NBS TECHNICAL NOTE 710-10

U.S. DEPARTMENT OF COMMERCE / National Bureau of Standards

BUILDING RESEARCH TRANSLATION

The Behavior of Concrete Structures in Fire—A Method for Prediction by Calculation

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BUILDING RESEARCH TRANSLATION
The Behavior of Concrete Structures in Fire—A Method for Prediction by Calculation

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Preface

The United States and Frances' Cooperative Program on Building Technology involves an exchange of personnel and information between the Center for Building Technology of the National Bureau of Standards and the Centre Scientifique et Technique du Batiment (CSTB). Information contained in this report was selected for translation and reproduction for sale to the building community by the Government Printing Office. The CSTB papers made public through this Technical Note do not necessarily represent the views of the National Bureau of Standards on either policy or technical levels. Building researchers at the National Bureau of Standards consider it a public service to share with the U. S. building community these insights into some of the research activities of CSTB.

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This document cancels and replaces "Recommendations for Predicting, by Calculation, the Behavior of Concrete Structures in Fire" issued in May 1972 and includes Supplement No. 1 of April 1975.

Introduction

This method gives a means for predicting, by calculation, the resistance to fire of a reinforced or prestressed element of construction, in accordance with the directives of 5 January 1959.

It is directly useful in allowing builders to design structures which would with normal likelihood show the degree of fire resistance required by the various construction regulations in force. According to those regulations, only a test furnishes legal proof of fire resistance.

We would hope that this first step will lead to the acceptance of calculation of fire resistance as legal proof of satisfactory resistance.

BACKGROUND NOTE BY THE RAPPORTEUR

Rationale

If fire effects are to be introduced into calculations, they must be given a simple physical definition. This work has already been done by the editors of the text defining fire test conditions, since they assumed that a fire could be simulated by a rise in temperature in a standard furnace.

We have adopted this basic hypothesis since it would give us, under the best possible conditions, the opportunity to compare the results of calculation of an element and those of the corresponding test, and then to compare the methods.

The fire study started with a study of the temperature distribution in an element when it was subjected to a heat flux on a part of its exterior surface. This concept has already been developed by numerous workers, and we know that the temperature rise follows the Fourier equation which, for plane problems, can be easily converted to an equation of finite differences. We did this by developing a calculational program on the terminal both to extract the results that can be used immediately for structures of simple shapes and also to handle more complex shapes on a case-by-case basis.

To carry out this calculation successfully it was necessary to know the "convection + radiation" coefficient of the furnace, which depends on too many parameters for a precise and usable formulation to be given. This difficulty was eliminated by giving it a probable expression dependent on adjusted parameters for finding the average "useful temperature-time-distance" laws that would give the same results as those of tests published so far.

The temperature distribution being thus defined, we could determine the first moment at which temperatures of 140 or 180°C are reached on the face of the element not exposed to the fire and then its thermal insulation in the sense of criterion No. 2.

We then studied the thermal expansion coefficients of materials which have been published by various authors. Evaluation of the complementary effects caused by fire thus becomes a problem of strength of materials, for which we can give solutions whose approximation can be evaluated.

Finally, by using the published results of tests on the weakening of the various mechanical properties of the materials as a function of temperature (nominal failure stress, elongation, modulus of elasticity, etc.) we can calculate at any instant the breaking moment of a cross-section.

Comparison, for all or part of the structure, of the associated failure load with the working load allows us to determine the point at which fire resistance is no longer assured. We were led at this phase of our study to adopt weighting coefficients for the weak stresses in the concrete and steel which we assumed in order to find the fire-resistance times given by published test results.

The Document

We then developed this document according to the above scheme, taking into account the new orientation of the regulations.

The text portions which discuss resistance to fire are drawn up in the form of results required and not as means for arriving at them.

This means that all latitude can be left to the engineer in proving out this text material. It would be unfortunate to restrict opportunities by too precise requirements, especially since the lack of perspective and experience could lead us to a poor evaluation of the means.

We decided to draw up an open text that would allow an engineer to apply, depending on the enthusiasm or interest he wanted to invest,

--Simple rules,

--Or failure calculations based on the results of given temperatures,

--Or temperature and failure calculations,

--Or any other method, tests in particular.

Thus our document contains, besides generalities:

--Various curves showing the variations in physical and mechanical properties of concrete and steel materials with temperature

--Examination of the different effects of fire

--The method of failure calculation which is proposed with the weighting coefficients adopted

--By type of structure (column, wall, slab, beams, hollow bodies)
  —The simple rules
  —Temperature results for the most current cases
  —The adaptation, proposed for the calculation method

--The method of calculating temperatures with the approximations and the list of corresponding instructions.

Comments

The reader will find that the validity of the given method is strongly affected by the reproducibility of the published experimental results, both with respect to the physical properties of the materials and to the temperature measurements. There are very large dispersions of a type such that it seems essential to us to continue the studies in these different areas. This could be done step by step by engineers who would have to solve a particular problem, or by the manufacturers who would try to find details of their product properties in order to improve the calculation justifications involved. This could be done more systematically by research laboratories whose collaboration would allow confirmation or improvement of the present method.

Andre Coin

TABLE OF CONTENTS

METHOD OF PREDICTING, BY CALCULATION, THE BEHAVIOR OF CONCRETE STRUCTURES IN FIRE

Chapter I

APPLICABILITY

This document sets forth the justifications or additional analyses needed to take into account the action of fire in reinforced or prestressed concrete structures with normal aggregates.

By extension, it could be applied to structures built with light or special aggregates, provided that one knows the physical and mechanical properties needed to use it.

Comment:

Structures (elements of construction or whole structures) should be first dimensioned and analyzed according to the rules of the trade governing their behavior in relation to other stresses.

The structures are, in other respects, assumed to conform to all appropriate safety regulations, but the present text discusses only one of the aspects of the more general problem.

Chapter II

NOMENCLATURE

The basic properties of materials are their nominal properties. When these materials are at a certain temperature θ, the properties have the subscript θ. The properties of concrete depend on its age, which is indicated by the subscript $j(j)$. The prime indicates compression (and its absence indicates tension). The presence of a dash over values indicates allowable values. The following notations are used:

In the cold	At a certain temperature		
σ'_j	$\sigma'_{j\theta}$	Concrete:	failure stress in compression
σ_j	$\sigma_{j\theta}$	Concrete:	failure stress in tension
σ_e	$\sigma_{e\theta}$	Steel:	elastic limit
ε_b	$\varepsilon_{arg\theta}$	Concrete:	strain limit
ε_a	$\varepsilon_{arg\theta}$	Steel:	strain limit
σ_{arg}	$\sigma_{arg\theta}$	Steel:	failure stress in tension
ε_{arg}	$\varepsilon_{arg\theta}$	Steel:	elongation at rupture
E_a	$E_{a\theta}$	Steel:	modulus of elasticity
$\bar{\sigma}'_{bo}$	$\bar{\sigma}'_{bo\theta}$	Concrete:	allowable stress in simple compression
$\bar{\sigma}'_b$	$\bar{\sigma}'_{b\theta}$	Concrete:	allowable compression in bending compression
$\bar{\sigma}_b$	$\bar{\sigma}_{b\theta}$	Concrete:	allowable stress in tension
$\bar{\sigma}_a$	$\bar{\sigma}_{a\theta}$	Steel:	allowable stress in tension
$\frac{\Delta l}{l}$	$\left(\frac{\Delta l}{l}\right)_\theta$	Unit extension	

The useful distance (u) is the distance from the axis of the steel to a wall subjected to the effect of the fire.

The criteria and requirements are defined by two letters and a number, which show precisely the nature of the requirement and its duration expressed in hours. We have, for example,

CF (Coupe-feu = fireproof):	1/2	1	2	3	or 4 hr
SF (fire-resistive):	1/2	1	2	3	or 4 hr
PF (Pare-flammes = flameproof):	1/2 hr				

Comment:

We recall that a nominal property is the average test value where the mean square variation is subtracted from the various results.

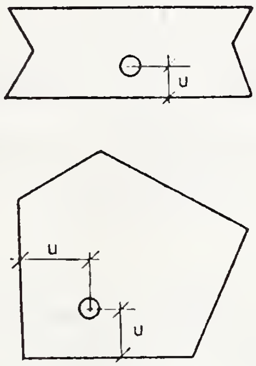


Fig. 1. Useful distance, *u*.

Reinforcing steel is also covered in accordance with the Regulations for design of reinforced or prestressed concrete in force.

The directive of the Ministry of the Interior of 5 January 1959 establishes the criteria for determining the degree of fire resistance of structural elements, the test methods, and the thermal program (normalized temperature-time curve) showing the effect of fires. The criteria are:

Criterion No. 1: mechanical strength

Criterion No. 2: thermal insulation (average heating of 140°C, maximum of 180°C for the unexposed face)

Criterion No. 3: imperviousness to flames

Criterion No. 4: no emission of flammable gas outside the exposed face.

