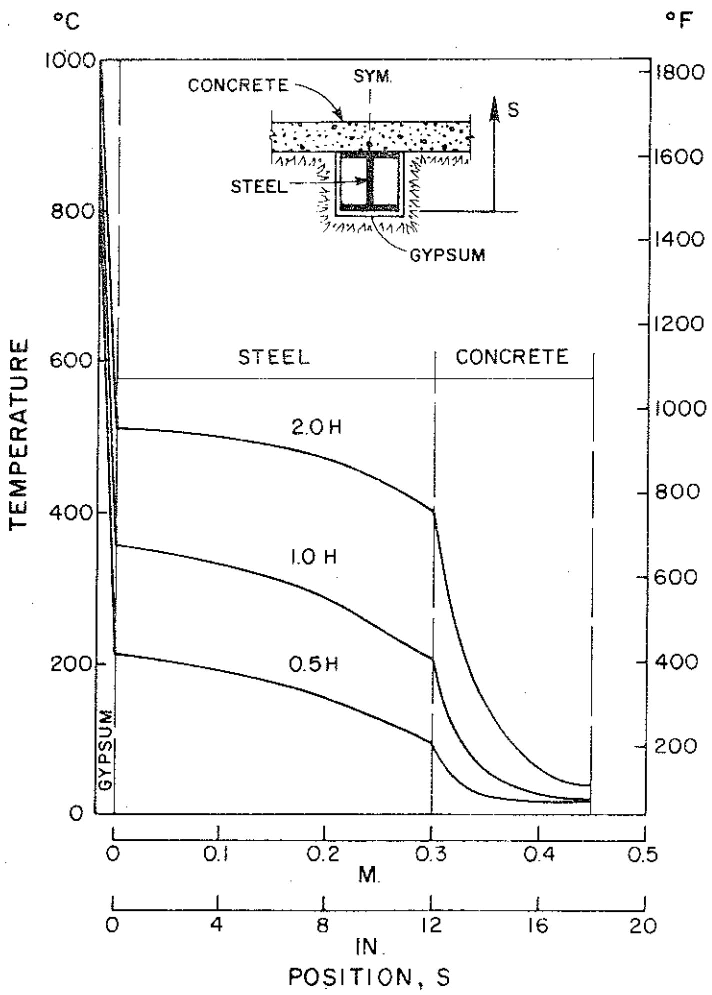


Figure 16. Calculated temperature distribution along line of symmetry of a steel beam, insulated by a 16 mm gypsum board and carrying a 150 mm concrete slab on top flange, at selected times of a thermal exposure according to ISO 834 [28]

voids. The algorithm described can easily be coupled to most finite element programs. By using geometry data already specified for the finite element mesh of a solid, view factors can be computed automatically for most void configurations. An illustration of the capability of the theory is given in Fig. 16, which shows calculated temperature distribution along the line of symmetry of gypsum insulated steel beam with a concrete slab at the top flange at selected times of a standard fire exposure in accordance to ISO 834.

5. Fire Exposed Steel Structures

In a series of publications from CTICM, ARNAULT, EHM and KRUPPA [29], [30], [31], [32] are reporting extensive full-scale fire tests on isostatic steel beams, hyperstatic steel beams and frames, exterior steel columns and mixed beams, built up by steel beams and a reinforced concrete slab in joint behaviour. The tests have been performed under the supervision of ECCS Commission 3. The test results constitute together a very large fund of important information, which certainly is useful to analyze in a much larger extent than has been the case up to now. As a sole example of the test results, Fig. 17 shows measured temperature distribution along the span and over the height of the cross section for two tests of steel beams, continuous in two spans.

