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BEHAVIOUR AND ANALYTICAL DESIGN OF FIRE EXPOSED STEEL STRUCTURES,
INSULATED WITH GYPSUM PLASTER SLABS

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A development of analytical design procedures, based on well-defined functional requirements, is an important task of the future fire research within different fields of the overall fire safety concept. Such procedures, successively replacing the present, internationally prevalent, schematic design methods, are necessary for getting an improved economy and for enabling more qualified and reliable fire safety analyses. A derivation of such analytical design systems is also in agreement with the present trend of development of the building codes and regulations in many countries towards an increased extent of functionally based requirements and performance criteria.

In the ideal case, a rational fire design methodology includes as essential components [1]

- * analytical modelling of relevant processes; verification of model validation and accuracy; determination of critical design parameters,
- * formulation of functional requirements, independent of choice of design process and expressed either in deterministic or probabilistic terms,
- * determination of design parameter values, and
- * verification by the means of a reliability analysis that the choice of safety factors leads to safety levels, which are consistent with the expressed functional requirements.

For a fire engineering design of load-bearing structures and partitions, a differentiated analytical procedure is permitted to be applied in Sweden, as one alternative, since about ten years. The procedure constitutes a direct design method based on temperature characteristics of the fully developed compartment fire as a function of the fire load density, the

ventilation of the fire compartment and the thermal properties of the structures enclosing the fire compartment. The design method is approved for a general practical use by the National Swedish Board of Physical Planning and Building [2]. For facilitating the practical application, design diagrams and tables are systematically produced, giving directly, on one hand, the design temperature state of the fire exposed structure, on the other, a transfer of this information to the corresponding design load-bearing capacity of the structure; c.f., for instance [3], [4], [5], [6].

1. Principles of an Analytical Design of Fire Exposed Structures

In a generalized summary way, an analytical design method for fire exposed structures, based on well-defined functional requirements, can be described according to Fig. 1.

The design fire load density, the fire compartment characteristics and the fire extinguishment and fire fighting characteristics constitute the basis for a determination of the design fire exposure, given as the gastemperature-time curve $T-t$ of the fully developed compartment fire. Depending on the type of practical application, the load-bearing function of the structure can be required to be fulfilled for

- * the complete fire process,
- * a shortened fire process, limited by the time t_{ext} , necessary for the fire to be extinguished under the most severe conditions, or
- * a shortened fire process, limited by the design evacuation time t_{esc} for the building.

Together with the structural design data, the design thermal properties and the design mechanical strength of the structural materials, the design fire exposure gives the design temperature state and the design load-carrying capacity R_d as the lowest value during the relevant fire process.

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

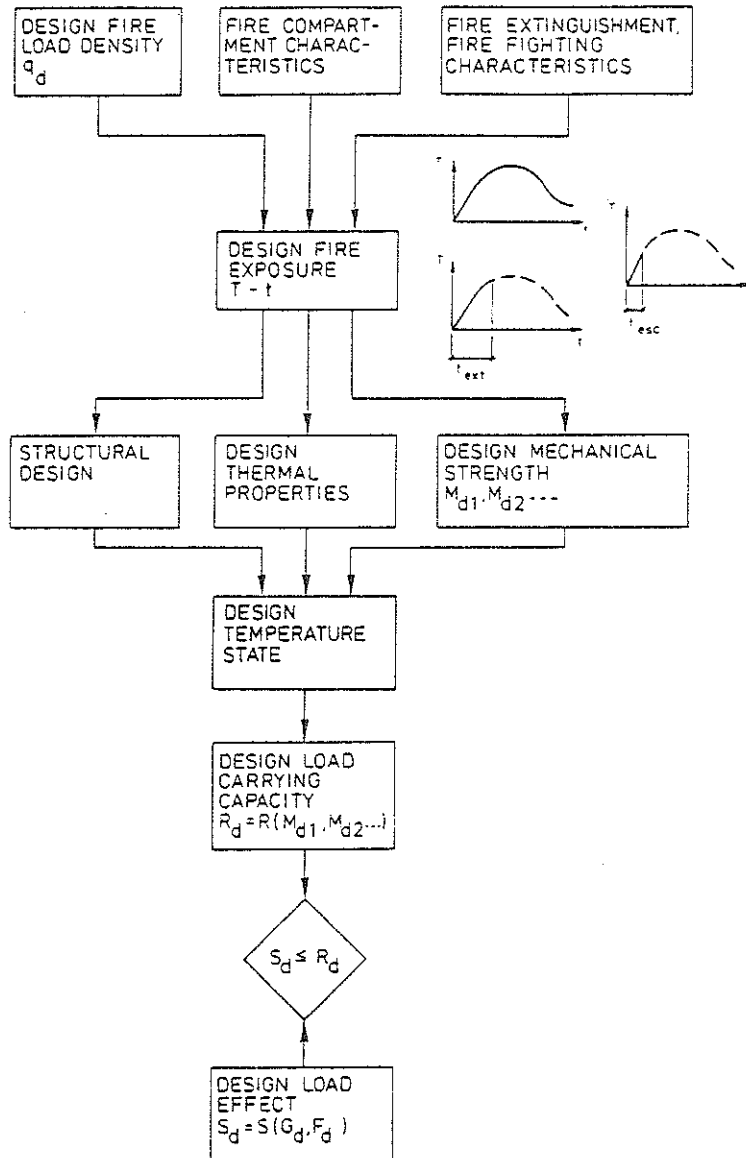


Figure 1. Procedure of an analytical design of fire exposed load-bearing structures

Following a recent draft of safety regulations [7], the determination of the design load effect S_d starts from characteristic values of permanent and variable loads G_k and F_k , connected to a defined probability of excess during a specified time period (Fig. 2). A multiplication by partial factors γ and load combination factors ψ transfers the characteristic load values to design loads G_d and F_d . The load combination factors ψ then may be differentiated with respect to whether a complete evacuation of people can be assumed or not in the event of fire. Finally, the design loads are combined and transformed to the design load effect at fire S_d .