Fire-resistant elements are classified into three categories:

--SF, fire-resistive, for which criterion No. 1 only is required.

--PF, flameproof, for which criteria 1, 3, and 4 are required.

--CF, fireproof, for which criteria 1, 2, 3, and 4 are required.

In each category the rating is expressed in terms of "degrees" as a function of the time during which the element has satisfied the tests defined in the directive.

Chapter III

CHANGES IN THE PROPERTIES OF MATERIALS AS A FUNCTION OF TEMPERATURE

The changes in the mechanical properties of materials as a function of temperature are defined by polygons whose peaks are shown in the tables below.

Instead of using these data, we can use experimental results obtained in laboratories approved for this purpose on materials actually in use. In this case it will be necessary to request and obtain the agreement of the Testing Committee for the use of the new figures proposed.

Likewise, future use of the results of new tests by the Testing Committee could lead to publication of additional sheets containing new curves to be substituted for or to supplement the older ones.

3.1 Properties of Concrete

3.11 Compression

Temperature	**0 °C**	**250 °C**	**600 °C**	**1 000 °C**
$\frac{\sigma'_{j\theta}}{\sigma'_j}$	1	1	0,45	0

Comment:

The values graphed in Figures 2-8 were taken from test results published up to this time.

Test results

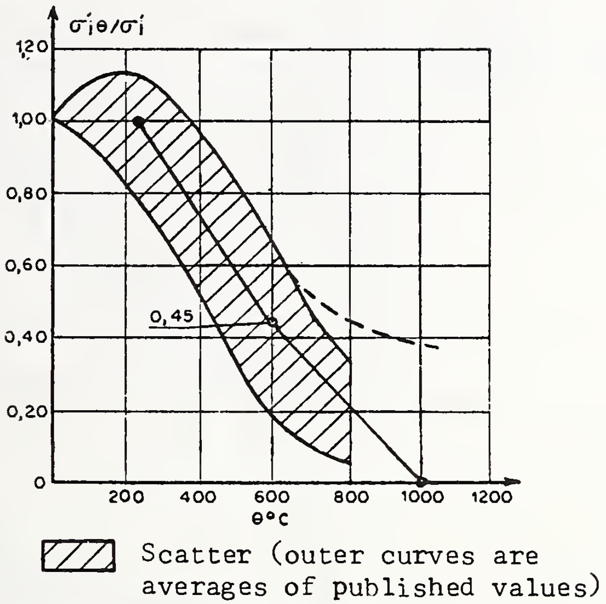


Figure 2

3.12 Tension

Temperature	0 °C	50 °C	600 °C
$\frac{\sigma_{j\theta}}{\sigma_j}$	1	1	0

Test results

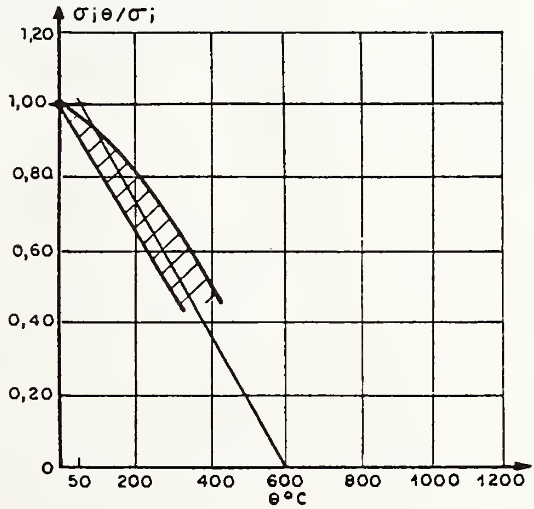


Figure 3

3.13 Expansion

The coefficient of expansion is constant, and is equal to $\frac{\Delta l}{l} = 10^{-5}$ per degree Celsius

Test results

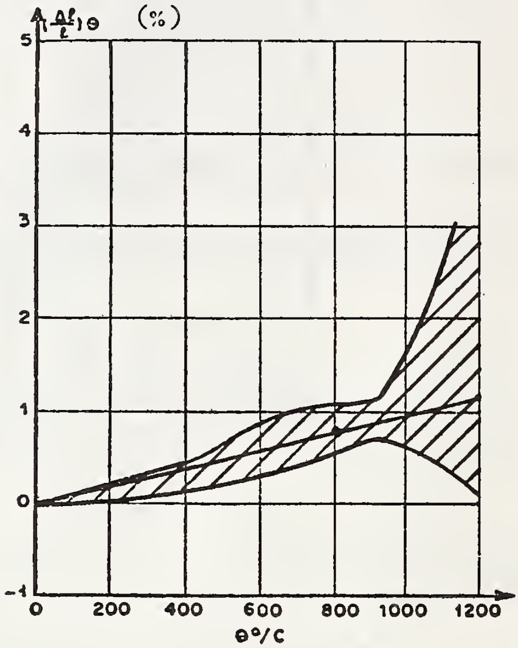


Figure 4

3.14 Modulus of Elasticity

Temperature	0 °C	50 °C	200 °C	400 °C	600 °C
$\frac{E_{b\theta}}{E_b}$	1	1	0,5	0,15	0,05

Test results

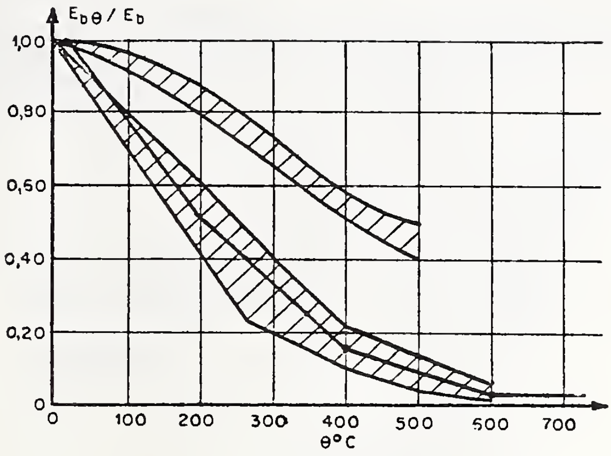


Figure 5

3.15 Conductivity

Temperature	0 °C	500 °C	1 000 °C
$\lambda_{kcal/m^2.h.°C}$	1,4	0,8	0,5

3.16 Specific Heat

At any temperature $c_{kcal/kg.\ °C} = 0.22$.

3.2 Properties of Steel

3.21 Concrete Reinforcing of steel

3.211 Failure Stress in Tension

	Type of steel	Temperature				
		0 °C	250 °C	350 °C	500 °C	800 °C
$\frac{\sigma_{arg\theta}}{\sigma_{arg}}$	Fe E22 or Fe E24 smooth round bars for reinforced concrete	1,00		1,00	0,50	0
	High adhesion Fe E40 rods, for reinforced concrete	1,00		1,00		0
	Welded mesh	1,00	1,00			0

Comment:

The fact that the values at temperature θ have the same indexes as those in the cold does not mean that the mechanical properties guaranteed in the cold will be the same at temperature θ.

3.212 Elastic Limit

Under any conditions we have:

Temperature	0 °C	250 °C	750 °C
$\sigma_e \theta / \sigma_e$	1,00	1,00	0

Test results

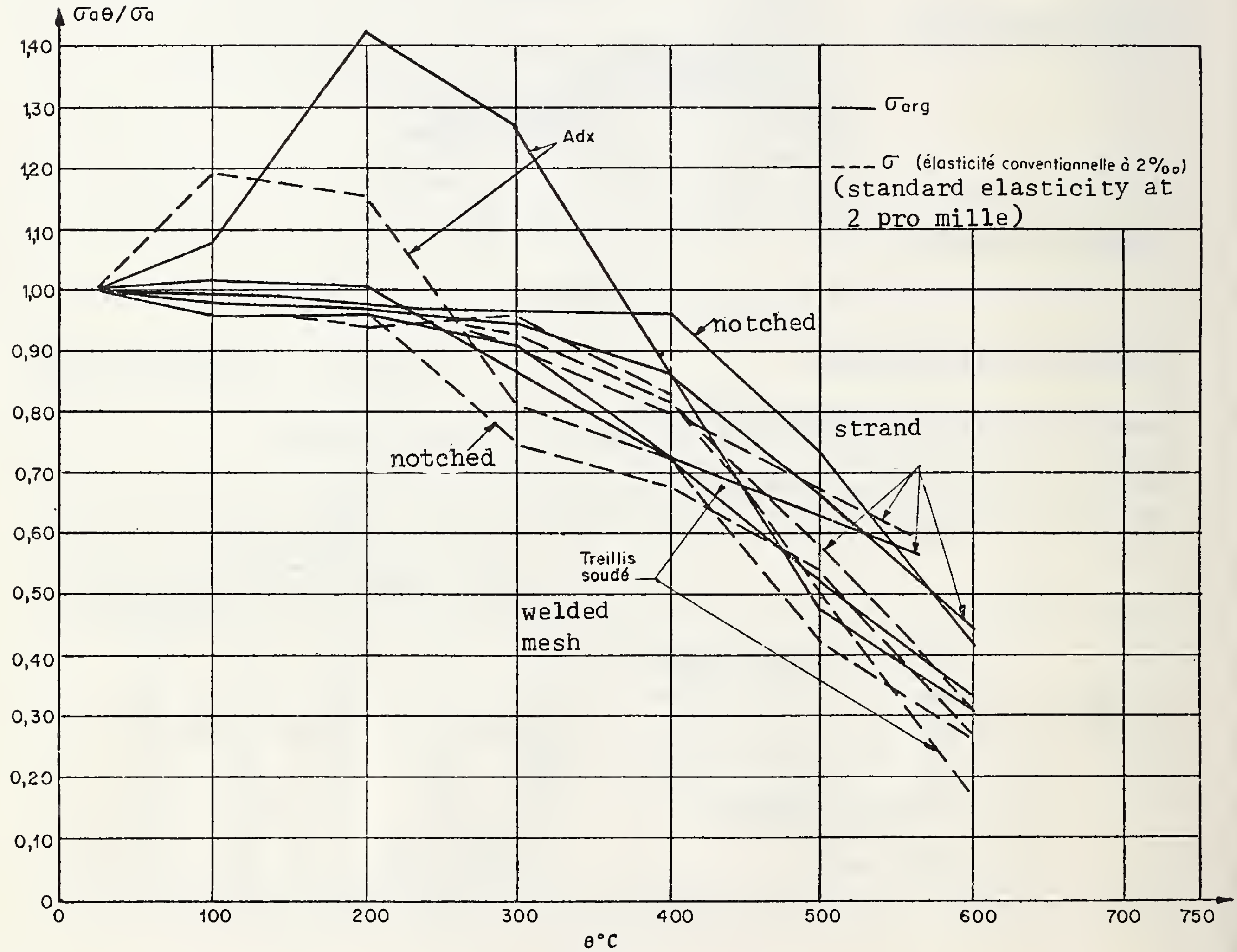


Figure 6

3.213 Elongation at Rupture

Values are the same as for elongation in the cold.

3.214 Coefficient of Expansion

The coefficient of expansion is constant, with the value:

$$\frac{\Delta l}{l} = 1.5 \times 10^{-5} \text{ per degree Celsius}$$

Test results

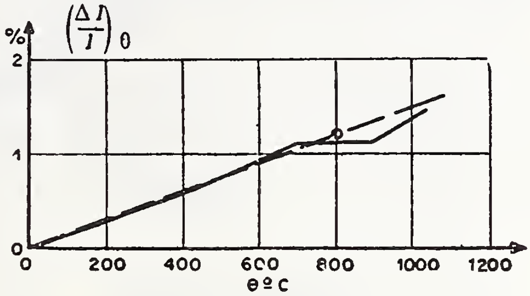


Figure 7

3.22 Hard Steels for Prestressing

3.221. Strain Limit and Standard Elasticity at 2 Pro Mille

	Temperature (°C)	0	100	150	500	750
$\frac{\sigma_e 0}{\sigma_e}$	Stabilized wire-drawn	1,00	1,00		0,20	0
and $\frac{\sigma_{arg} 0}{\sigma_{arg}}$	Laminates and strands	1,00		1,00	0,30	0

Test results

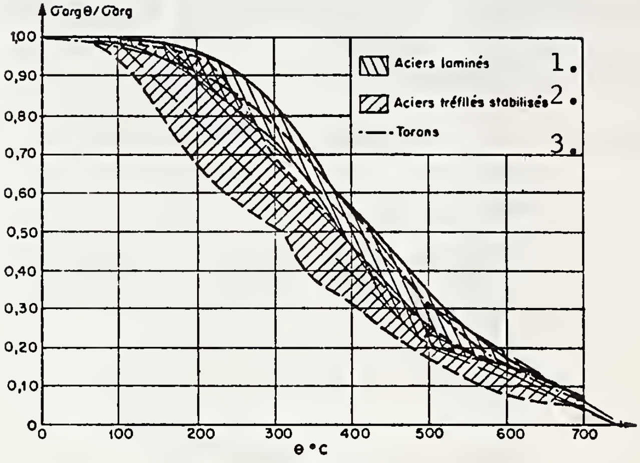


Figure 8

Key: 1. Laminated steels;
2. Stabilized wire-drawn steels;
3. Strands

3.222 Elongation at Rupture

Values are the same as for rupture in the cold.

3.223 Expansion

The coefficient of expansion is constant and equal to:

$$\frac{\Delta l}{l} = 1.5 \times 10^{-5} \text{ per degree Celsius}$$

Comment:

The value adopted is the same as for the Fe E22 or Fe E24 smooth round bars.

Chapter IV

TEMPERATURE DISTRIBUTION IN CONCRETE

The distribution of temperature in concrete is determined directly or by interpolation from experimental results of tests made in approved laboratories.

One knows directly the temperature in the concrete of a structure when he has available the experimental results for identical geometric arrangements, and provided approval of the testing commission for the results has been requested and granted.

The temperature in the concrete of a structure is obtained by interpolation when it has been determined analytically according to heat transmission laws. For geometric shapes closest to those proposed, for which we will have available experimental results that are known and accepted by the Testing Commission, this method gives values essentially equal to those obtained during tests.

Thus we proceed as follows:

The curve of temperature increase toward the face exposed to the fire follows the standard curve,

$$\theta - \theta_0 = 345 \log_{10} (8t + 1)$$

where θ is the temperature at time t expressed in minutes and θ_0 is the initial temperature.

The temperature toward the face not exposed to the fire is assumed to remain at temperature θ_0.

The temperatures at any point in a structural element in this thermal state follow the heat laws, the so-called "laws of Fourier."

Comment:

For example, Fig. 9 shows known results for the temperature reached inside a slab with different increasing values of the useful distance u (measured from the axis of the steel to the free surface of the slab exposed to the fire), as a function of the time it has been subjected to the effects of the fire in a standard furnace.

The curves in Fig. 9 are averages, since the temperature values depend in part on the thickness of the slabs.