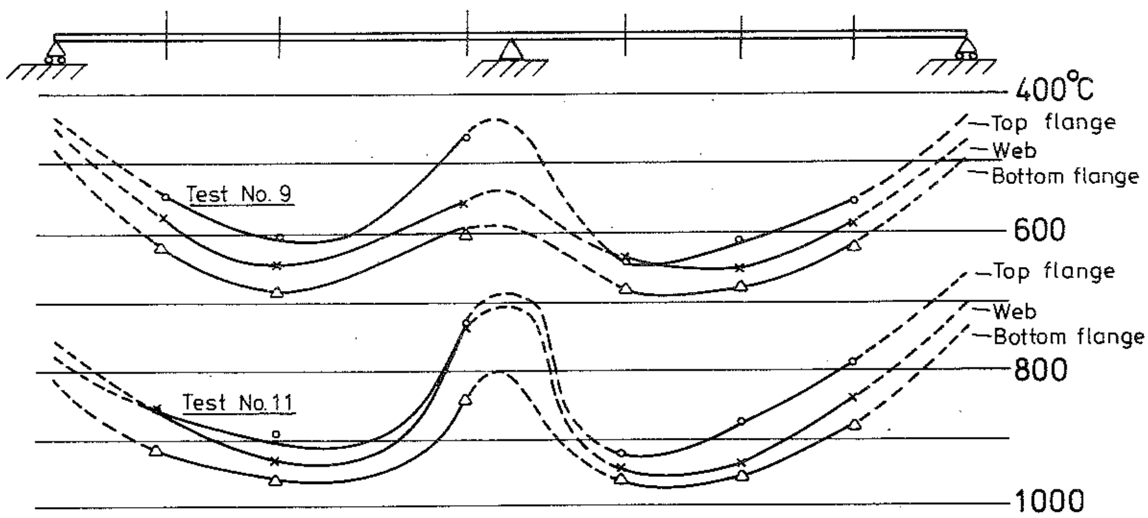


Figure 17. Measured temperature distribution along span and over cross section for two tests of fire exposed, continuous steel beams. Test No. 9: steel beam, insulated by 15 mm vermiculate plaster, test No. 11: non-insulated steel beam [29]

As a complement to the test results exemplified in Fig. 17, Fig. 18 from a recent paper by KRUPPA [33] illustrates the calculated variation of the plastic bending moment of an I cross section as a function of the maximum temperature for various temperature distributions over the cross section.

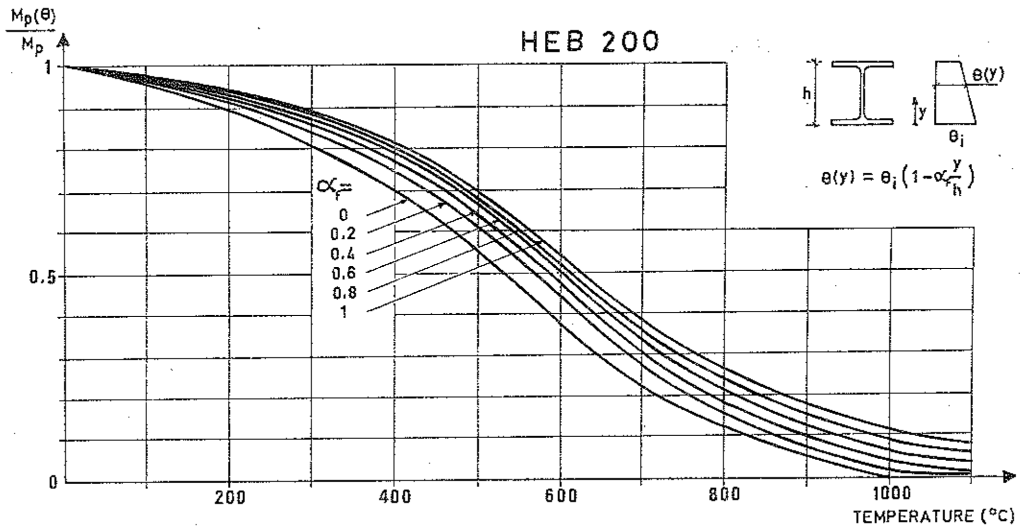


Figure 18. Calculated variation in plastic bending moment in terms of various temperature distribution over height of an I cross section [33]

In a fundamental paper by HARMATHY [34], the creep deflection of metal beams in transient heating processes is analyzed and discussed. Starting from the temperature history as known, a numerical technique is presented for a calculation of the connected stress-deformation behaviour with respect to creep bending of simply supported steel beams. The deflection history of the beam is determined from the stress-strain history of the mid-span cross-section, subdivided in elementary volumes by planes parallel to the plane of bending. The technique uses the creep model according to DORN-HARMATHY, which operates with a temperature compensated time. The total strain consists of

(1) elastic strain $\left.\begin{matrix}\\ \\ \end{matrix}\right\}$ quasi-instantaneous and
(2) thermal strain $\quad$ fully recoverable
(3) creep strain, representing all time-dependent and non-recoverable strains.