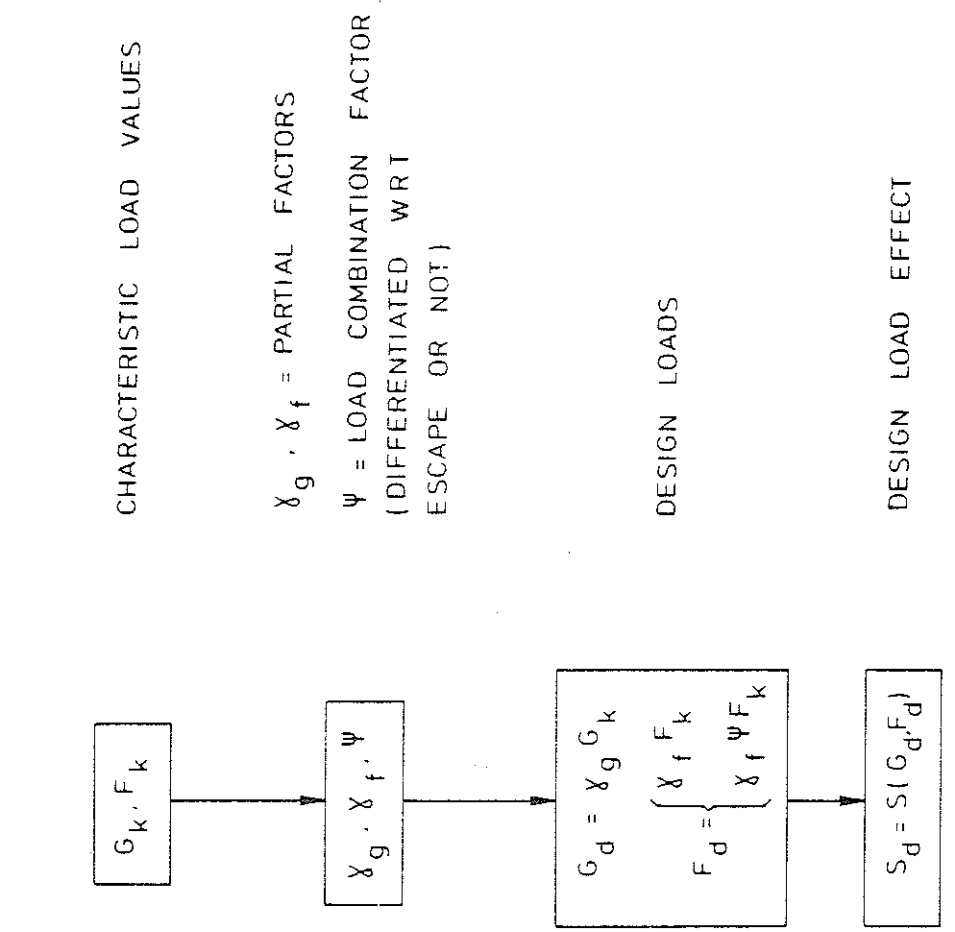


Figure 2. Procedure of determination of design load effect S_d

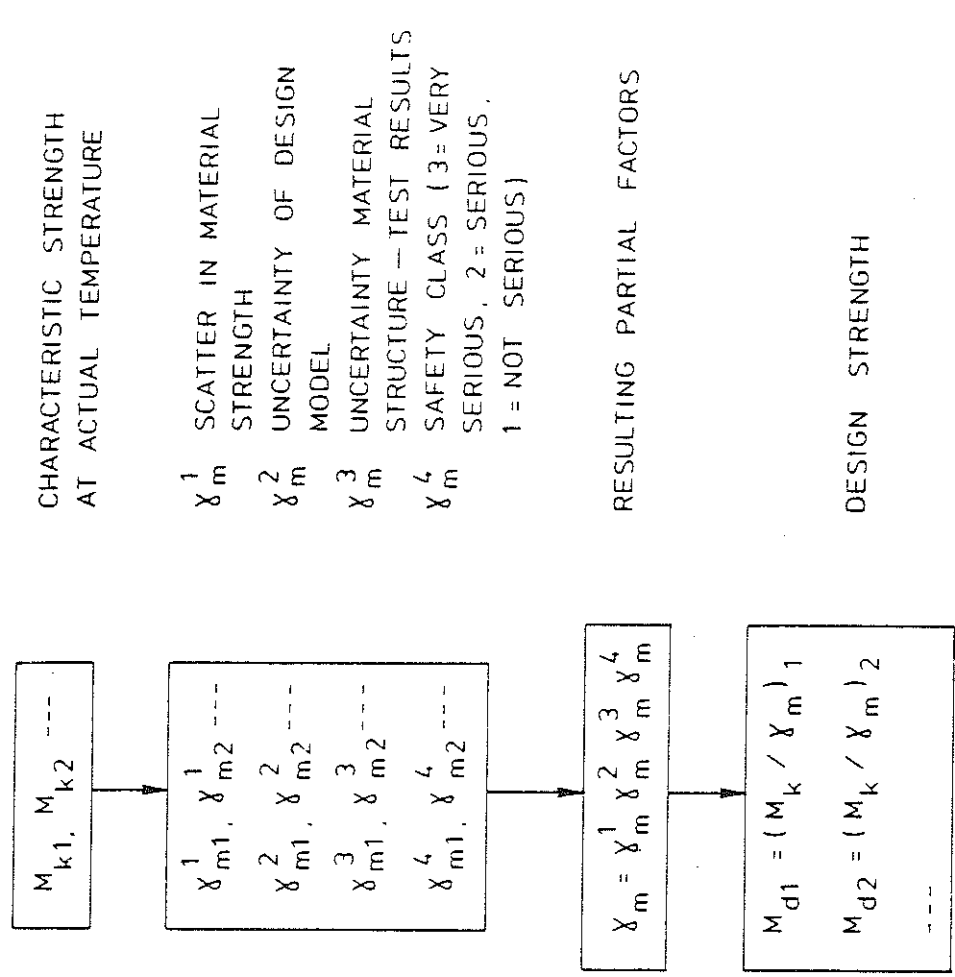


Figure 3. Procedure of determination of design strength M_d

Analogously, the design material strength M_d is to be calculated via characteristic strength values M_k at actual temperature, divided by resulting partial factors γ_m (Fig. 3). The characteristic strength values are defined as corresponding to specified fractiles of the probability density distribution. The different partial factors γ_m^1 , γ_m^2 , γ_m^3 , and γ_m^4 , are expressing the influence of the scatter in material strength, the uncertainty of the design model, the uncertainty in relation between material property in the structure and material property determined in test, and the safety class, respectively. The predicted extent of personal and property damage at failure - very serious, serious, not serious - decides the safety class.

A similar approach - as outlined for the design load effect S_d and the design mechanical strength M_d - can be applied also to the design fire load density q_d and the design thermal properties of the structural materials.

A methodology for a probabilistic analysis of fire exposed steel structures, connected to the described design method, has been developed in [8]. The methodology comprises a general systematized scheme for the identification and evaluation of the various sources and kinds of uncertainty in the differentiated structural fire engineering design. The structure of the methodology is quite general and applicable to a wide class of structures and structural elements.

Described in a more detailed way, a direct, differentiated, analytical design of fire exposed load-bearing structures or structural members, inside a fire compartment, includes the following steps - Fig. 4.

The basis of the design is given by the fully developed compartment fire exposure. Decisive entrance quantities then are

- (1) nominal load and load factor for fire load density,
- (2) combustion properties of this design fire load,
- (3) size and geometry of the fire compartment,
- (4) ventilation characteristics of the fire compartment, and
- (5) thermal properties of structures enclosing the fire compartment.

These quantities jointly determine the rate of burning, the rate of heat release, and the design gas temperature-time curve of the complete fire process. Together with

- (6) structural data for the proposed structure,
- (7) thermal properties of structural materials, and
- (8) coefficients of heat transfer for various surfaces of the structure

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

- (9) mechanical properties of structural materials (Fig. 3), and
- (10) load characteristics

as further entrance quantities the time variation of restraint forces and moments, thermal stresses, and load-carrying capacity R can be determined. The lowest value of R during the complete fire process 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, the design load effect at fire S_d is defined, interdependent on non-fire design procedure (Fig. 2).

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

For buildings containing activities, which are particularly important from, for instance, an economical point of view, there may be the motive for requiring that the building can be used again after a fire, almost immediately or very soon, for the current activities in a full extent. If a fire engineering design also includes such a requirement on re-serviceability of the structure after fire, the design procedure is to be as follows.

From the time curve of the load-carrying capacity R , the design residual load-carrying capacity R_{rd} of the structure after fire is obtained as an end information. This quantity R_{rd} must be compared with the design load effect at service, non-fire state, on the structure S_{rd} , given by the corresponding nominal loads and load factors for dead load, live load, etc.

For fire-exposed, exterior, load-bearing structures, the procedure for a direct, differentiated design will be modified. For such a structure,

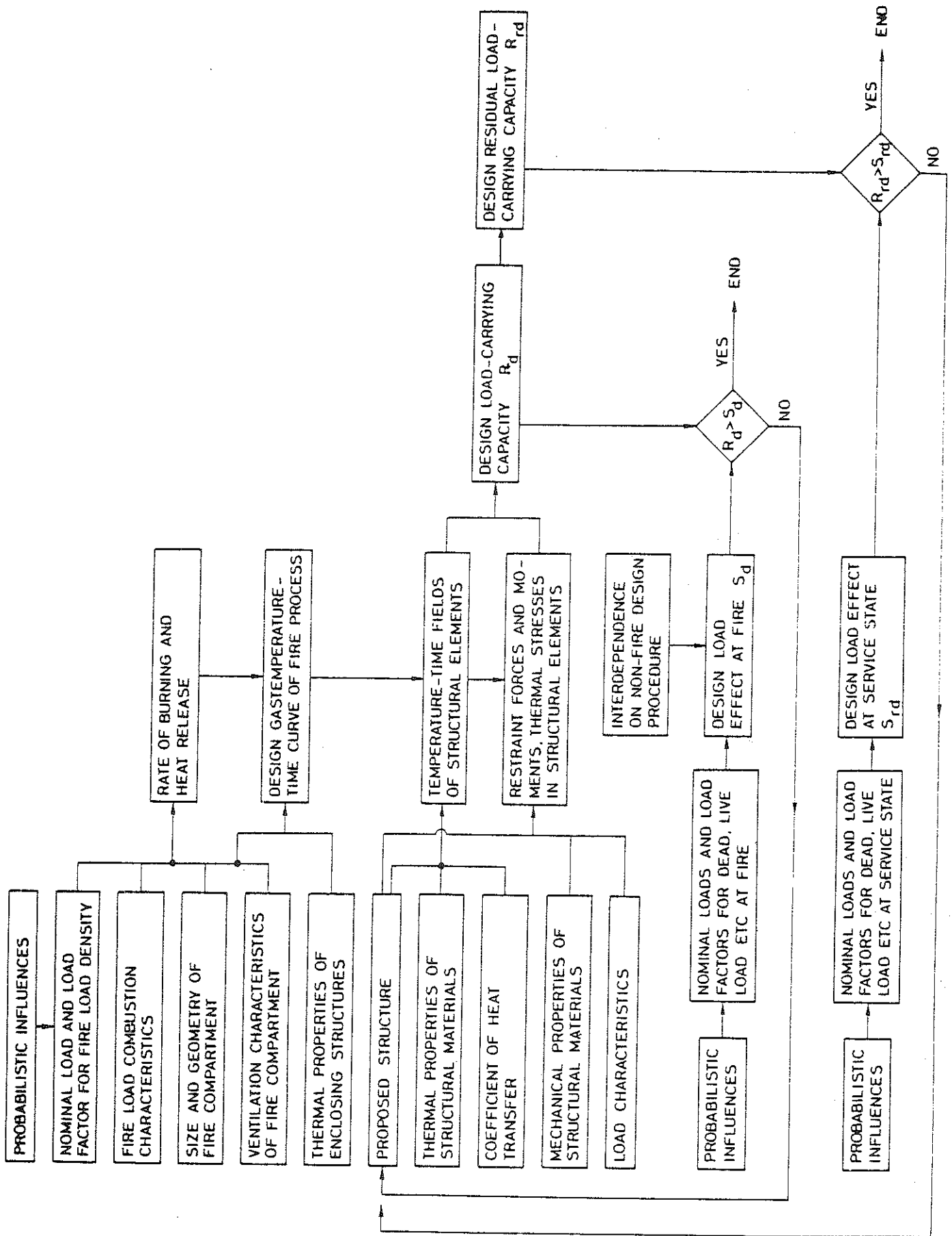


Figure 4. Procedure of a differentiated, analytical fire engineering design of load-bearing structures with additional requirement on re-serviceability after fire. Interior structures

the transient temperature fields are 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; cf., for instance [9], [10].

2. Fire Load Density and Gas Temperature-Time Curves of a Fully Developed Compartment Fire

At known combustion characteristics of the fire load, the gas temperature-time curve of a fully developed compartment fire can be calculated in the individual practical application from the heat and mass balance equations of the fire compartment with regard taken to the size, geometry and ventilation of the compartment, and to the thermal properties of the structures enclosing the compartment - Fig. 5 [2], [4], [6], [11], [12], [13], [14], [15], [16], [17].

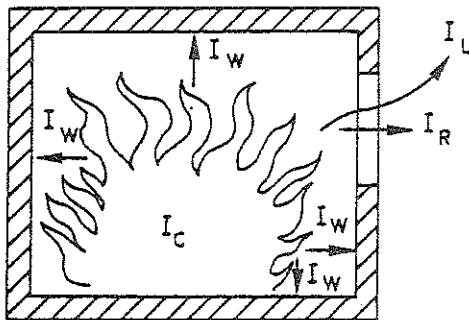


Figure 5. Energy balance equation $I_C = I_L + I_W + I_R$ of a fire compartment. I_C is the heat release per unit time from the combustion of the fuel, and I_L , I_W and I_R the quantities of energy removed per unit time by change of hot gases against cold air, by heat transfer to the surrounding structures, and by radiation through the openings of the compartment, respectively

For interior, load-bearing structures and partitions, the fire engineering design provisionally can be based on gas temperature-time curves T_t-t according to Fig. 6, [2], [4], [6], [13], which applies to a fire compartment with surrounding structures made of a material with a thermal conductivity $\lambda = 0.81 \text{ W}\cdot\text{m}^{-1}\cdot\text{°C}^{-1}$ and a heat capacity $\rho c_p = 1.67 \text{ MJ}\cdot\text{m}^{-3}\cdot\text{°C}^{-1}$ (fire compartment, type A). Entrance parameters of the diagrams are the fire load density q , defined by the formula

$$q = \frac{1}{A_t} \sum \mu_{\nu} m_{\nu} H_{\nu} \quad (\text{MJ}\cdot\text{m}^{-2}) \quad (1)$$

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

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

The non-dimensional factor μ_v is a function of type of fuel, geometrical properties of fuel, and the position of fuel in a fire compartment, among other things. For some types of fire load components, μ_v will depend on the time of fire duration and on the gas temperature-time characteristics of the fire compartment. Bookcases and floor coverings are examples of fire components whose real degree of combustion is low, and whose μ_v values are probably appreciably below unity. At present, however, there is a lack of experimentally substantiated and verified μ_v values, and it is therefore usually necessary in the course of practical design to employ a fire load calculation with μ_v generally put equal to unity.

As a rule, the design fire load density is to be determined on the basis of statistical investigations for the type of building or premises in question. Such statistical investigations have been carried out for dwellings, offices, administration buildings, schools, stores, and hospitals [2], [4], [6]. As a temporary regulation, the Swedish Building Code authorizes the 80 percent level of the statistical distribution curve to be applied as the design fire load density.