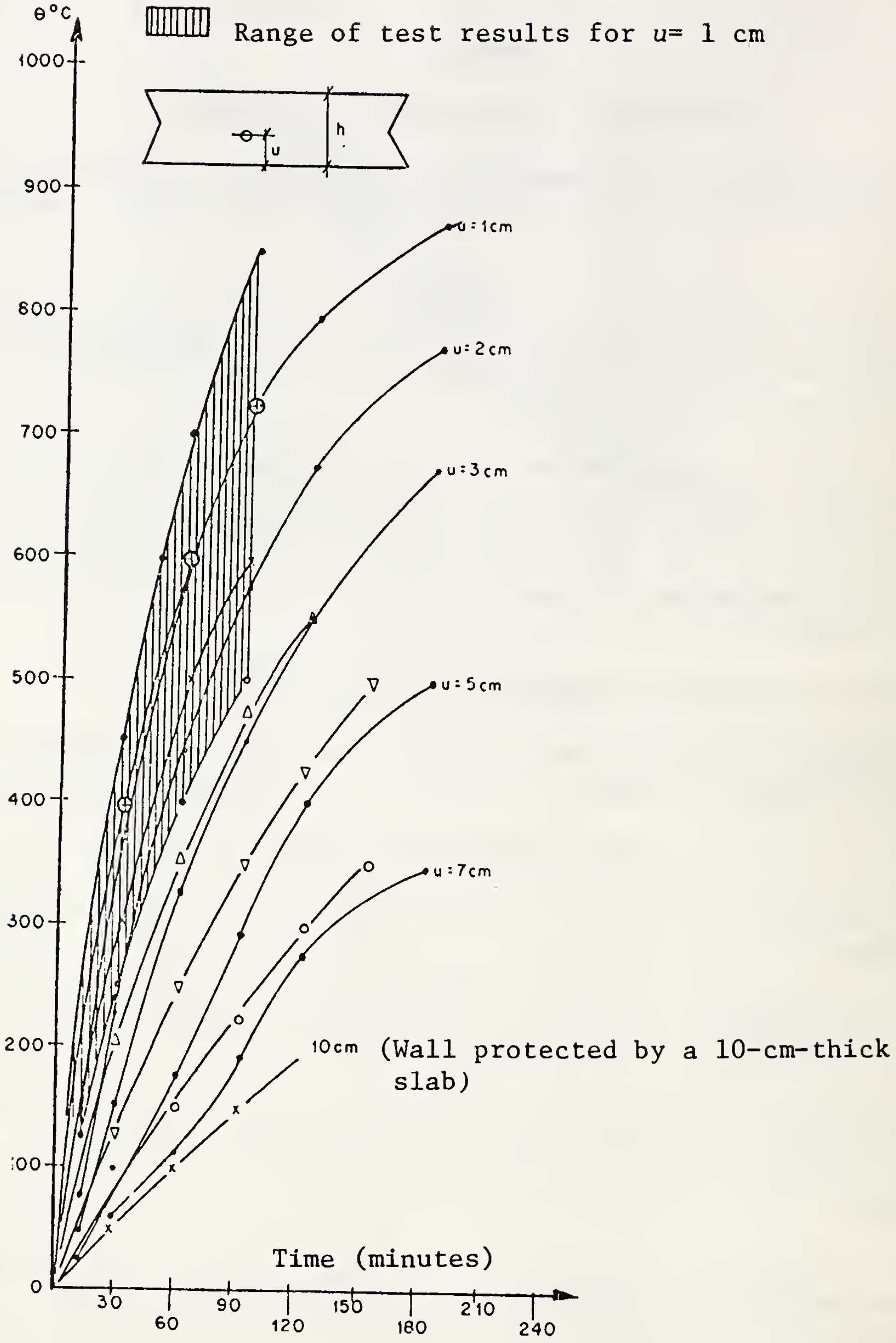


Fig. 9

⊕ CSTB results for an 8.4-cm slab

The curves above are averages, since temperature values depend partly on the slab thickness.

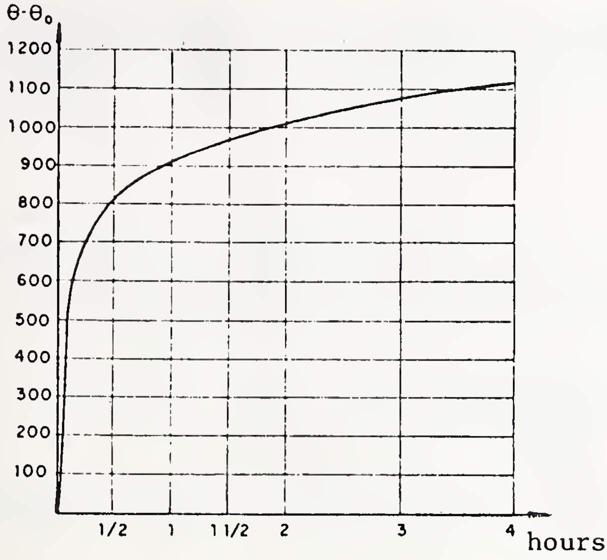


Figure 10. Standard curve for temperature rise.

In the case of floors the face exposed to the fire is the underface; for columns it is the whole surface; and for walls or columns incorporated in the walls it is the one or two faces exposed, depending on whether or not the wall is the dividing wall of a compartment.

In the case of a two-dimensional problem, the temperature, τ, at the point with the coordinates x and y is given by

$$\frac{\delta \tau}{\delta t} = \frac{\lambda}{c\rho}\left(\frac{\delta^2 \tau}{\delta x^2} + \frac{\delta^2 \tau}{\delta y^2}\right)$$

If this equation is worked out by the finite difference method, the section is divided into squares with sides $\Delta x = \Delta y$ and the time is divided into intervals

$$\Delta t = \frac{c\rho}{4\lambda}\overline{\Delta x}^2$$

such that

	i − 1	i	i + 1
j − 1		$\tau_{i,j-1}$	
j	$\tau_{i-1,j}$	$\tau_{i,j}$	$\tau_{i+1,j}$
j + 1		$\tau_{i,j+1}$	

$$(\tau_{ij})_{t+\Delta t} = (\tau_{ij})_t + (\Delta\tau_{ij})_{\Delta t}$$
$$= \frac{\tau_{i,j-1} + \tau_{i,j+1} + \tau_{i-1,j} + \tau_{i+1,}}{4}$$

Heat exchange between the structure and the hot and cold surroundings are found from furnace exchange coefficients normalized for the corresponding faces.

Comment:

If τ_0 is the temperature at the center of a square with side Δx located in the surroundings and in contact with the interface of the solid along the square with side Δx marked i, j, and if θ is the ambient temperature, we have

$$\tau_0 = \frac{\theta \Delta x + \left(\frac{\lambda}{h} - \frac{\Delta x}{2}\right) \tau_{ij}}{\frac{\lambda}{h} + \frac{\Delta x}{2}}$$

where h is the overall exchange coefficient of the furnace.

For the side exposed to the fire we can write

$$h = \alpha_c + \alpha_r \frac{T_m - TPC}{\theta - TPC}$$

$$\alpha_r = 40 \left(\frac{T_m + 273}{1\,000}\right)^3 (1+X+X^2+X^3)$$

$$X = \frac{TPC + 273}{T_m + 273}$$

where α_c is the convection coefficient, 6 kcal/m^2•hr•$^\circ$C

α_r is the radiation coefficient

θ is the temperature given by the standard curve

TPC is the temperature of the wall exposed to the fire and thus the average of the different values

$(\tau_0+\tau_{i,j})/2$ is the length of this wall, and

T_m is the temperature of the furnace masonry, which is difficult to estimate, so we set $T_m = \beta\theta$

The function β is adjusted to find the results already obtained for similar structures. We assume that the value $\beta = 0.85$ is readily adapted to the results for slabs, walls, and beams and that it is the value adopted in determining the results given below for these structures. By modifying this value of β in particular, we can determine the temperatures by interpolation.

On the side not exposed to the fire, which remains at temperature θ_c, the coefficient h is given by the larger of the two values

$$\begin{cases} 8\,(TPF - \theta_0)^{0,1} \\ 0,1 \end{cases}$$

where TPF is the average value of the temperature of the unexposed wall, and thus of $(\tau_0+\tau_{ij})/2$ along this wall. More sophisticated programs could, perhaps, be conceived which take into account the heat of vaporization of the water contained in the concrete.

Chapter V

BASIS FOR THE JUSTIFICATIONS AND EVALUATIONS OF STRESSES

The stability of a structure should be guaranteed for a definite time by the exposure criterion. Thus we have to be sure that the ultimate stresses over the whole cross-section of a prismatic element are, after this time, at least equal to the stresses due to the forces which are applied to it (taking into account the mechanical properties of the materials as a result of the fire).

We therefore are dealing with methods of calculation or analysis in the final state of resistance, aimed at working out a stability diagram that is statically permissible, taking into account possibilities of adaptation of the structure.

Comment:

Use of the method has allowed us to work out, by category of structure, the simple rules given in Chapter VII.

5.1 Stresses to Take into Account

The reduction elements in every cross-section are calculated by the usual rules appropriate to the structures by taking into account the possibilities of adaptation and are obtained by considering the following loads and effects:

If (G) is the stress due to the continuous load,

(P_1) is the stress resulting from service loads but not including the possible increases for dynamic effect

(V) is the stress due to normal climatic effects

(T_1) is the stress due to total expansion effects, taking into account the indications and departures given in Sect. 5. 11, and

(Y) represents instability phenomena expressed, for example, by introduction of imaginary arrows to calculate the columns,

then the total stress is defined symbolically by

$$(S) = (G) + (P_1) + (V) + (T_1) + (Y)$$

We see that in the analysis of a section in the ultimate states, the hyperstatic effects of the thermal gradient are not to be taken into account because of adaptation phenomena.

Comment:

The difference in temperature between the exposed and unexposed faces of a frame element leads, for the latter, to a variation in curvature which most of the time is opposed by the hyperstatic connections of this element. As a result, there still are internal effects which can be calculated.

The analyses of stability at failure are made in the final state of adaptation, and rotations of the plastic sections thus have the effect of dissipating the forces mentioned.

The rotations of the sections behaving in plastic fashion are, however, large and could result, in some cases, in rupture of the steel reinforcing parts that go through them, which explains the limits actually imposed on considering continuities in slabs and rough masonry.

(See comment on Sect. 7.44.)

5.11 Stresses Due to Overall Effects of Expansion

Variation in length of each structural element subjected to a fire is, in general, opposed by surrounding elements, and this results in forces that can be calculated.

The expansion caused by a fire in a reinforced concrete element will be calculated from the unit coefficient of expansion, $\frac{\Delta l}{l} = 10^{-5}$, and the average temperature reached by this element after an exposure time determined by the exposure criterion used for the structure in question.

In the absence of more precise calculations of this temperature, we assume the values

F		1/2 h	1 h	1 1/2 h	2 h	3 h	4 h
1. Elément exposé au feu	1 face (plancher) 2.	100	200	300	350	450	500
	2 faces (poteau) 3.	150	300	400	500	600	700

Key: 1--Element exposed to fire; 2--(floor); 3--(column)

The distance over which the expansion is to be applied is usually, for vertical elements, the height of a floor. For horizontal elements this distance depends on the size of the fire at a given instant, which can be related to the fire load of the premises and to their partitioning by masonry (or other incombustible materials) and to any other means for compartmenting the fire.