Fundamental assumptions for the theory are
(1) monosymmetrical beam in plane bending,
(2) Bernoulli's hypothesis is valid,
(3) temperature varies only along the depth of the beam,
(4) deflected shape of the beam can be approximated by a sine curve.

A close agreement between experimental and computed mid-span deflection histories proves the accuracy of the calculation technique. The technique derived could be used as an important tool in a systematic study of the necessity of taking the influence of creep into account in a practical structural fire engineering design. Some guidance in this respect is given in [11].

Important, combined theoretical and experimental studies of steel structures at elevated temperatures are reported by BIJLAARD, TWILT and WITTEVEEN [35], [36]. The studies comprise beams, columns and frames. The experiments were carried out by applying a new technique of small scale model tests. As a single illustration of the results, Fig. 19 shows how the critical steel temperature varies with an initial inclination α of the vertical column members for unbraced frames.

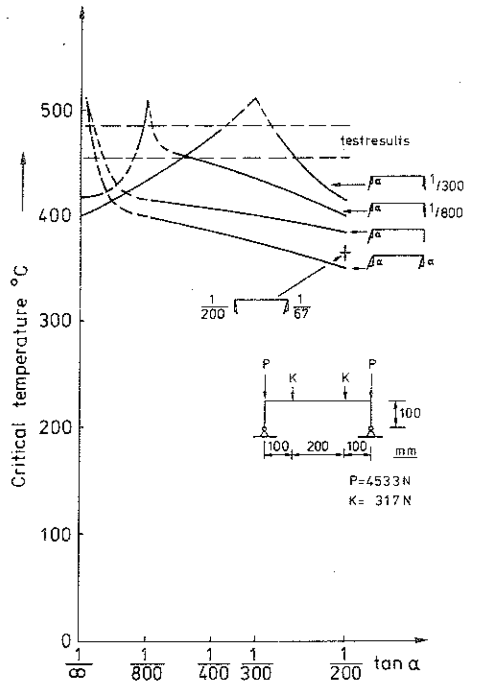


Figure 19. Influence of an initial inclination α of vertical members on critical temperature of unbraced steel frames [35], [36]

An inventory of projects - recently terminated, in progress or in preparation - on fire protection of steel structures has recently been published by ECCS Commission 3 [37]. In its present form, the inventory is limited to projects within the ECCS member countries, but it is planned to generalize it internationally and all CIB/W14 members are kindly asked to send information concerning other relevant projects.

6. Fire Exposed Concrete Structures

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.

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, derived by ANDERBERG-THELANDERSSON [38]

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

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. 20, 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.