The gas temperature-time curves in Fig. 6 have generally been determined on the assumption of ventilation controlled fires. For fires, which are fuel bed controlled in reality, this assumption leads to a structural fire engineering design on the safe side in practically every case, giving an overestimation of the maximum gastemperature and a simultaneous, partly

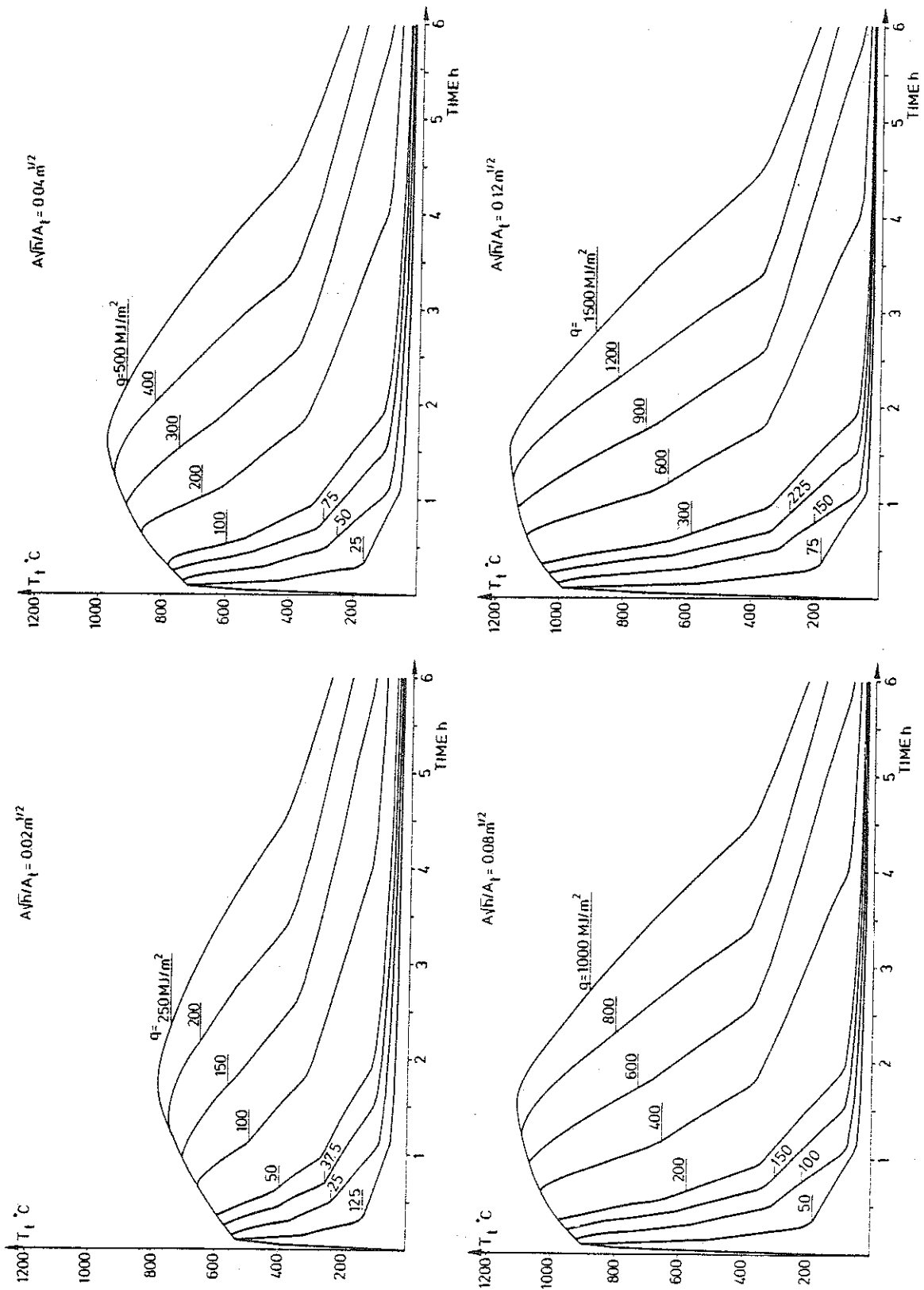


Figure 6. Gas temperature-time curves T_t - t of the complete process of fire development for different values of the fire load density q and the opening factor $A\sqrt{h}/A_t$. Fire compartment, type A

balancing underestimation of the fire duration. For the minimum load-bearing capacity, which thermally can be seen as an integrated effect, the gas temperature-time curves in Fig. 6 give reasonably correct results, verified in [4], [8], [14].

As pointed out, the gas temperature-time curves in Fig. 6 apply to a certain fire compartment, type A, specified with respect to the thermal properties of its surrounding structures. Fire compartments with surrounding structures of deviating thermal properties can be transferred to fire compartment, type A, via fictitious values of the fire load density q_f and the opening factor $(A\sqrt{h}/A_t)_f$ in accordance to Table 1 in the appendix [2], [4], [6].

3. Design Temperature State of Fire Exposed Steel Structures and Partitions

For a fire exposed, uninsulated steel structure, the energy balance equation gives the following formula for a determination of the steel temperature-time curve T_s-t - Fig. 7

$$\Delta T_s = \frac{\alpha}{\rho_s c_{ps}} \cdot \frac{F_s}{V_s} (T_t - T_s) \Delta t \quad (^\circ\text{C}) \quad (2)$$

where

ΔT_s = change of steel temperature ($^\circ\text{C}$) during time step Δt (s),

α = coefficient of heat transfer at fire exposed surface of structure ($\text{W}\cdot\text{m}^{-2}\cdot^\circ\text{C}^{-1}$),

ρ_s = density of steel material ($7850 \text{ kg}\cdot\text{m}^{-3}$),

c_{ps} = specific heat of steel material ($\text{J}\cdot\text{kg}^{-1}\cdot^\circ\text{C}^{-1}$),

F_s = fire exposed surface of steel structure per unit length (m),

V_s = volume of steel structure per unit length (m^2),

T_t = gas temperature ($^\circ\text{C}$) within fire compartment at time t (s).

Eq. (2) presupposes that the steel temperature T_s is uniformly distributed over the cross section of the structure at any time t .

The coefficient of heat transfer α can be calculated from the approximate formula

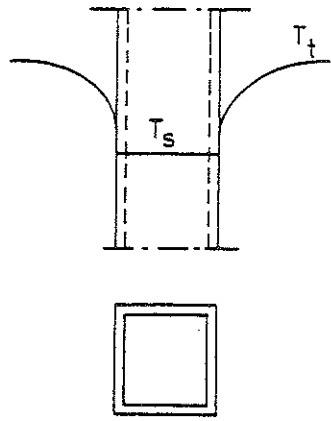


Figure 7. Fire exposed, uninsulated steel structure. T_t = gas temperature within fire compartment, T_s = steel temperature at time t

$$\alpha = 23 + \frac{5.77 \epsilon_r}{T_t - T_s} \left[\left(\frac{T_t + 273}{100} \right)^4 - \left(\frac{T_s + 273}{100} \right)^4 \right] \quad (\text{W} \cdot \text{m}^{-2} \cdot \text{C}^{-1}) \quad (3)$$

giving an accuracy which is sufficient for ordinary practical purposes. ϵ_r is the resultant emissivity which for practical applications can be chosen according to the following table, giving values which generally are on the safe side.

1. Column, fire exposed on all sides	$\epsilon_r = 0.7$
2. Column, outside a facade	0.3
3. Floor structure, composed of steel beams with a concrete slab on the lower flange of the beams	0.5
4. Steel beams with a floor slab on the upper flange of the beams	
4a. Beams of I cross section with width/height ≥ 0.5	0.5
4b. Beams of I cross section with width/height < 0.5	0.7
4c. Beams of box cross section and trusses	0.7

In [2], [4], [5], [6], more accurate values are given for the resultant emissivity ϵ_r , as concerns the application case 4.

At a given gas temperature-time curve T_t - t of the fire compartment, the steel temperature T_s can be directly calculated from Eqs. (2) and (3) with regard taken to the temperature dependence of c_{ps} and α . Such computations have been carried out in a systematized way, giving the basis of design in Table 2 [4]. From this table, the maximum steel temperature $T_{s,max}$ during a complete compartment fire can be determined directly as a function of the fictitious fire load density q_f , the fictitious opening

factor $(A\sqrt{h}/A_t)_f$, the F_s/V_s ratio and the resultant emissivity ϵ_r . The values of the table are connected to gas temperature characteristics according to Fig. 6.

For a fire exposed, insulated steel structure, analogously, a simplified energy balance equation gives the following formula for a direct determination of the steel temperature-time curve T_s-t - Fig. 8

$$\Delta T_s = \frac{A_i}{(1/\alpha + d_i/\lambda_i)\rho_s c_{ps} V_s} (T_t - T_s)\Delta t \quad (^\circ\text{C}) \quad (4)$$

with the additional quantities

A_i = interior jacket surface area of insulation per unit length (m),

d_i = thickness of insulation (m),

λ_i = thermal conductivity of insulating material ($\text{W}\cdot\text{m}^{-1}\cdot^\circ\text{C}^{-1}$).

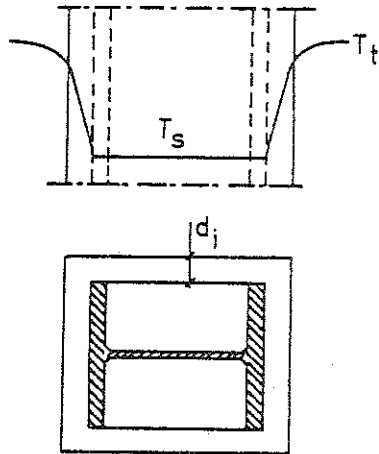


Figure 8. Fire exposed, insulated steel structure. T_t = gas temperature within fire compartment, T_s = steel temperature at time t

Eq. (4) presupposes that the steel temperature T_s is uniformly distributed over the cross section of the structure at any time t , that the temperature gradient is linear and the heating contribution negligible for the insulation, and that the heat transfer is one-dimensional.

Computations, originating from Eqs. (3) and (4), enable a production of a systematized design basis, facilitating an analytical, differentiated fire engineering design in practice. An example from such a design basis is referred in Table 3 [4], giving the maximum steel temperature $T_{s,\text{max}}$ during a complete compartment fire for varying values of the fictitious fire load density q_f , the fictitious opening factor $(A\sqrt{h}/A_t)_f$, the structural parameter A_i/V_s , and the insulation parameter d_i/λ_i . The values of the table are connected to gas temperature characteristics according to Fig. 6.

Table 3 has been computed on the assumption of a constant thermal conductivity of the insulating material λ_i , chosen as an average value for the whole compartment fire process. Calculations, carried through systematically, are verifying that this average value of λ_i approximately coincides with the value, determined for an insulation temperature equal to the maximum steel temperature $T_{s,max}$.