Comment:

Indeed, partitioning creates an obstacle to the spread of the fire, and burning of the fire load during the temperature rise according to the standard curve results in a maximum duration of the fire at a particular site. If this length of time corresponds to the exposure criterion specified, we can assume that the fire occurs simultaneously throughout a compartment. In the opposite case, this means either that the fire center is displaced in the damage zone by affecting only a part of this zone at each instant, or that the fire could not last throughout the whole time corresponding to the exposure criterion, if it occurs everywhere at once, so that the average temperature will only be that at the end of the actual time of the fire.

In the absence of more precise calculations, the following types can be distinguished:

- Linear buildings with a normal partition density and low fire load, such as dwellings, for which a distance of about 10 m (or a running house width) can be assumed.

- Buildings or premises with few partitions and a high fire load, such as warehouses, for which the distance will be the smallest distance between compartment partitions, unless special arrangements for compartmentation of the fire other than the partitions (smoke outlets) do not permit this distance to be reduced.

- Office buildings, schools, hospitals, warehouses with a small fire load, parking garages, etc., for which the distance will be that between the above-mentioned limits, depending on their fire potential, partition arrangement, and fire compartmentation.

One could, for example, assume only 50% of the smaller of two distances between joints, or between compartment partitions in linear cases. This limit could be decreased without going below 25%, and below 10 m in the case of compartments divided off by masonry or concrete slab partitions extending beyond the false ceilings.

There is thus an advantage in increasing the number of expansion joints and making them wide enough (based on the probable overall expansion) without forgetting, however, that the fire protection requirements should not lead to planning structures whose design will be second-rate with respect to linear stresses. Therefore it is sometimes better to not plan for too many joints, even if it means taking into account the effects of overall expansion in the structure and their consequences in masonry such as fire walls (e.g. by providing them with means to resist the consequences of this expansion or surrounding them with specially treated joints so as to protect them from the forces generated). On the other hand, it appears always possible to make the expansion joints wide enough.

In justification for linear buildings related to the distances between joints, based on the regulations in force, it is assumed that no account is taken of the stresses due to the effects of overall expansion.

Comment:

The behavior of linear buildings, as determined during actual fires, indicates that, when the requirements of the regulations for reinforced concrete regarding the frequency of expansion joints are satisfied, it is not necessary to take into account the effects of overall expansion in structural calculations and the arrangement of structural elements.

The expansion joints of the superstructures should, however, have a minimum width.

Comment:

We assume, for example, 2 cm for 2 hr of exposure to fire, with a proportional rule applied to the average temperature for exposure criteria of different duration.

5.2 Analysis in Simple or Compound Bending

The analysis in the ultimate strength state will be made by assuming conventionally:

--For the concrete, a rectangular parabolic diagram characterized by a maximum stress $\sigma'_{j\theta}/1.5$ and by elongations of $2x10^{-3}$ and 3.5×10^{-3}.

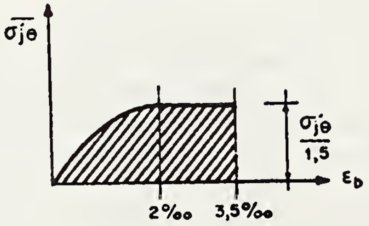


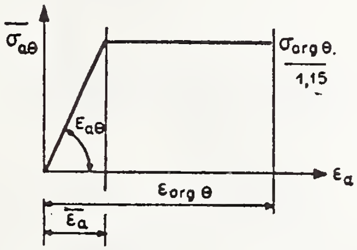
Figure 11

Comment:

In the absence of precise values, the shortening of 3.5 per thousand is assumed not to be affected by the temperature.

The symbol $\sigma_{j\theta}$ represents the value of the failure stress on day j of the fire (j being normally greater than 90) at the temperature of the point located at the barycenter of the compressed zone.

--For the steel, an elastoplastic diagram characterized by $\sigma_{arg\ \theta}/1.15$ and by elongations of $\overline{\varepsilon}_a$ and $\varepsilon_{arg\ \theta}$



$$\left(\varepsilon_a = \frac{\sigma_e}{E_a} = \frac{\sigma_a\ \theta}{E_a\ \theta}\right)$$

Figure 12

In the case of steel with a flow bearing, we take the smaller of the two values $\sigma_{arg\ \theta}/1.15$ or $\sigma_{e\theta}$ in the diagram above.

To the extent that one uses the real diagram, $\sigma_{a\theta} = f(\varepsilon_a)$, the value of $\overline{\sigma_{a\theta}}$ is obtained by assuming a reducing coefficient of 1.15.

For simplification, we assume the following diagrams which correspond to several frequent cases:

--Tension with eccentricity such that the center of pressure lies between the two layers of steel (equilibrium force N);

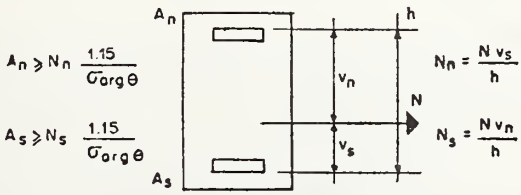
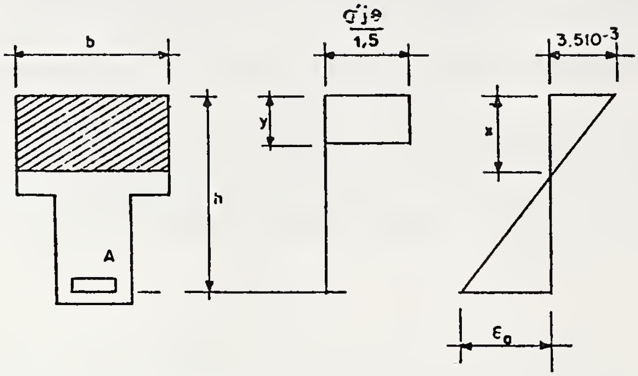


Figure 13

--Simple bending (equilibrium force M):



$$F_a = F'_b = A\,\frac{\sigma_{arg\,\theta}}{1{,}15}$$

soit $y = \dfrac{F'b}{b\,\sigma'_{j\theta}/1{,}5}$ $\quad x = y/0{,}8 \quad \varepsilon_0 = 3{,}5\,10^{-3}\,\dfrac{h-x}{x}$

$M \leqslant F_0\left(h - \dfrac{y}{2}\right)$ ovec $\varepsilon_{arg\,\theta} \geqslant \varepsilon_a \geqslant \overline{\varepsilon_a}$

Figure 14

--Compression with little or no eccentricity (equilibrium force N):

$$N \leqslant \frac{\sigma'_{j\theta}}{1{,}5}\,b.2v$$

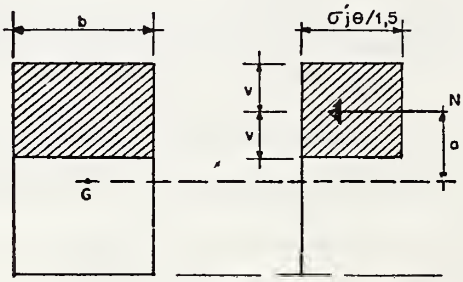


Figure 15

The compressed steel can be taken into account by using the smaller of the two following stresses:

Either the value $\sigma_{arg\ \theta}/1.15$

or the value which results from the shortening of the concrete fibers located at the same height

When the justification of the exposure criteria is done by calculation, we can assume that it is satisfactory if the results obtained do not deviate more than 5% from the required values.

5.3 Shattering

The materials used in making concrete, the amounts of each, and the placement procedures should be selected so that premature shattering of the concrete will not have adverse effects on the behavior of the structure.

Comment:

Shattering can result in the elimination of section of concrete necessary for stability. Shattering may also accelerate heat transmission phenomena.

Observing good practice concerning the mixing of the concrete will make it possible to achieve this objective for all structures that meet the simple rules defined later on on the basis of the type of structure.

Comment:

It should be kept in mind in this regard that a protective mesh is needed for useful distances of more than 7 cm.

The justification for the arrangement of the forms and the various reinforcements for them resulting from the application of the simple rules will be carried out for primary and secondary beams on the assumption that shattering may expose reinforcement locally, perhaps in a simple cross section.

Besides the analyses called for by the specifications given in Paragraph 5.1 and 5.2, the design engineer should justify the strength of the cross sections by disregarding the stronger reinforcement bars among those placed near the edge. The span in question should be justified by substituting a factor of 1.05 for the factor 1.15 given in Paragraph 5.2, and this should be done for the support as well as the span. It is assumed that this analysis will not be used when beams contain more than 8 rebars at mid-span and in the case of slabs.

Comment:

It is assumed that this loss of formwork can affect only the beams containing more than 8 bars, all of the same type and the same diameter.

One could omit taking into account this shattering when one of the following conditions is met:

--justification, by tests, of good shattering behavior on the part of the type of construction planned.