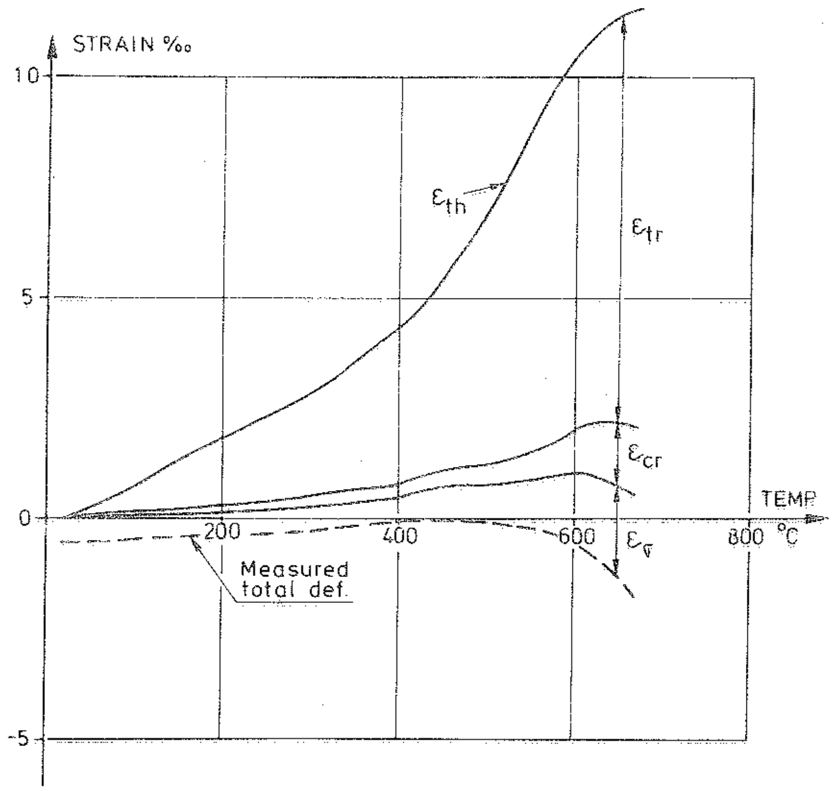


Figure 20. Relation between different strain components for a concrete specimen, initially stressed in compression to a stress level of 35% of the strength at 20°C, and heated to failure [38]

Additionally, Fig. 21 gives an independent verification of the validity of the described material behaviour model. The figure illustrates the stress relaxation process for initially unloaded concrete specimens, being heated at a constant rate of 1 and 5 $^{o}C \cdot min^{-1}$ respectively, under a fully restrained axial deformation. Measured and calculated values of the restraint load, specified in per cent of the ultimate load at ambient temperature, are compared and the agreement between the experimental and calculated results is extremely good.

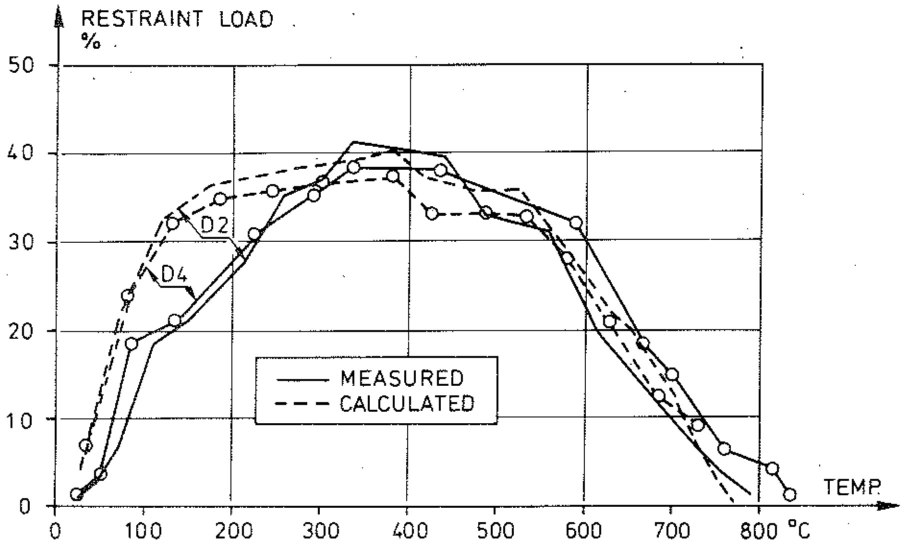


Figure 21. Measured and calculated restraint load - in per cent of ultimate load at ambient conditions - as a function of temperature for concrete specimens being heated under fully restrained axial expansion. Rate of heating: $5^{o}C \cdot min^{-1}$ (D2) and $1^{o}C \cdot min^{-1}$ (D4) [38]

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 [39] 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.

Fires-RC has been modified by ANDERBERG for a theoretical analysis of hyperstatic concrete beams, fire exposed from below [40]. Fig. 22 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.