For a specific insulating material, systematized design diagrams or tables can be computed very accurately with regard to the temperature dependence of the thermal properties of the steel as well as the insulating material. The influence of an initial moisture content and of a disintegration of the insulating material can be considered, too. Practically, such a determination can be carried out over a numerical data processing by computers on the basis of a finite difference or a finite element method. A great number of design tables, computed according to such an accurate procedure, are presented in [4]. Table 4 exemplifies this, giving the maximum steel temperature $T_{s,max}$ at varying fire and structural design characteristics for a fire exposed steel structure, insulated with gypsum plaster slabs, type Gyproc, of density $790 \text{ kg}\cdot\text{m}^{-3}$. The thermal properties of the gypsum plaster slabs then have been assumed to depend on the insulation temperature according to Fig. 9 [18], constructed on the basis of results from small scale and full scale tests and of information in the literature [19]. The influence of the disintegration of the slab material is considered.

In [4], an analytical model is derived for a simplified determination of the temperature-time fields of a steel beam construction according to Fig. 10 - composed of a reinforced concrete slab, load-bearing steel beams, and an insulating ceiling - exposed to a fire from below. By applying this computational model in a systematic way, a design basis has been determined, facilitating a calculation of the steel beam temperature T_s , assumed as uniformly distributed over the cross section of the beams. The design basis is exemplified in Table 5 [4], which gives the maximum steel beam temperature $T_{s,max}$ during a complete compartment fire for ^{varying} values of the fictitious fire load density q_f , the fictitious opening factor $(A\sqrt{h}/A_t)_f$, the structural parameter F_s/V_s , and the insulation parameter d_i/λ_i . F_s denotes the surface area of the steel beam, less the part covered by the concrete slab, and V_s the volume of the steel beam, per unit length. The

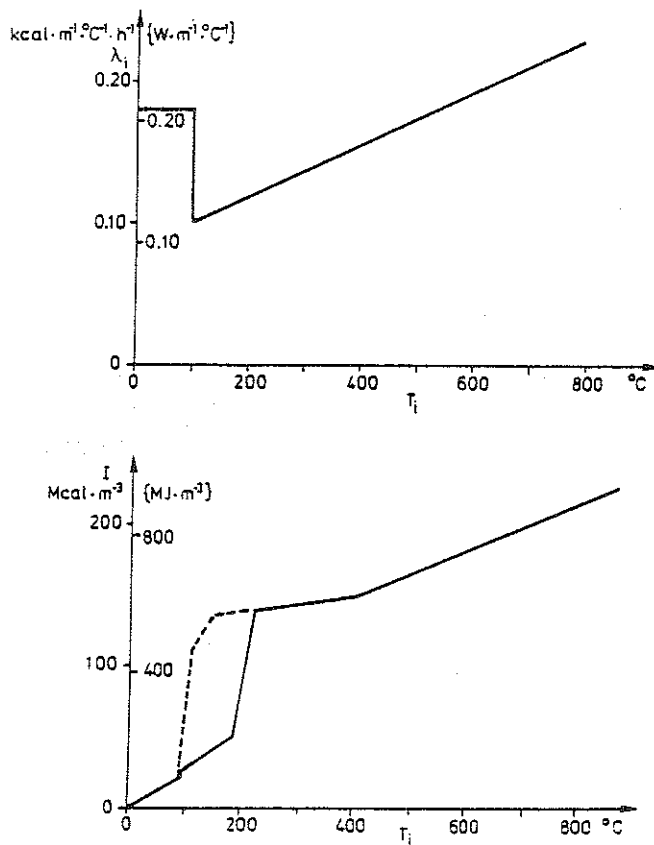


Figure 9. Thermal conductivity λ_i and enthalpy $I (= \int_0^T c_p dT)$ as a function of insulation temperature T_i for gypsum plaster slabs, type Gyproc, of density $790 \text{ kg}\cdot\text{m}^{-3}$. For enthalpy I , full line refers to a rapid heating and dashed line to a slow heating [18]

values, given in brackets in the table, denote the corresponding maximum temperature at the centre level of the ceiling. The values of the table are connected to gas temperature characteristics according to Fig. 6.

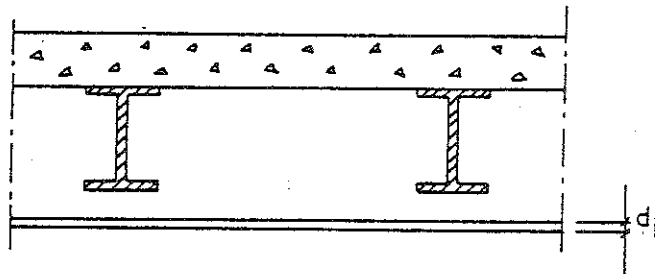


Figure 10. Floor structure, composed of a reinforced concrete slab, load-bearing steel beams, and an insulating ceiling

For several types of steel beam constructions with a suspended, insulating ceiling, the fire resistance of the ceiling and its fastening devices will be the decisive design criterion instead of the temperature of the steel beams. The ceiling can get a serious crack formation or fall down, partially or completely, after a comparatively short fire exposure. Under such conditions, the maximum steel beam temperature cannot be determined from Table 5 solely on the basis of the thickness d_i and the thermal conductivity λ_i of the ceiling. If results are available for a type of a suspended ceiling from a standard fire resistance test, these results can be used for deriving a fictitious value of the insulation parameter $d_i/\lambda_i - (d_i/\lambda_i)_{\text{fict}}$ - which describes the real fire behaviour of the suspended ceiling, including its fastening devices. From the test results, also a possible critical failure temperature of the suspended ceiling can be estimated. Cf., further [4].

After the determination of $(d_i/\lambda_i)_{\text{fict}}$ and the critical temperature of a type of a suspended ceiling, the analytical differentiated fire design can be carried out by a direct application of Table 5. Parallely, then the maximum temperature at the centre level of the ceiling according to the table must be controlled against the critical temperature of the ceiling.

Fictitious d_i/λ_i values and critical temperatures have been determined for a number of types of suspended ceilings in a series of standard fire resistance tests performed at the National Swedish Institute for Testing and Metrology in Stockholm [20]. The compositions of these suspended ceilings, the results obtained and the characteristics derived are set out in Table 6 [4].

The design basis, reproduced in Table 2 to 5, generally assumes the steel temperature to be uniformly distributed over the cross section of the beam or column at any time t . A more accurate theory, which enables a determination of the temperature variation over the cross section of the steel structure, is presented in [21], together with computer routines. The algorithm described can easily be coupled to most finite element programs. An illustration of the capability of the theory is given in Fig. 11, which shows calculated temperature distribution along the line of symmetry of a gypsum insulated steel beam with a concrete slab at the top flange at selected times of a standard fire resistance test according to ISO 834.

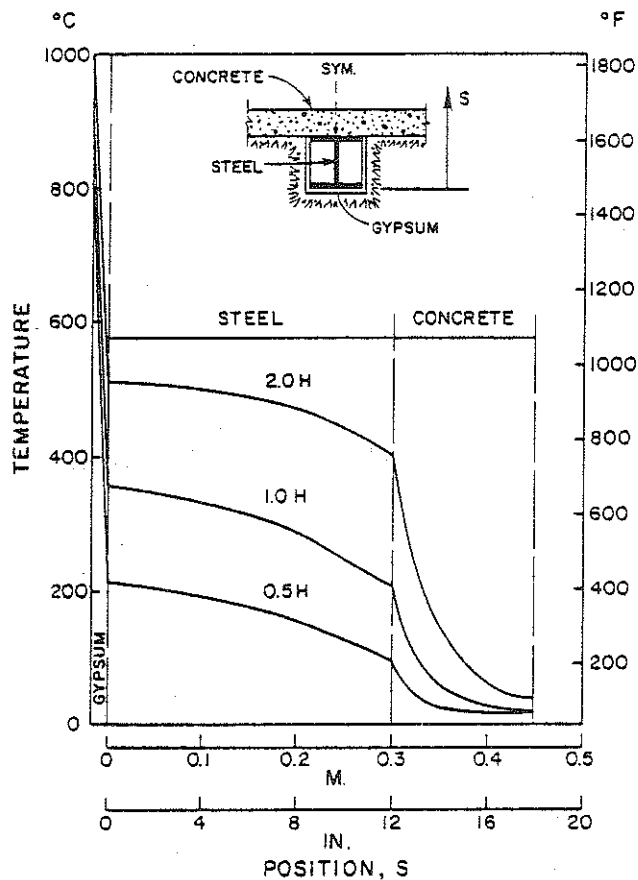


Figure 11. Calculated temperature distribution along line of symmetry, of a steel beam, insulated by a 16 mm gypsum board (density $770 \text{ kg}\cdot\text{m}^{-3}$) and carrying a 150 mm concrete slab on top flange, at selected times of a thermal exposure according to ISO 834 [21]

As a complement to the design temperature state of fire exposed load-bearing steel structures, dealt with above, also some remarks will be given on the fire engineering design of partitions. The performance requirements for partitions imply that these must prevent a penetration of flames and hot gases and limit the rise in temperature on the unexposed side of the construction during a complete compartment fire.

An analytical method for a determination of the temperature-time field in a multi-layer partition is presented in [18]; cf. also [4]. The method considers the temperature dependence of the thermal material properties, an initial moisture content, and a possible material disintegration at specified temperature criteria. An illustrating application of the method is shown in Fig. 12 [18], which gives a summary conception