Comment:

This justification concerns more specifically industrial products which are offered as prefabricated.

--justification based on observing the rules of concrete preparation in a manner to limit the shattering hazard.

Comment:

These rules will be worked out during a test program in progress.

--providing additional protection for exposed walls,

--providing a protective grid in areas particularly sensitive or arranging reinforcement in these zones (frames and longitudinal bars) in a manner to create a reduced mesh under the conditions set forth in Paragraph 6.3.

Comment:

The sensitive zones are, in the case of secondary beams, the vicinity of the lower bars in the span and the zone of compressed concrete on the support (in the continuous case).

CHAPTER VI

GENERAL STRUCTURAL RULES

This chapter sets forth a number of rules common to all types of structures which make it possible to arrange the choice of structural elements so as to satisfy most easily the required criteria for duration of resistance to fire.

6.1 Expansion Joints

The reader is referred to Paragraph 5.11 and the comments on it, where these matters have already been examined.

Expansion joints dividing a flameproof or fireproof element should be studied taking into account the dimensional variations to be considered.

Comment:

The object is to oppose the passage of flames and the omission of inflammable gases through such joints. The following solutions may be studied by way of example:

--Insert, anchored partly to the skeleton and partly to the joint,

--Rabbeted support between two stiffened elements with sponge material between. (see Figure 16)

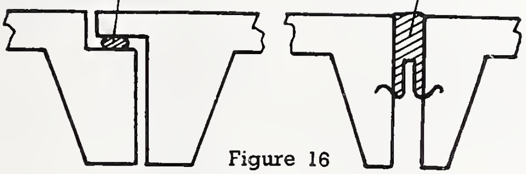


Figure 16

6.2 Geometric Arrangements

6.21. Floors and Beams

In the case of floors and beams, these elements should not, after being built, exhibit weakened sections.

Comment:

The three principal causes of weakening are the following:

--first, keying between elements, when it does not extend over the full thickness of the elements joined nor restores the monolithic state of the structure by means of reinforcing steel (especially for the fireproof criterion):

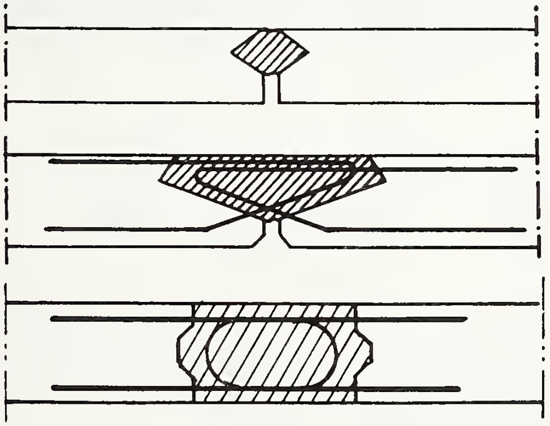


Figure 17

--second, the statically determinate state of the elements, because continuity on supports enters into the transmission of forces with the help of the upper reinforcement steel which is less affected during a fire:

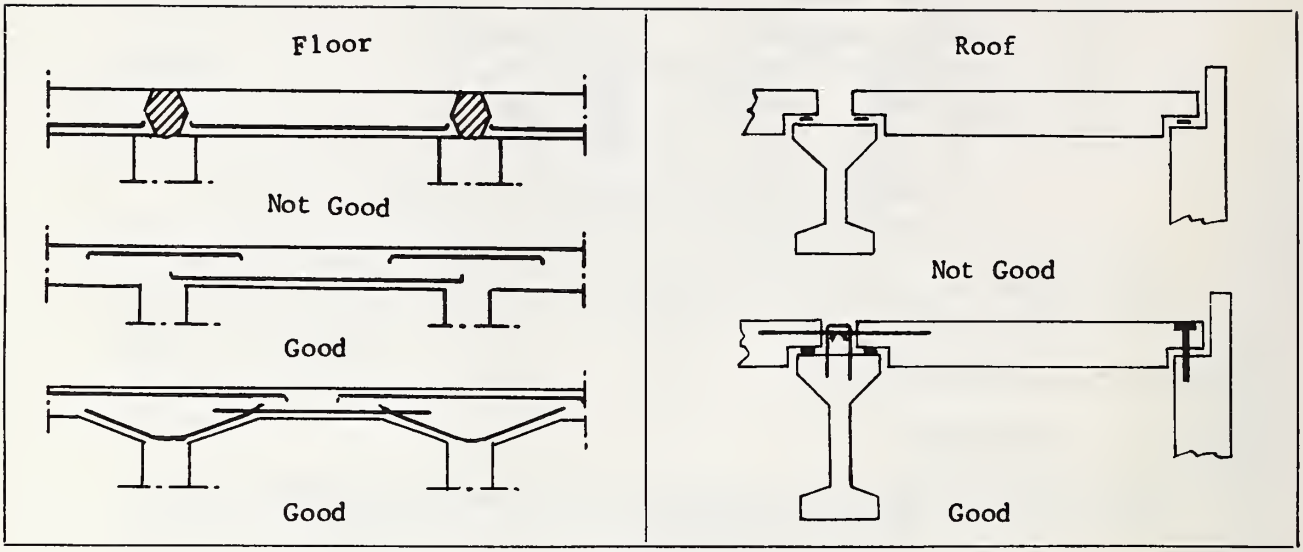


Figure 18

--Finally, examination of the temperature curves in a cross-section shows that the parts that are hottest are the projecting angles, and that the temperature has more effect, the more slender the element. Thus we would want to avoid ribs that are too slender and walls that are too thin.

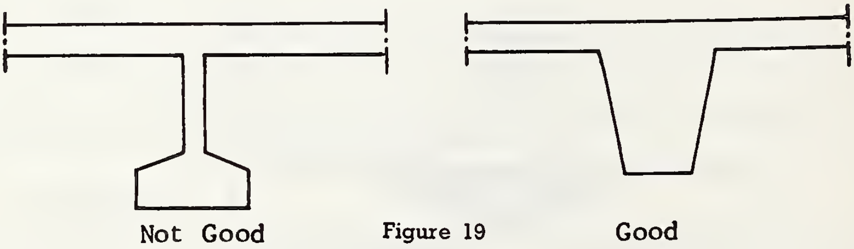


Figure 19

One can, however, conceive of structures consisting of elements which are basically keyed elements, to the extent that they are handled in a manner to be fireproof.

One can also conceive of isostatic structures where the fireproof concept is satisfied by the supports and resistance to fire is obtained by covering the steel or by external protection.

On the other hand, when the phenomena of overall expansion predominate, the deformation tendency of the structures seems to be a favorable element.

When it is impossible to avoid grooved supports in a zone that may be exposed to fire, if there is no additional shielding we have to protect by reinforcement the angles of the concrete whose spalling or shattering could result in collapse of the structure, the main steel reinforcement being placed with the necessary covering.

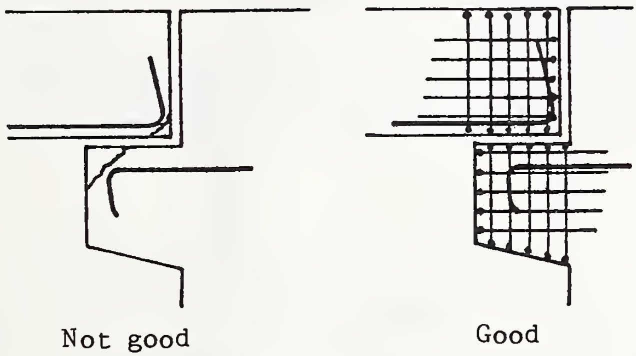


Figure 20

6.22 Loadbearing Elements

As far as loadbearing elements are concerned, their sections will be designed for the best possible adaptation to the effects of overall expansion.

Comment:

Thus one should see that the largest dimension of the element is positioned parallel to the smallest dimension of a block between joints or to that of a fireproof compartment.

At the same time, it is preferable that the longitudinal wind-bracing elements be concentrated at the center of the blocks between expansion joints.

Plan view of building

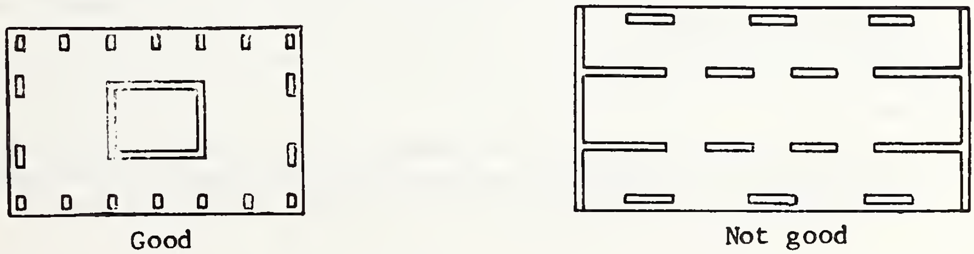


Figure 21

6.3. Arrangement of Reinforcement

Increasing the covering of the steel is favorable for resistance to fire.