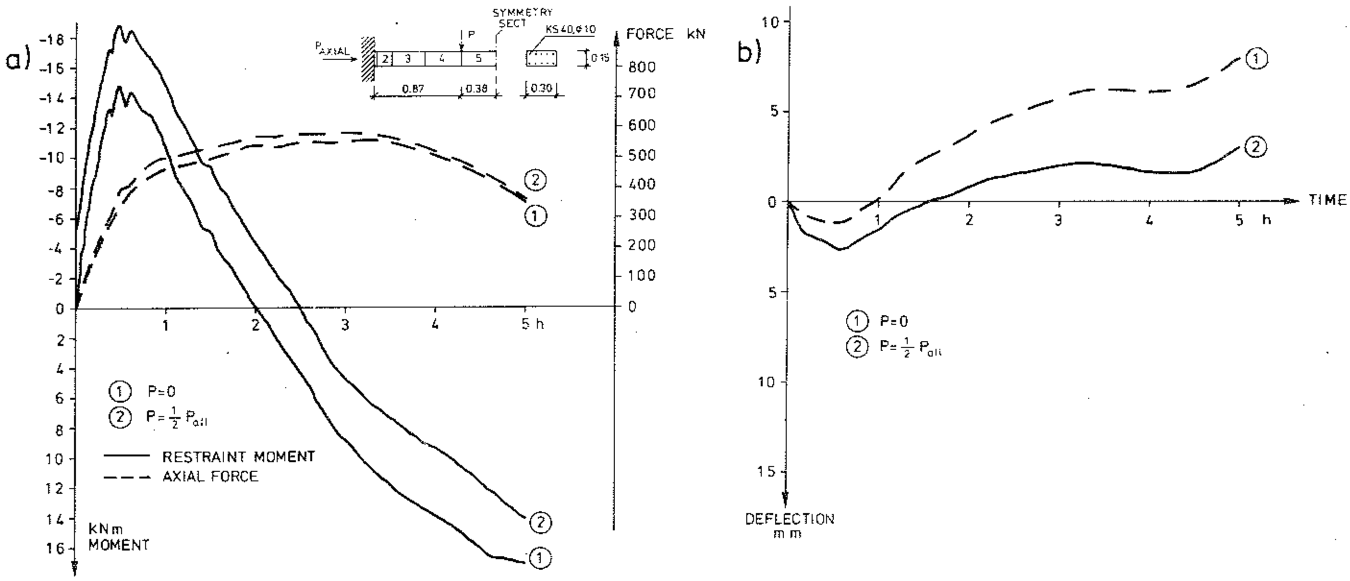


Figure 22. Structural fire behaviour of an unloaded, respectively loaded plate strip restrained against rotation as well as longitudinal movement at both ends. a) Bending restraint moment and axial restraint force, b) midpoint deflection. Fire process characteristics: $q = 500\ MJ \cdot m^{-2}$, $A\sqrt{h}/A_t = 0.04\ m^{1/2}$ according to Fig. 7 [40]

7. Fire Exposed Wooden Structures

In a paper by LIE [41], approximate formulas are derived for the fire resistance of laminated timber beams and columns, thermally exposed according to ISO 834. The formulas are directly based on earlier theoretical studies by IMAIZUMI (1962) and ÖDEEN (1970) and on the results of a large number of standard fire resistance tests on timber beams and columns, published in the international literature. The risk of lateral buckling, which not unfrequently decides in a fire engineering design of timber beams, is not discussed in the paper.

A very comprehensive manual for a fire engineering design of wooden structures on the basis of classification and results of standard fire resistance tests has been drawn up by KORDINA and MEYER-OTTENS [42]. As an introduction, the manual gives some basic information on ignition temperature, rate of char formation, temperature distribution within the cross section at fire exposure and mechanical properties of wood within the temperature range up to 100^{0}C. The main part of the manual comprises detailed lists with structural design guidance of the fire resistance of beams, columns, joints, floors, roofs and walls.

A useful thermal model for a calculation of one-dimensional charring rates in wood at a fire exposure is derived in a paper by HADVIG and PAULSEN [43]. The model is based on heat transfer by radiation as the only thermal influence. A comparison between the measurements and the model indicates that, with the parameters used, a reasonable physical-chemical adaption is achieved. Fig. 23 shows calculated and measured curves for mass loss and rate of mass loss as a function of time at an irradiation according to the ISO 834 temperature-time curve.