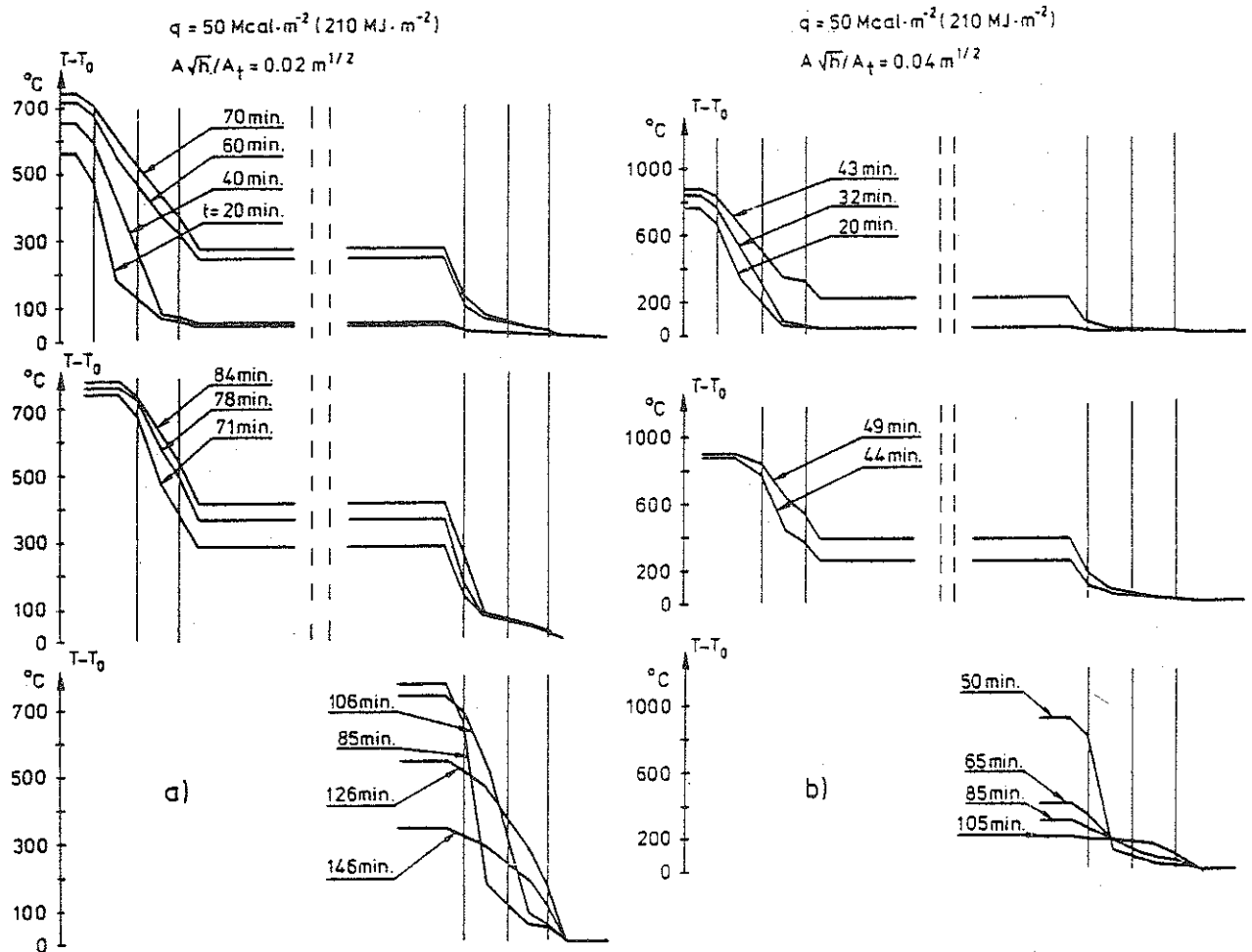


Figure 12. Calculated temperature-time fields for a steel stud wall, insulated on each side with two 13 mm gypsum plaster sheets, type Gyproc, of density $790 \text{ kg}\cdot\text{m}^{-3}$. The wall is fire exposed on one side with compartment fire characteristics according to Fig. 6: a) $q = 50 \text{ Mcal}\cdot\text{m}^{-2}$ ($210 \text{ MJ}\cdot\text{m}^{-2}$), $A\sqrt{h}/A_t = 0.02 \text{ m}^{1/2}$; b) $q = 50 \text{ Mcal}\cdot\text{m}^{-2}$ ($210 \text{ MJ}\cdot\text{m}^{-2}$), $A\sqrt{h}/A_t = 0.04 \text{ m}^{1/2}$. T_0 = temperature at time $t = 0$ [18]

of the fire behaviour of a steel stud wall, insulated on each side with two 13 mm gypsum plaster sheets, type Gyproc, of density $790 \text{ kg}\cdot\text{m}^{-3}$, fire exposed on one side and acting as a partition. The behaviour has been determined on the basis of temperature dependent thermal properties of gypsum plaster material according to Fig. 9 and a critical failure temperature for a gypsum plaster sheet of 550°C on that side of the sheet facing away from the fire. The results of full scale fire tests confirm this failure criterion.

Fig. 12a describes the fire behaviour of the wall, when it is fire exposed on one side by a compartment fire with gas temperature-time

characteristics according to Fig. 6 - fire load density $q = 50 \text{ Mca} \cdot \text{m}^{-2}$ ($210 \text{ MJ} \cdot \text{m}^{-2}$), opening factor $A\sqrt{h}/A_t = 0.02 \text{ m}^{1/2}$. The figure gives a calculated failure of the directly fire exposed gypsum plaster sheet after about 70 min and of the next gypsum plaster sheet after about 85 min. The maximum temperature rise on the unexposed side of the wall amounts to 180°C during the complete fire process, i.e. precisely the maximum permissible value according to [2]. Fig. 12b analogously describes the fire behaviour of the wall, when it is exposed to a more rapid compartment fire - opening factor $A\sqrt{h}/A_t = 0.04 \text{ m}^{1/2}$ - at the same fire load density q . The increase of the opening factor results in a considerably decreased value of the maximum temperature rise on the unexposed side of the wall, which amounts to only about 55°C in this case.

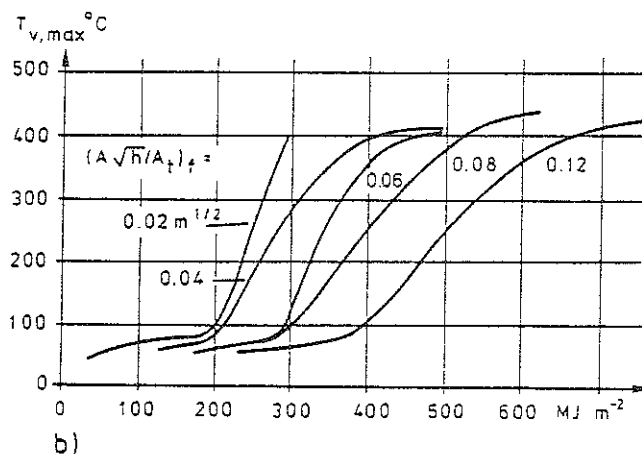
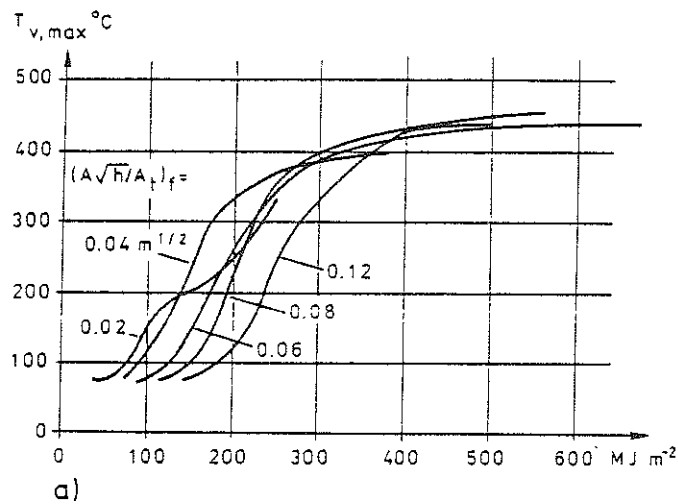


Figure 13. Maximum temperature $T_{v,max}$ during a complete fire process according to Fig. 6 on the unexposed side of a steel-gypsum plaster sheeting wall as a function of the fictitious fire load density q_f and the fictitious opening factor $(A\sqrt{h}/A_t)_f$ of the fire compartment. The wall is insulated on each side with one (fig a) or two (fig b) 13 mm gypsum plaster sheets, type Gyproc, of density $790 \text{ kg} \cdot \text{m}^{-3}$ [4], [6]

Systematic calculations of the type, illustrated by Fig. 12, lead to design diagrams as shown in Fig. 13 [4], [6], giving the maximum temperature $T_{v,max}$ during a complete fire process on the unexposed side of a steel stud-gypsum plaster sheeting wall as a function of the fictitious fire load density q_f and the fictitious opening factor of the fire compartment $(A\sqrt{h}/A_t)_f$. The two diagrams apply to an insulation on each side of the wall with one and two 13 mm gypsum plaster sheets, type Gyproc, of density $790 \text{ kg}\cdot\text{m}^{-3}$, respectively. The calculated $T_{v,max}$ values are to be compared with the corresponding maximum temperature, permitted in the Swedish Building Code, which implies 200°C as an average temperature and 240°C as a temperature over limited areas of the unexposed side of the partition [2].

4. Design Load-Bearing Capacity of Fire Exposed Steel Structures

By applying the design tables 2 to 5, the maximum steel temperature $T_{s,max}$ can be determined comparatively quickly for an uninsulated or insulated steel structure, exposed to a complete compartment fire with gas temperature-time characteristics according to Fig. 6. The corresponding design load-bearing capacity of the structure then is obtained by design diagrams of the type exemplified in Fig. 14, 15 and 16.

Fig. 14 and 15 [4], [6] give the design load-bearing capacity (M_{cr} , P_{cr} , q_{cr}) of fire exposed beams of constant I cross section at different types of loading and support conditions, as a function of the steel beam temperature T_s . The design curves in Fig. 14 apply to a slow rate of heating - assumed to be $4^\circ\text{C}\cdot\text{min}^{-1}$, followed by a cooling with a rate of $1.33^\circ\text{C}\cdot\text{min}^{-1}$ - and Fig. 15 gives the correction $\Delta\beta$ of the load-bearing capacity coefficient β due to a more rapid rate of heating. In the formulas for the load-bearing capacity

σ_s = yield stress of steel material at room temperature (MPa),

L = span of beam (m),

W = elastic modulus of beam cross section (m^3).

The design curves in Fig. 14 and 15 have been determined on the basis of the deformation curve of the fire exposed beams calculated by an