Comment:

Since temperatures are always higher in the vicinity of hot walls and in projecting angles, the calculated reinforcing steel should be spaced further away from them to balance the forces (bending and shear force).

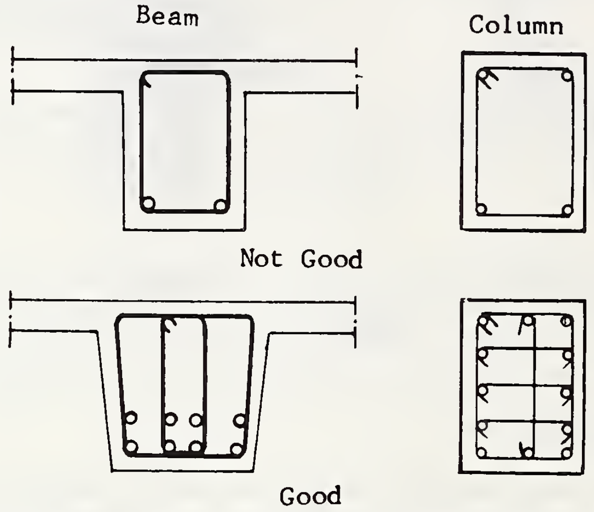


Figure 22

When the useful distance of the first loadbearing bars is more than 7 cm to the face exposed to the fire, it is necessary to provide a protective mesh enclosed in 1.5 cm of concrete, the grid spacing of which is not to exceed 10 cm in one directing, the diameter not being significant.

Comment:

Beyond a certain thickness of covering it has always been found that during actual fires early shattering of the concrete will rapidly reduce the effect sought by using the covering.

Remember that too long a useful distance may not be advisable for the structure to behave well under service stresses.

The slabs and beams should not have reduced reinforcement on the face exposed to the fire.

Comment:

It seems desirable, even when they are not justified by design calculations, to provide systematically for bars in the underface of the beams or slabs which are anchored in the supports.

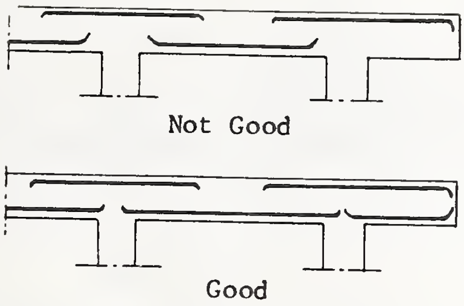


Figure 23

Reinforcing bars which need not be taken into account in the fire-resistance calculations should be placed in the vicinity of the exposed walls.

Comment:

This procedure results in keeping close to the walls rebars placed in the structural elements for percentage reasons (columns, shells) and not determined to balance calculated forces. It should be added that such a rule also offers the advantage that the structure remains properly reinforced for its routine stresses.

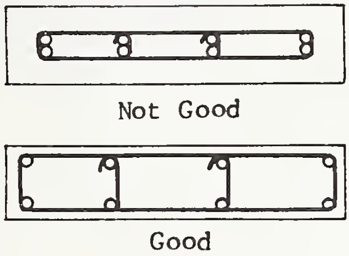


Figure 24

6.4 Execution tolerances

When the fire-resistance depends mainly on the value of the coverings selected and the latter are larger than those provided by the rules under routine stresses, it is necessary:

--On the one hand, to indicate by appropriate detail on the implementation plans the value of these coverings; and

--On the otherhand, to more particularly pay attention to observing these values in implementation with a tolerance of ± 10%.

Chapter VII

CONSTRUCTION RULES ACCORDING TO STRUCTURE CATEGORY

7.1 Columns

7.11 Simple Rules

These simple rules concern columns subjected to simple compression, reinforced longitudinally with a percentage of 2% or less, and a slenderness of 50 or less.

The following table gives for F (expressed in hours) fireproof or fire-resistive exposure criteria:

--The minimum dimension, a_{cm}, of the side of the column for

a square column
a rectangular column where b = 5a; and

--The minimum useful distance, u, from the steel to the face.

F		1/2	1	1 1/2	2	-3	4
a_{cm}	Square column	15	20	24	30	36	45
	Square column, 1 face exposed to fire	10	12	14	16	20	26
	Column b = 5a	10	12	14	16	20	26
u_{cm}	Case of reinforcing steel with percentage	none					
	Case of reinforcing steel taken into account in calculation	1	2	3	4	6	7

Linear interpolation is used to define, as a function of the fire resistance time sought, the minimum permissible section of concrete for the values of the ratio b/a between 1 and 5.

Comment:

For example, for a time of 2 hr, one can select

16 x 18 (or more)
23 x 69 (or more)
30 x 30 (or more)

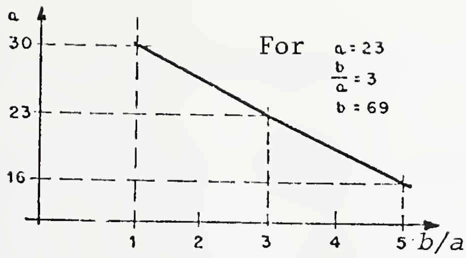


Figure 25

For columns on either side of an expansion joint, when they can be surrounded by the fire, the minimum section is defined as though the joint did not exist.

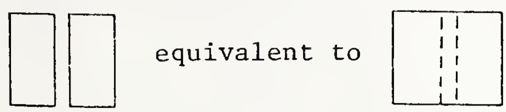


Figure 26

Round columns can be treated according to the rules applicable to square columns of the same area.

7.12 Temperature of Concrete and Steels

The temperature is estimated by the procedure given in Chapter IV.

Comment:

Application of this method to some current cases has given the following values:

Table of Average Temperatures of Concrete (and Reinforcing Steel) in Columns for Distances u to the Free Wall as a Function of Length of Exposure to Fire

u (cm)	Length of exposure to fire			
	1/2	1	1-1/2	2
1.5	410 - 350	600 - 550	700 - 660	770 - 930
3	280 - 240	490 - 430	610 - 540	990 - 620
4.5	120 - 90	340 - 300	480 - 410	580 - 500

The first figure is given for a concrete thickness of 18 cm.

The second is for a concrete thickness of 50 cm.

An average temperature for a concrete thickness between these two values is obtained by linear interpolation (at a given distance u).

The temperatures corresponding to distances intermediate between the values given in the table are also defined by linear interpolation.

Finally, temperature distribution is not uniform along the edge.

A parabolic variation will be allowed by adding a weighting coefficient of 1.2 for the temperature at the corners and of 0.9 on the axes of symmetry of a face to the temperature value in the Table, as shown in Fig. 27.

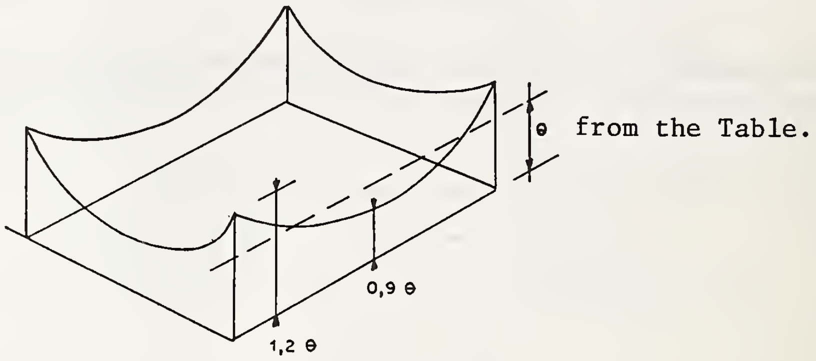


Figure 27

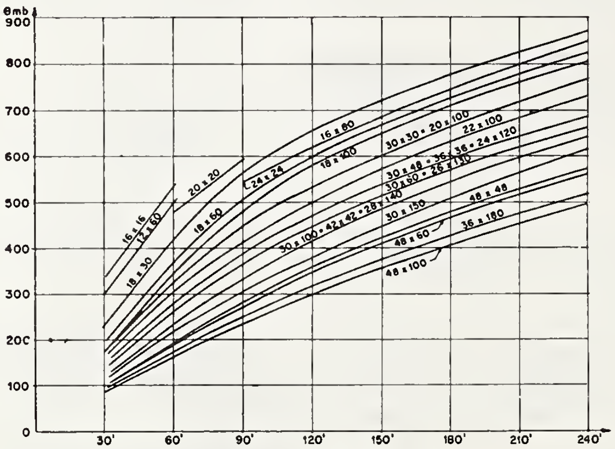


Figure 28.
Mean temperature in columns surrounded by the fire.

It could be useful to know the temperature at the center of a column. The following table give some temperature values for certain column sections.