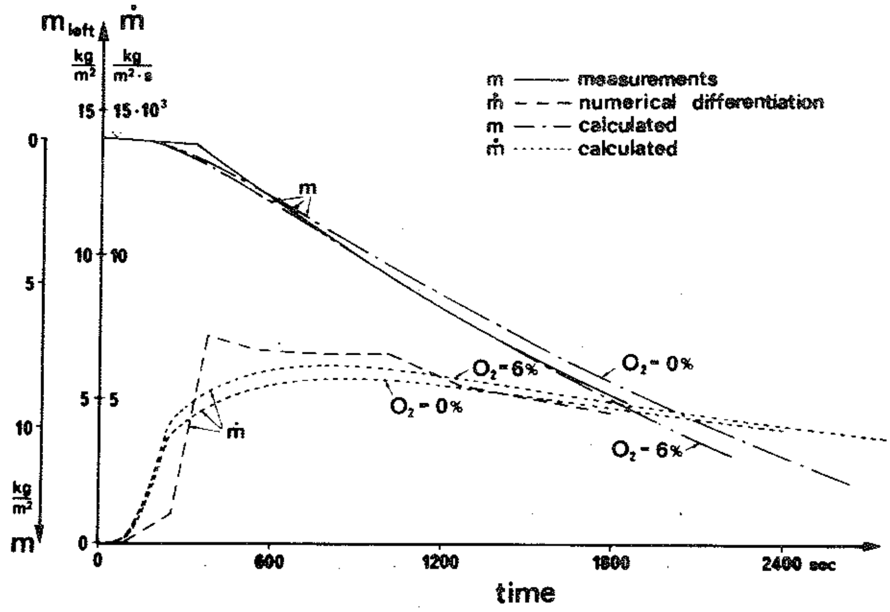


Figure 23. Mass loss m and rate of mass loss ṁ as a function of time. The experimental curves are valid for chip boards exposed to an irradiation according to ISO 834. The calculated curves indicate a small influence of varying oxygen concentration within the range shown, 0-6 %. The measured oxygen concentration at the test was approximately 3 volume % [43]

The contribution of HADVIG and PAULSEN marks a first break through in a development of a differentiated analytical design procedure for timber beams, based on a real fire exposure. The thermal model then can be used, for instance, for a calculation of the time curve of the charring depth of a laminated wooden structure, fire exposed according to the differentiated gas temperature-time curves in Fig. 7 with the fire load density q and the opening factor of the fire compartment $A\sqrt{h}/A_t$ as influencing parameters. The information received in that way gives, in combination with data on the mechanical properties of the virgin wood within the actual temperature range, the basis for a determination of the load-bearing capacity at varying time of a fire exposure. As an illustration to this, Fig. 24 [44] shows how the bending moment capacity of a rectangular wooden cross section with specified initial dimensions decreases during a fire in a compartment with an opening factor $A\sqrt{h}/A_t = 0.04\ m^{1/2}$ and with a fire load density q, varying between 25 and 502 $MJ \cdot m^{-2}$. The smooth full-line curves refer to a fire load of wood and connected fire exposure characteristics according to Fig. 7. The crossed full-line curves refer to a fire load of polyethylene and connected fire exposure characteristics in accordance to results of full-scale tests, made by BØHM [45].