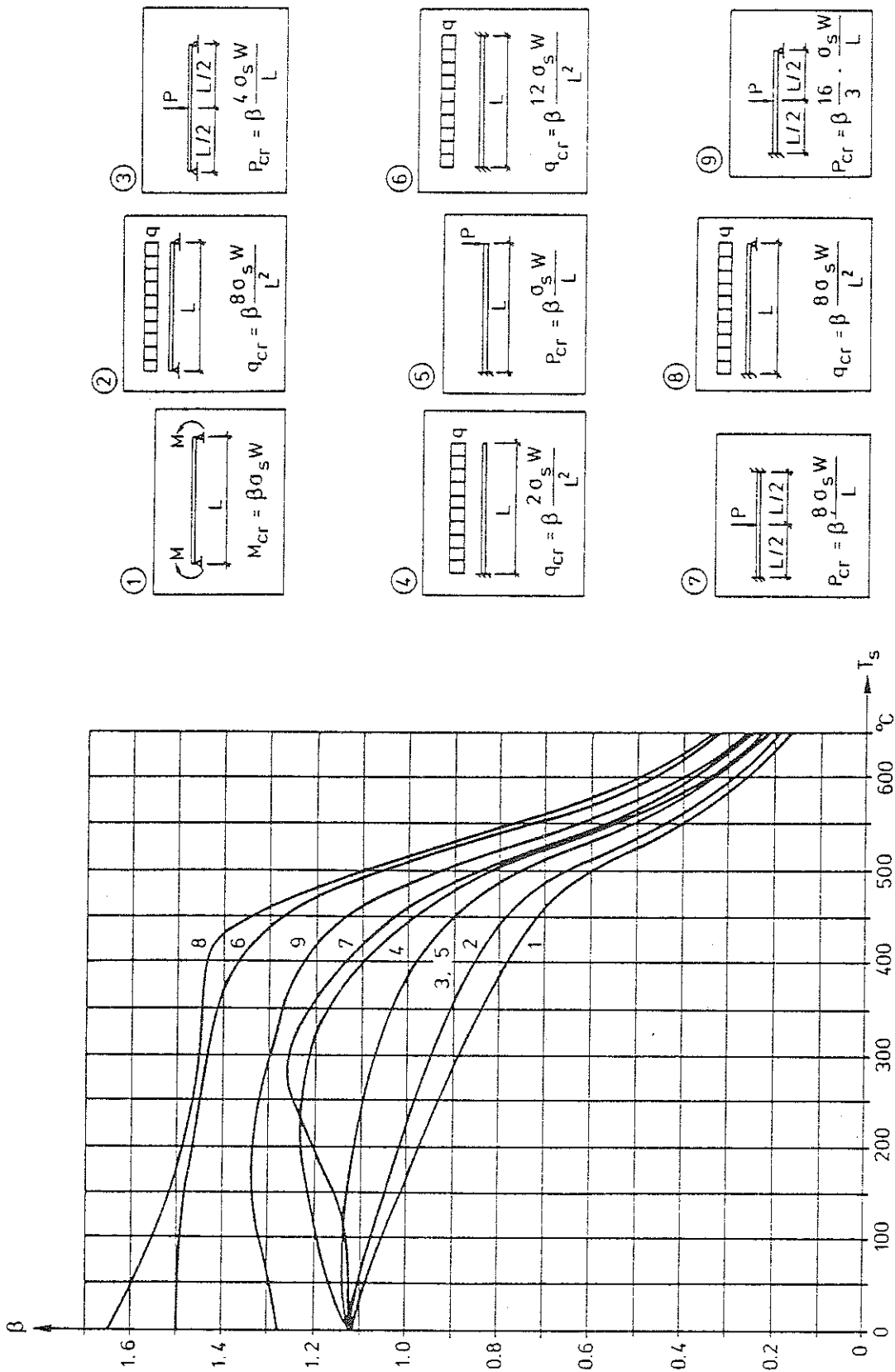


Figure 14. Coefficient β for determination of critical load (M_{cr} , P_{cr} , q_{cr}) for fire exposed beams of I cross section at different types of loading and support conditions, as a function of the steel beam temperature T_s . The curves have been calculated for a slow rate of heating of $4 \text{ }^{\circ}\text{C}\cdot\text{min}^{-1}$ and a subsequent cooling, assumed to be one third of the rate of heating [4], [6]

analytical model, presented in [22], which takes into account the softly rounded shape of the stress-strain curve of steel at elevated temperatures as well as the influence of creep strain. As can be seen from Fig. 15, this influence of creep begins to be noticeable for ordinary structural steels at temperatures in excess of about 450°C. The load-bearing capacity of the beams is defined by the limit deflection criterion according to ROBERTSON and RYAN [23].

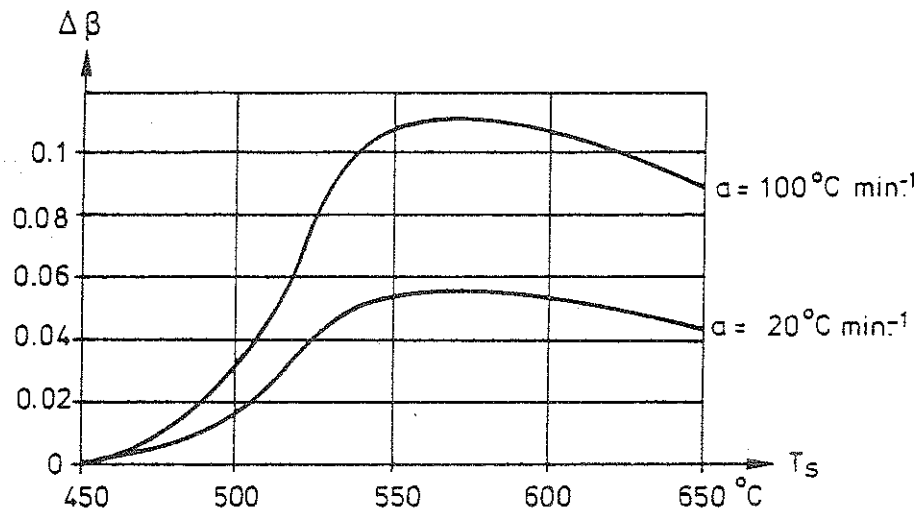


Figure 15. Increase $\Delta\beta$ of coefficient β , determined according to Fig. 14, for a rate of heating $\alpha \geq 4^\circ\text{C}\cdot\text{min}^{-1}$, as a function of the steel beam temperature T_s [4], [6]

The diagrams in Fig. 16 [4] determine the variation with the steel temperature T_s of the relationship between the buckling stress σ_{cr} and the slenderness ratio λ for fire exposed columns, axially loaded in compression. The diagrams apply to steel having a yield stress at room temperature $\sigma_s = 220, 260$ and 320 MPa, respectively, and are valid under the presumption that the column is unrestrained with respect to longitudinal expansion during the fire exposure. The σ_{cr} - λ curves have been computed for an initially deflected and excentrically loaded column on the basis of data on the change of the 0.5 % proof stress $\sigma_{0.5}$ and the secant modulus with the temperature, obtained in tension tests at a very slow rate of loading. This implies that a considerable influence of short-time creep at elevated temperatures is included.

For a fire engineering design of columns, partly restrained to a longitudinal expansion, reference is made to [4].

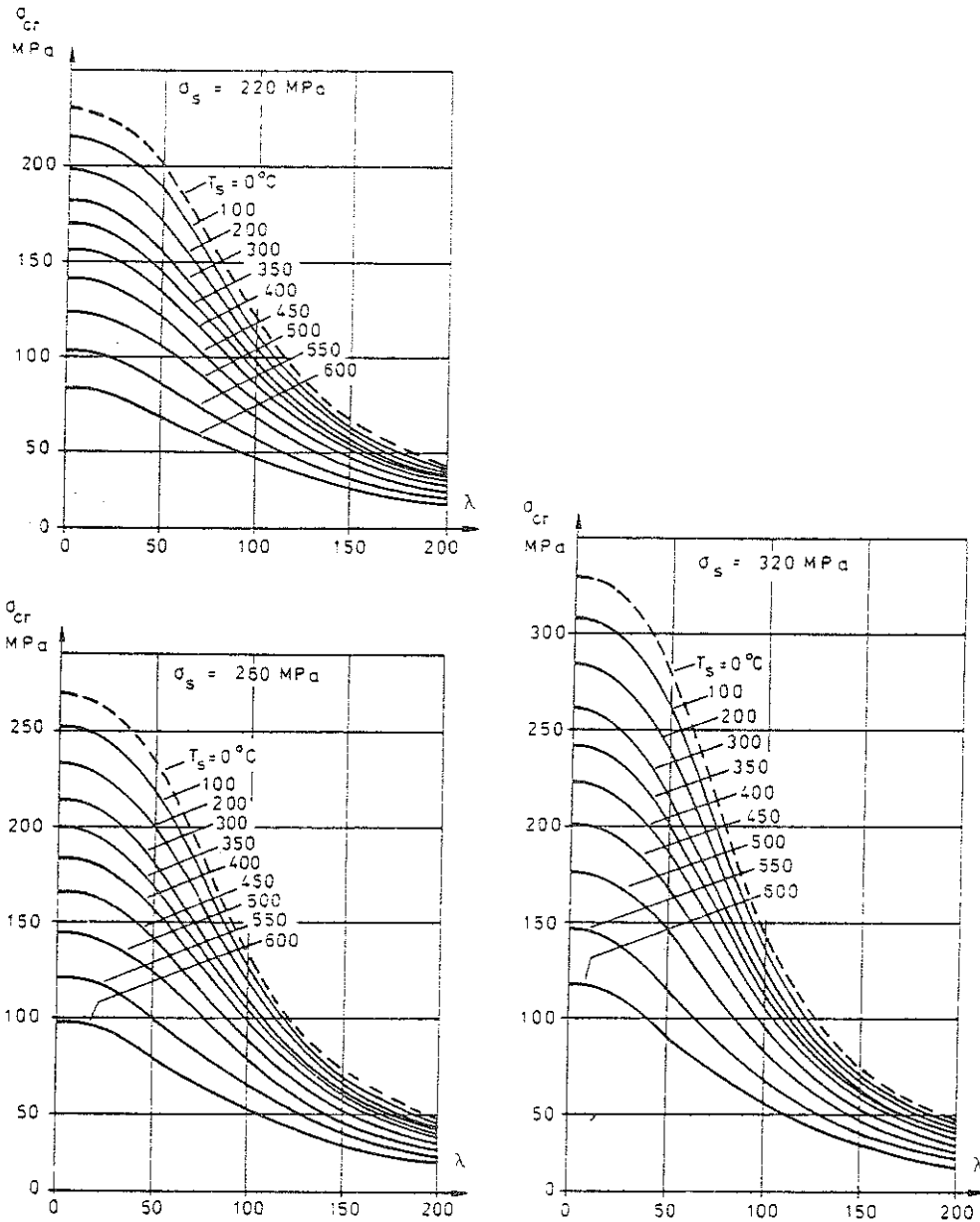


Figure 16. Variation with steel temperature T_s of the relationship between buckling stress σ_{cr} and slenderness ratio λ for fire exposed steel columns, axially loaded in compression, free to expand longitudinally and made of steel having a yield stress at room temperature $\sigma_s = 220, 260$ and 320 MPa, respectively [4], [6]

The design curves, reproduced in Fig. 14, 15 and 16, are generally based on the assumption of a uniformly distributed temperature over the cross section of the steel structure at any time t during the fire exposure. By this assumption, the design curves are directly connected to Tables 2 to 5, determining the design temperature state of the steel structure.

If the analytical, differentiated design of fire exposed steel structures will be further developed in future towards a more accurate determination

of the design temperature state, with regard taken to the temperature variation over the cross section of the steel structure, this will also require a more refined basis of design for the transfer of the design temperature state to the design load-bearing capacity of the fire exposed structure. The first attempts of developing such a more refined design basis now can be noticed in the literature. As a fragmentary example of this development, Fig. 17 [24] shows the calculated variation of the plastic bending moment of a fire exposed steel I cross section as a function of the maximum temperature for various linear temperature distributions over the cross section.