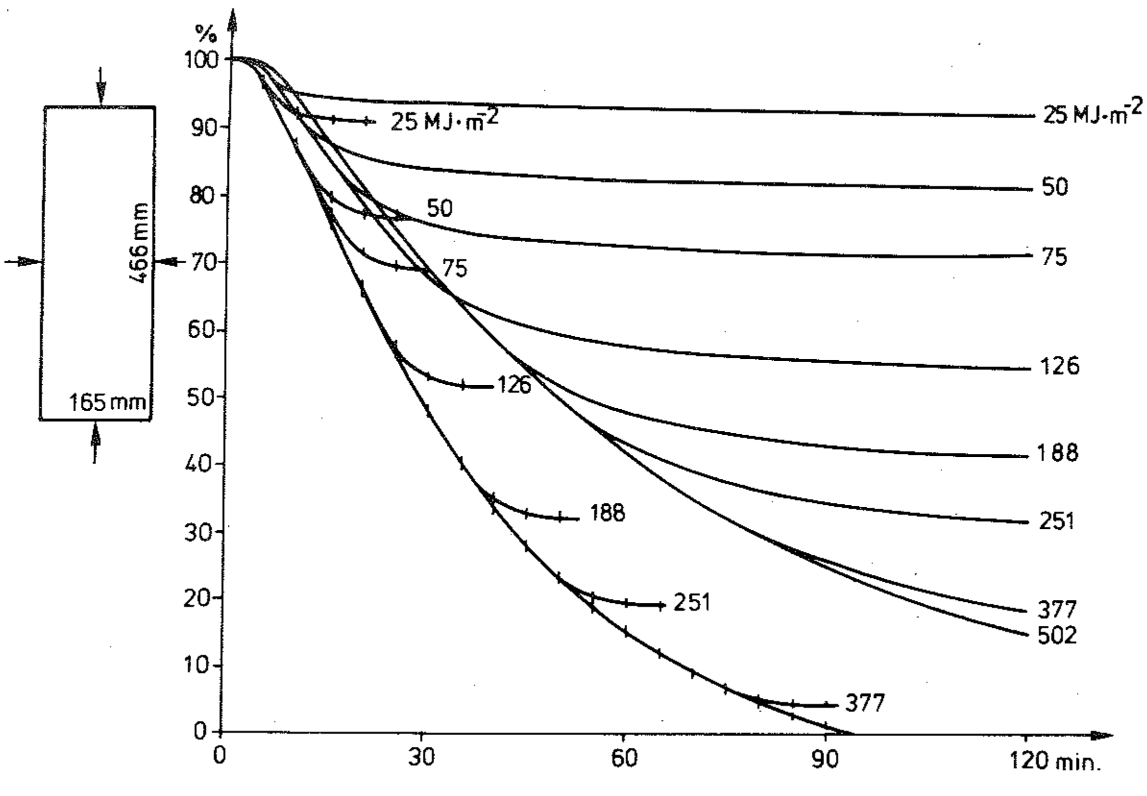


Figure 24. Time curves for the bending moment capacity of a rectangular wooden cross section - in percent of the initial bending moment capacity - fire exposed on all sides. Fire compartment with an opening factor $A\sqrt{h}/A_t = 0.04\ m^{1/2}$. Fire load density q, varying between 25 and 502 $MJ \cdot m^{-2}$. Smooth curves refer to a fire exposure according to Fig. 7 (fire load of wood), crossed curves to a corresponding fire exposure at a fire load of polyethylene [44]

8. Fire Safety of Load-Bearing Structures

In a general sense, the fire engineering design problem is non-deterministic. Performance has to be described and measured in probabilistic terms.

This is one essential perspective from which we have to judge or appraise the building fire safety code systems now in force. Historically, they had to be written without actually stating their objective level of safety and, still far less, without any analytical measurement of the objectives involved. For this reason, there is an urgent need for future attempts to evaluate the levels of safety inherent in present local and national fire protection regulations and to develop rational, reliability-based design methods, leading to safety levels which are consistent with the relevant functional requirements [46].

The problem to evaluate the level of safety in structural fire protection codes is partly dealt with by LIE and STANZAK for fire exposed steel structures in [16]. This paper describes a structural fire design based entirely on scientific and engineering methods. It refers rough formulas for the critical steel temperature for axially loaded columns in elastic buckling and for joists or trusses and beams, transversally loaded. The critical temperature is defined as the cross sectional average temperature, at which the structural member no longer can perform its load-bearing function. The paper further refers approximate formulas for the standard fire resistance of unprotected steel columns, steel columns with a light protection and concrete protected steel columns, based on work at National Research Council, Canada.

For a specified protected steel column, the critical fire load density, corresponding to a critical steel temperature of $1000^{o}F$ ($538^{o}C$), is evaluated theoretically for three fire compartments of different opening factor: $A\sqrt{h}/A_t$ = 0.1 (large opening factor), 0.05 (intermediate opening factor) and 0.02 $m^{1/2}$ (small opening factor). The evaluation is based on fully developed fires which generally are assumed to be ventilation controlled. In combination with a known frequency distribution of the fire load density - exemplified in the paper for offices - the critical fire load densities can be transformed to a probability of collapse for the fire exposed steel column, put equal to the probability of exceeding the respective critical fire load density. In the case studied, this gives a failure probability varying between less than 10^{-6} and $8 \cdot 10^{-3}$ for the steel column - having a standard fire resistance of 1.5 hours - placed within the three different fire compartments. In the study, all other influencing parameters are dealt with as fully deterministic.

These results may be supplemented by the results of a corresponding Swedish probabilistic study, in which the uncertainties in all influencing parameters have been taken into account - cf. [46]. The following table fragmentarily illustrates these latter results, showing the range of variation for the safety index β and the probability of failure P_f for three different design procedures: I. Design based on classification and standard fire resistance test, II. Differentiated Swedish design procedure according to the scheme in Fig. 6, and III. Improved differentiated design, based on II. The values in the table apply to an insulated, simply supported steel beam as a part of a floor or roof assembly in office buildings.

Design procedure	Range of β	Range of P_f	$(P_f)_{max}/(P_f)_{min}$
I. Classification, standard fire resistance test	1.77-3.69	$(1-400)10^{-4}$	~ 400
II. Present differentiated Swedish design model	1.66-2.84	$(23-500)10^{-4}$	~ 20
III = II, improved by statistically derived load factors	2.35-2.45	$(72-95)10^{-4}$	~ 1.5

For the structural member, designed in accordance to the conventional method based on classification and standard fire resistance test (case I), the table gives a very large variation range of β from 1.77 to 3.69. For the present Swedish differentiated design model (case II), the variation range of β is essentially more narrow. Completing this design model with statistically derived load factors (case III) will improve the consistency of β considerably by giving a very narrow range from 2.35 to 2.45.

The corresponding range of the probability of failure P_f is shown in the table, too. Related to this quantity, the difference between the three design procedures is extremely striking with the respective ratios $(P_f)_{max}/(P_f)_{min} \sim 400$, 20 and 1.5. The P_f values presented then are connected to a probability = 1 for a fire outbreak leading to flashover within the fire compartment.

The urgency of extensive, internationally co-ordinated research activities for improving the present building code systems in order to achieve fire safety levels, which are consistent with the relevant functional requirements, is strongly stressed by the CIB W14 Symposium "Fire Safety in Buildings: Needs and Criteria", held in Amsterdam 1977-06-02/03 and organized by TNO [47]. Essential policy related questions to be dealt with within this work are

- the social acceptability of fire risks,
- the money to be spent on fire safety, and
- the optimum combination of different fire prevention measures for a given budget.

The necessity of joint activities between all responsible - the regulatory bodies, the fire services, the structural engineers and architects, the insurance companies, the fire scientists - then is obvious.

9. Summary Remarks

The report deals with recent contributions within the subject field of structural fire protection, primarily those contributions circulated within CIB W14 since the commission meeting 1976. The report is structured partly with respect to different fire engineering design systems, partly with respect to different structural materials.

The contributions give evidence of a development towards an increased application of analytical design methods, directly or indirectly based on real fire exposure characteristics. For interior load-bearing structures or structural members, such analytical design methods have arrived at a comparatively advanced level, especially as fire exposed steel structures are concerned. Recent important contributions now are opening the door for an analytical approach also for fire exposed external columns and beams. 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, further will enable an essential enlargement of the area of application for an analytical qualified design.

The CIB W14 Symposium "Fire Safety in Buildings: Needs and Criteria", held 1977 in Amsterdam, strongly stresses the urgency of research activities for improving the present building code systems in order to achieve fire safety levels which are consistent with strictly defined functional requirements. The contributions within this research area are as yet very few.

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