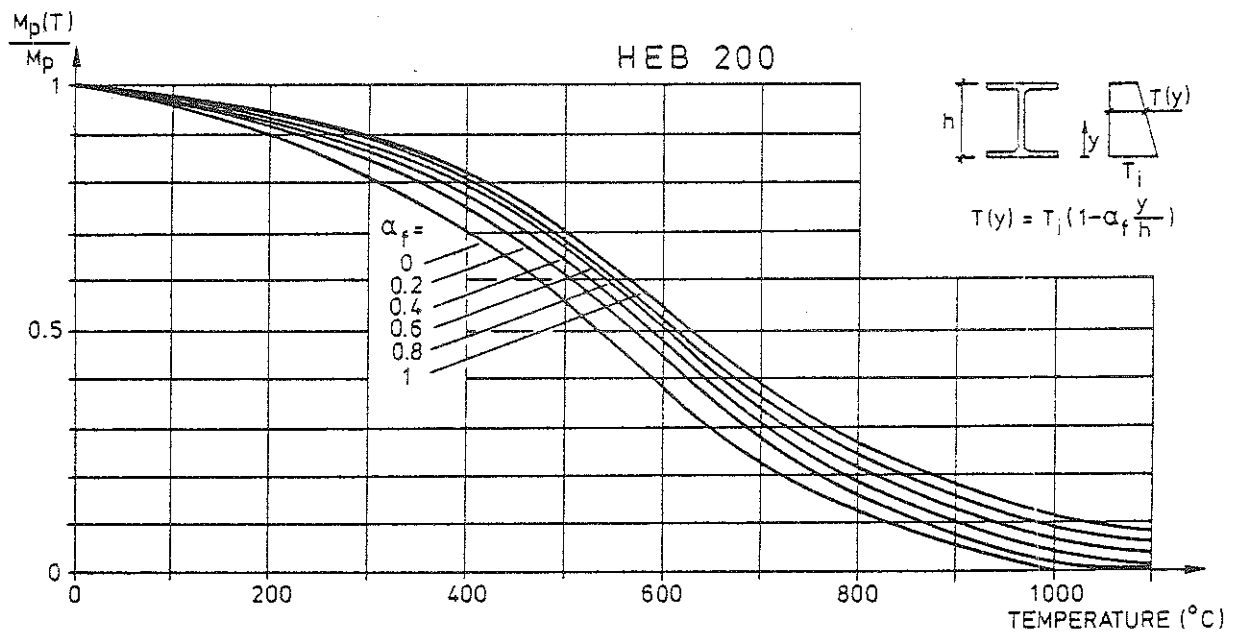


Figure 17. Calculated variation of plastic bending moment $M_p(T)$ in terms of various linear temperature distribution over height of a steel I cross section [24]

5. Summary

A differentiated procedure is presented for an analytical fire engineering design of load-bearing steel structures and partitions. The procedure is a direct design method based on gas temperature-time characteristics of a complete compartment fire, which depend on the fire load density, the ventilation of the fire compartment and the thermal properties of the structures enclosing the fire compartment. The practical use of the design procedure has been approved by the National Swedish Board of Physical Planning and Building.

For the practical application of the design procedure, a comprehensive design basis in the form of diagrams and tables has been worked out for a direct determination of the maximum steel temperature during a complete compartment fire and the corresponding design load-bearing capacity of the fire exposed structure. This design basis is exemplified in the paper, focused to steel structures with an insulation of gypsum plaster slabs, primarily for giving a rough impression of the character of the analytical design procedure.

Compared with the conventional fire engineering design, based on classification and results of standard fire resistance tests, the presented analytical design procedure has a more logical structure, based on well-defined functional requirements and performance criteria, gives a structural fire design with a better economy, and leads to a more consistent fire safety level.

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APPENDIX

Table 1. Coefficient K_f for transforming a real fire load density q and a real opening factor of a fire compartment $A\sqrt{h}/A_t$ to a fictitious fire load density q_f and a fictitious opening factor $(A\sqrt{h}/A_t)_f$ corresponding to a fire compartment, type A

$$q_f = K_f q \quad (A\sqrt{h}/A_t)_f = K_f A\sqrt{h}/A_t$$

Type of fire compartment	Opening factor $A\sqrt{h}/A_t$ $m^{1/2}$					
	0.02	0.04	0.06	0.08	0.10	0.12
Type A	1	1	1	1	1	1
Type B	0.85	0.85	0.85	0.85	0.85	0.85
Type C	3.00	3.00	3.00	3.00	3.00	2.50
Type D	1.35	1.35	1.35	1.50	1.55	1.65
Type E	1.65	1.50	1.35	1.50	1.75	2.00
Type F ¹⁾	1.00-	1.00-	0.80-	0.70-	0.70-	0.70-
	0.50	0.50	0.50	0.50	0.50	0.50
Type G	1.50	1.45	1.35	1.25	1.15	1.05

¹⁾ The lowest value of K_f applies to a fire load density $q > 500 \text{ MJ}\cdot\text{m}^{-2}$, the highest value to a fire load density $q \leq 60 \text{ MJ}\cdot\text{m}^{-2}$. For intermediate fire load densities, linear interpolation gives sufficient accuracy.

The different types of fire compartment are defined as follows

Fire compartment, type B: Bounding structures of concrete.

Fire compartment, type C: Bounding structures of lightweight concrete (density $\rho = 500 \text{ kg}\cdot\text{m}^{-3}$).

Fire compartment, type D: 50% of the bounding structures of concrete, and 50% lightweight concrete (density $\rho = 500 \text{ kg}\cdot\text{m}^{-3}$).

Fire compartment, type E: Bounding structures with the following percentage of bounding surface area:

50% lightweight concrete (density $\rho = 500 \text{ kg}\cdot\text{m}^{-3}$),
33% concrete,

17% of from the interior to the exterior: plasterboard panel (density $\rho = 790 \text{ kg}\cdot\text{m}^{-3}$), 13 mm in thickness - diabase wool (density $\rho = 50 \text{ kg}\cdot\text{m}^{-3}$), 10 cm in thickness - brickwork (density $\rho = 1800 \text{ kg}\cdot\text{m}^{-3}$), 20 cm in thickness.

Fire compartment, type F: 80% of the bounding structures of sheet steel, and 20% of concrete. The compartment corresponds to a storage space with a sheet steel roof, sheet steel walls, and a concrete floor.

Fire compartment, type G: Bounding structures with the following percentage of bounding surface area:

20% concrete,

80% of from the interior to the exterior: double plasterboard panel (density $\rho=790 \text{ kg}\cdot\text{m}^{-3}$), 2x13 mm in thickness - air space, 10 cm in thickness - double plasterboard panel (density $\rho = 790 \text{ kg}\cdot\text{m}^{-3}$), 2x13 mm in thickness.

For fire compartments, not directly represented in the table, the coefficient K_f can either be determined by a linear interpolation between applicable types of fire compartment in the table or be chosen in such a way as to give results on the safe side. For fire compartments with surrounding structures of both concrete and lightweight concrete, then different values can be obtained of the coefficient K_f , depending on the choice between the fire compartment types B, C, and D at the interpolation. This is due to the fact that the relationships, determining K_f , are non-linear. However, the K_f -values of the table are such that a linear interpolation always gives results on the safe side, irrespective of the alternative of interpolation chosen. In order to avoid an unnecessarily large overestimation of K_f , that alternative of interpolation is recommended which gives the lowest value of K_f .

Table 3. Maximum steel temperature $T_{s,max}$ ($^{\circ}C$) for insulated steel structure as a function of fictitious fire load density q ($Mcal \cdot m^{-2}$) $\{MJ \cdot m^{-2}\}$, fictitious opening factor AVh/A_t ($m^{1/2}$), structural parameter A_j/V_s (m^{-1}), and insulation parameter d_i/λ_i ($m^2 \cdot ^{\circ}C \cdot h \cdot kcal^{-1}$)^a. d_i denotes insulation thickness (m) [4]

Table with 16 columns and multiple rows. Columns include parameters AVh/A_t, A_j/V_s, insulation parameters d_i/lambda_i, and temperature T_s,max. Rows are grouped by q values (15, 20, 25, 30, 35, 40, 45) and AVh/A_t values (0.01, 0.02, 0.04, 0.06, 0.08, 0.12). Includes a note at the bottom right: 'a [0,05 m^2 Ch/kcal = 0,043 m^2 C/W]'

Table 4. Maximum steel temperature $T_{s,max}$ ($^{\circ}C$) for a steel structure insulated with gypsum plaster slabs, type Gyproc ($\rho_j = 790 \text{ kg}\cdot\text{m}^{-3}$), as a function of fictitious fire load density q ($\text{Mcal}\cdot\text{m}^{-2}$) ($\text{MJ}\cdot\text{m}^{-2}$), fictitious opening factor $A\sqrt{h}/A_t$ ($\text{m}^{1/2}$), structural parameter A_j/V_s (m^{-1}), and insulation thickness d_j (mm) [4]

q	$A\sqrt{h}/A_t$	A_j/V_s	$T_{s,max}$			q	$A\sqrt{h}/A_t$	A_j/V_s	$T_{s,max}$			q	$A\sqrt{h}/A_t$	A_j/V_s	$T_{s,max}$			q	$A\sqrt{h}/A_t$	A_j/V_s	$T_{s,max}$			
			d_j 13	d_j 26	d_j 26				d_j 13	d_j 26	d_j 13				d_j 26	d_j 13	d_j 26				d_j 13	d_j 26		
15 [63]	0,01	125 150 200 300 400	315	200	150	0,01	25 50 75 100	315 415 465 495	210	305	195	0,02	125 150 200 300	300 325 350 405	250	370	250	0,04	25 30 75	390 550 635	250	395	350	
			215	305	200				305	435	290				250	370	250				350	620	535	
			235	335	235				360	525	345				500	635	500				630	735	630	
			260	395	260				395	600	390				570	725	570				465	815	705	
	0,02	125 150 200 300 400	300 325 350 405	150	150	150	0,02	125 150 200 300	510 525 535 550	420	425	425	0,02	125 150 200 300	640 675 700 720	425	500	425	0,04	100 125 150	725 755 765	425	510	465
				165	165	165				440	500	440				425	500	425				475	590	475
				200	200	200				465	565	465				425	500	425				530	775	530
				215	215	215				495	640	495				425	500	425				630	815	630
	0,04	300 400	330 320	230	230	230	0,04	50 75 100	350 410 460	215	265	215	0,04	125 150 200	560 595 605	215	275	215	0,08	125 150	630 645	215	325	215
				250	250	250				265	300	265				215	275	215				275	410	275
				265	265	265				265	325	265				215	275	215				300	460	300
				275	275	275				265	350	265				215	275	215				350	530	350
20 [84]	0,01	75 100 125 150 200 300 400	345	230	180	0,02	150 200 300 400	495 520 535 550	185	265	185	0,04	150 200 300 400	605 630 640	185	265	185	0,08	150 200 300 400	695 730 745	185	320	185	
			260	260	260				215	300	215				185	265	185				215	300	215	
			280	280	280				245	345	245				185	265	185				245	345	245	
			300	300	300				260	395	260				185	265	185				260	395	260	
	0,02	75 100 125 150 200 300 400	300 325 350 405	180	180	180	0,02	150 200 300 400	445 470 480	185	260	185	0,02	150 200 300 400	535 560 570	185	260	185	0,04	150 200 300 400	635 670 685	185	320	185
				200	200	200				215	300	215				185	260	185				215	300	215
				235	235	235				245	345	245				185	260	185				245	345	245
				265	265	265				260	395	260				185	260	185				260	395	260
	0,04	300 400	490 520	320	320	320	0,04	150 200 300 400	560 595 605	320	320	320	0,04	150 200 300 400	640 670	320	320	320	0,08	150 200 300 400	735 770	320	390	320
				330	330	330				350	420	350				320	320	320				350	420	350
				350	350	350				375	470	375				320	320	320				375	470	375
				375	375	375				400	520	400				320	320	320				400	520	400
25 [105]	0,01	50 75 100 125 150 200 300 400	360	250	200	0,02	100 125 150 200 300 400	500 520 535	110	150	110	0,04	100 125 150 200 300 400	595 620 630	110	150	110	0,08	100 125 150 200 300 400	695 730	110	200	110	
			300	300	300				120	170	120				110	150	110				120	170	120	
			335	335	335				125	175	125				110	150	110				125	175	125	
			360	360	360				130	180	130				110	150	110				130	180	130	
	0,02	50 75 100 125 150 200 300 400	300 325 350 405	140	140	140	0,02	100 125 150 200 300 400	445 470 480	115	150	115	0,02	100 125 150 200 300 400	535 560 570	115	150	115	0,04	100 125 150 200 300 400	635 670	115	200	115
				160	160	160				125	175	125				115	150	115				125	175	125
				180	180	180				130	180	130				115	150	115				130	180	130
				200	200	200				135	185	135				115	150	115				135	185	135
	0,04	300 400	490 520	320	320	320	0,04	100 125 150 200 300 400	560 595 605	320	320	320	0,04	100 125 150 200 300 400	640 670	320	320	320	0,08	100 125 150 200 300 400	735 770	320	390	320
				330	330	330				350	420	350				320	320	320				350	420	350
				350	350	350				375	470	375				320	320	320				375	470	375
				375	375	375				400	520	400				320	320	320				400	520	400

Table 5. Maximum steel beam temperature $T_{s,max}$ ($^{\circ}\text{C}$) for a steel beam construction according to Fig. 10, with an insulation in the form of a suspended ceiling, as a function of fictitious fire load density q ($\text{Mcal}\cdot\text{m}^{-2}$) $\{\text{MJ}\cdot\text{m}^{-2}\}$, fictitious opening factor $A\sqrt{h}/A_t$ ($\text{m}^{1/2}$), structural parameter F_s/V_s (m^{-1}), and insulation parameter d_i/λ_i ($\text{m}^2\cdot^{\circ}\text{C}\cdot\text{h}\cdot\text{kcal}^{-1}$)^c. The maximum temperature in the suspended ceiling is given in brackets [4]

q	$\frac{A\sqrt{h}}{A_t}$	$\frac{F_s}{V_s}$	Maximum steel temperature $T_{s,max}$ and () maximum suspended ceiling temperature				q	$\frac{A\sqrt{h}}{A_t}$	$\frac{F_s}{V_s}$	Maximum steel temperature $T_{s,max}$ and () maximum suspended ceiling temperature											
			$(d_i/\lambda_i)_{fict}$							$(d_i/\lambda_i)_{fict}$											
			0,05	0,10	0,20	0,30				0,05	0,10	0,20	0,30								
15 {63}	0,02	50	130	90	65	50	60	0,02	50	435	315	200	160								
		100	180	(470)	130	(440)			90	(410)	70	(390)	200	455	(615)	350	(570)	250	(530)	200	(500)
		200	230		170				115		90		300	455		350		250		200	
		300	260		190				130		100		50	340		225		145		110	
	0,04	50	100	70	45	40	250	0,04	50	340	225	145	110								
		100	150	(565)	100	(530)			65	(500)	50	(475)	200	435	(680)	320	(630)	185	(590)	140	(560)
		200	200		140				90		70		300	445		330		230		180	
		300	240		170				110		80		50	250		160		100		75	
	0,08	50	65	50	35	25	60	0,08	50	250	160	100	75								
		100	95	70	50	40			200	415	(750)	285	(700)	185	(650)	135	(625)				
		200	150	(675)	100	(630)			65	(590)	50	(570)	300	445		315		210		155	
		300	190		125				90		60		50	190	(780)	120	(725)	75	(680)	60	(660)
0,12	50	40	35	30	25	60	0,12	50	190	120	75	60									
	100	60	45	40	30			200	285	(740)	185	(725)	110	(680)	80	(660)					
	200	120	(735)	70	50			300	375		250		155		110						
	300	155		100				60		45		50	420		290		185		130		
25 {105}	0,02	50	200	140	95	75	90	0,04	50	475	330	205	150								
		100	260	(510)	185	(470)			125	(435)	100	(420)	200	515	(740)	370	(680)	250	(630)	190	(600)
		200	300		225				155		120		300	515		385		270		210	
		300	320		245				170		130		50	345		225		130		100	
	0,04	50	160	110	75	55	380	0,08	50	345	225	130	100								
		100	230	(600)	150	(565)			100	(530)	75	(515)	200	430	(790)	290	(730)	180	(675)	130	(650)
		200	290		205				135		100		300	495		360		250		190	
		300	325		235				155		115		50	560		400		260		200	
	0,08	50	115	75	50	40	120	0,04	50	570	420	290	220								
		100	160	110	70	55			200	575	(780)	425	(715)	300	(660)	230	(630)				
		200	240	(680)	160	(635)			100	(595)	75	(570)	300	575		425		300		230	
		300	285		195				120		90		50	425		280		160		120	
0,12	50	80	60	40	30	500	0,08	50	425	280	160	120									
	100	130	80	60	45			200	495	(810)	345	(750)	210	(695)	160	(670)					
	200	190	(740)	125	(690)			80	(650)	60	(620)	300	525		385		260		205		
	300	235		160				100		75											
40 {168}	0,02	50	300	220	145	110	0,02	50	300	220	145	110									
		100	360	(560)	260	(520)		175	(480)	135	(460)	200	380	(560)	290	(520)	200	(480)	160	(460)	
		200	380		290			200		160		300	385		295		210		165		
		300	385		295			210		165		50	240		160		105		80		
	0,04	50	240	160	105	80	0,04	50	315	(645)	220	(600)	140	(560)	100	(535)					
		100	315		220			140		135		200	375		270		180		135		
		200	375		270			180		135		300	390		290		195		150		
		300	390		290			195		150		50	170		110		70		55		
	0,08	50	170	110	70	55	0,08	50	170	110	70	55									
		100	245	(715)	160	(665)		100	(625)	75	(600)	200	335		220		140		105		
		200	335		220			140		105		300	380		260		165		120		
		300	380		260			165		120		50	130		85		55		45		
0,12	50	130	85	55	45	0,12	50	130	85	55	45										
	100	200	(750)	130	(700)		85	(660)	60	(630)	200	290		190		115		85			
	200	290		190			115		85		300	340		225		145		100			
	300	340		225			145		100												

^c $\left\{ \begin{array}{l} 0,05 \text{ m}^2 \cdot ^{\circ}\text{C}\cdot\text{h}/\text{kcal} = 0,043 \text{ m}^2 \cdot ^{\circ}\text{C}/\text{W} \\ 0,10 \text{ } \gg \gg = 0,086 \text{ } \gg \gg \\ 0,20 \text{ } \gg \gg = 0,172 \text{ } \gg \gg \\ 0,30 \text{ } \gg \gg = 0,258 \text{ } \gg \gg \end{array} \right\}$

Table 6. Summary results of standard fire resistance tests on some types of suspended ceilings and connected values, derived from the test results, for $(d_i/\lambda_i)_{fict}$ and critical temperature of the ceilings [4]

No	Make	Material	Resistance time in standard fire test (min)	Remarks	Estimated $(d_i/\lambda_i)_{fict}$		Estimated critical suspended ceiling temperature (°C)
					$\left(\frac{m^2 \cdot ^\circ C \cdot h}{kcal}\right)$	$\left(\frac{m^2 \cdot ^\circ C}{W}\right)$	
1	Gyproc	2x13 mm gypsum plaster slabs no glass fibre reinforcement	30-40	All tests were discontinued because the suspended ceiling fell down. The critical temperature had not been reached in the steel girders	0,075	0,064	625
2		1x13 mm gypsum plaster slabs 0.25% g f r	48		0,075	0,064	650
3		1x16 mm gypsum plaster slabs 0.25% g f r	49		0,10	0,086	650
4		2x13 mm gypsum plaster slabs 0.25% g f r	60		0,15	0,129	650
5		3x13mm gypsum plaster slabs 0.25% g f r	75-80		0,25	0,215	625
6		2x20 mm gypsum plaster slabs 0.25% g f r	80		0,30	0,258	625
7	WST	2x13 mm gypsum plaster slabs with 13 mm mineral wool between them	45	All tests were discontinued for the same reason as above. The gypsum plaster slabs were not reinforced	0,30	0,258	550
8		2x13 mm gypsum plaster slabs with 13 mm mineral wool between them	50		0,30	0,258	550
9		2x13 mm gypsum plaster slabs with 43 mm straw between them	47		0,30	0,258	550
10		2x13mm gypsum plaster slabs with 43 mm straw between them	54		0,30	0,258	550
11	Ingenjör-firma Zero	Soundex special suspended ceiling tiles. Cast glass fibre reinforced gypsum plaster tiles with "ridges" in a grid pattern. Tile thickness 18 mm, at the ridges 38 mm	90	Parts of the ceiling fell down after 90 minutes. Max. steel temperature approx. 440°C	0,15	0,129	700
12	Consensus	Armstrong 13 mm thick	30	No visible damage to suspended ceiling. Max steel temperature about 450 °C	0,05	0,043	550
13		Mineral wool acoustic 16 mm thick	80		0,075	0,064	>(725) ^a
14		Type minaboard 13 mm thick	85	No visible damage to suspended ceiling. Max steel temperature about 300 °C	0,075	0,064	>(725) ^a
15	Dansk Eternitfabrik	Deflamit-Asbestolux (9 mm Deflamit + 15 mm mineral wool + 9 mm eternit)	90		0,20	0,172	>(679) ^a
16	Nordakustik	Celotex Acoustiformat 15 mm thick glass fibre slab	90	No visible damage to suspended ceiling. Max steel temperature about 450 °C. The test was discontinued because the suspended ceiling fell down. The critical temperature had not been reached in the steel girders.	0,10	0,086	(725) ^a
17	Rockwool	Rockfon Decor 85l (15 mm thick mineral wool slab)	60		0,20	0,172	600

^a No damage to the suspended ceiling. Calculated temperature in the suspended ceiling when the test was discontinued.