Temperature at the center of columns

	1/2	1	1 1/2	2
18 × 30 (ou 60 ou 100)	30	150	300	400
30 × 30	0	30	100	140
30 × 60 (ou 100)	0	15	60	110
48 × 30	0	15	50	100
48 × 60 (ou 100)	0	0	0	10

Note: ou = or.

7.13. Other Columns

If the rules of Paragraph 7.11 are not applied, the columns are calculated as indicated in Chapter V, using the temperature tables calculated according to Paragraph 7.12.

Buckling is taken into account using the methods for calculation in the cold.

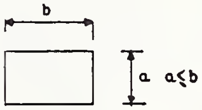
Comment:

Certain of these methods again introduce reduction coefficients for the stresses. Others introduce additional eccentricities. And still others make a failure calculation taking into account second-order effects.

7.2. Stringers

This name is applied to elements subject to tension or in composite bending with tension when the neutral axis is located outside the 2 layers of steel.

It is assumed that the lengths of time (expressed in hours) of the exposure criteria (fireproofness or fire resistance) are satisfied when the minimum values indicated in the table below are met.



F	1/2	1	1 1/2	2	2 1/2	3
a (cm)	8	12,5	15	20	24	28
u (cm)	2,5	4,0	5,5	6,5	8,0	9,0
Concrete area a x b (cm^2) equal to or greater than	128	312	450	800	1 150	1 570

Resistance can also be justified using the methods set forth in Chapter V using the temperature distributions given for columns in Paragraph 7.12.

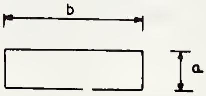
Nevertheless, when one is concerned with reinforced concrete stringers where the overlaps are in the zone exposed to fire, the useful distance values given in the table above should be maintained.

Comment:

This restriction as to maintaining a minimum useful distance flows from the uncertainty that adhesion persists. It is logical to assume that reinforced encircling steel will make this condition manageable.

7.3. Bearing Walls

This article applies to loadbearing elements in which the larger dimension is more than 5 times greater than the smaller ($b \geq 5a$).



7.31. Simple Rules

These simple rules concern bearing walls in which the slenderness is equal to 50 or less.

The durations (F, in hours) of the exposure criteria (fireproofness and fire resistance) are achieved if the minimum values of a and u in the table below are maintained.

F		1/2	1	1 1/2	2	3	4 h
a (cm)		10	11	13	15	20	25
u (cm)	Case of reinforcing steel based on percentage	none					
	Case of reinforcing steel taken into account in the calculations	1	2	3	4	6	7

7.32. Temperatures in the Concrete

The temperature is estimated according to the provisions of Chapter IV.

Comment:

Application of this method to several everyday cases gave the following values:

Temperature at the useful distance(u)

Depending on the fire-exposure times, the temperatures reached by the concrete at distance u from the external surface (exposed to the fire) are those given in the following table:

	F	1/2	1	1 1/2	2	3
Fire on 2 sides	u = 4,5 cm	100	270	400	500	
	u = 3	225	405	520	600	
	u = 1,8	340	540	640	700	
Fire on 1 side only	u = 4,5 cm	100	270	400	475	570
	u = 3	225	405	520	590	680
	u = 1,5	340	540	640	700	785

Intermediate values (duration and useful distance) will be obtained by linear interpolation.

Average temperature of the concrete

The table below gives average temperatures for the concrete of shells, calculated by the general method, for several cross-sectional dimensions and fire-exposure times.

	F	1/2	1	1 1/2	2	3
Fire on 2 sides thickness 18 cm or more	Average temperature	170	350	480	570	
	Temperature in the center	25	140	270	380	
Fire on 1 side only (average temperature)	a = 15 cm	100	200	280	350	440
	a = 20	75	160	220	275	350
	a = 30	50	100	150	180	250
	a = 45	30	70	100	120	165

7.33 Other Walls

If the rules of Sect. 7.31 are not applied, the walls are calculated according to Chapter V using the temperature tables calculated in Sect. 7.32.

Walls whose reinforcement sections have been determined by the methods applied generally to columns will be analyzed as such.

Other walls, not reinforced or reinforced at percentages not taken into account in the homogenized section, will be analyzed by taking account of the phenomena of buckling and without an actual experiment, as is customary for static analyses in the cold.

Comment:

> Some methods take into account these phenomena in the form of undervaluation coefficients of the nominal stress. These will be used for analyses in fire.
>
> Other methods take into account these phenomena in the form of additional eccentricities which will be used for analyses in fire.

These walls will, however, have a thickness greater than 8 cm, and when they must be fireproof, their thicknesses will be at least equal to those given in Sect. 7.31

7.4 Cast Concrete on Continuous Supports

This section discusses solid slabs or pre-slabs supported on 2, 3, or 4 sides with linear reinforcement of reinforced or prestressed concrete.

7.41 Construction Arrangements

Besides the arrangements already described, the lower face of the slab exposed to fire should contain a continuous grid of reinforcement using the lower rebars, a part of which should be anchored to the supports.

Comment:

> This rule will be considered satisfied if, in the case of welded fabric, there is regularly a weld to the supports and if, in the case of high-adhesion rods, we extend beyond the support alignment (anchoring it) a section about 1/6 of the span section in the bottom strand:
>
> —Either all the rods with l_d,
> —or 1/6 of the rods with l_d, (l_d, is the anchoring length)

7.42 Simple Rules for Slabs and Pre-Slabs of Reinforced Concrete

The duration (in hours) of the exposure criteria (fireproofness and resistance to fire) are achieved for slabs when the minimum layouts in the table given below are used.

We note that:

h is the slab thickness (cm),

e is the thickness of the covering and its surfacing (cm),

M_o is the isostatic moment under the loads and additional loads according to the indications in Chapter V, and

M_w and M_e are the bending moments balanced by the reinforcing steel on supports with a free length within the span in question, l_w and l_e.

Comment:

One could take into account, in calculating M_w and M_e, theoretical capping added by construction on supports with low continuity to take into account the moments that are not calculated.

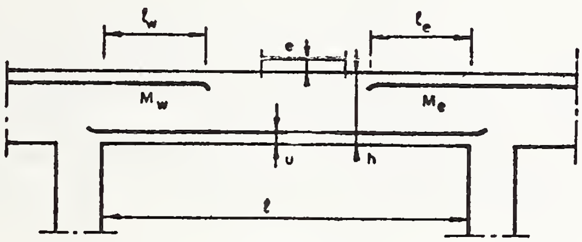


Figure 29

We use linear interpolation to find u and $(l_w+l_e)/l$ as a function of $(M_w+M_e)/2M_o$.

Time of resistance to fire		F	1/2	1	1 1/2	2	3	4 h
Minimum thickness		$h + e$ (cm)	6	7	9	11	15	17,5
Without rebars on supports	$\frac{M_w + M_e}{2 M_0} = 0$	u (cm) $\frac{l_w + l_e}{l}$	1 0	2 0	3 0	4 0	6 0	8 0
With rebars on supports	$\frac{M_w + M_e}{2 M_0} \geqslant 0{,}50$	u (cm) $\frac{l_w + l_e}{l}$	1 0,25	1,5 0,3	2 0,4	2,5 0,5	3,5 0,55	4,5 0,6

Comment:

The minimum thicknesses involved in the degree of fireproofness depend, probably, on the water content of the concrete. Whenever possible, therefore, thicknesses greater than those given in the table should be used.

In the case of slabs which carry load in a single direction, the above rules concern only the loadbearing rods.

In the case of slabs bearing on three or four sides, the same verification should be made for two bearing layers.

Comment:

It is recalled that bars positioned either to meet a minimum percentage or a condition of resistance to shattering or for distribution reasons are not subject to the above rules.

Any moment on a support greater than 0.15 times the isostatic moment of the span concerned could not be taken into account in the application of the above rules unless the shattering resistance conditions are verified in the corresponding section.

Comment:

To visualize adaptation, the breaking moment balanced by the steel must be at least equal to the moment of first cracking of the concrete.

In the case of a rectangular section of width b and total height h_j, the value of the moment of first cracking of the concrete is $\sigma_j b h_i^2/3.6$.

In the case of slabs made in two stages and whose lower part has not been handled by the Avis Technique ["Technical Advice"] procedure with monitored self-control, the following rules should be applied:

-- The minimum thicknesses are those of the table above, increased by 2 cm, and

-- The lower rods of the first stage should form a grid whose openings do not exceed 10 cm in one direction.

In the case of slabs made in two stages and whose lower part has been handled by the Avis Technique procedure with monitored self-control, the rules of the Cahier des Prescriptions Techniques, an appendix of the Avis Technique, should be followed.

Comment:

These arrangements result from the present state of our knowledge concerning shattering.

Duration of exposure to fire F		1/2	1	1 1/2	2	3
With any h	$u = 1,5$	340	540	640	700	785
	$u = 3,0$	225	405	520	590	680
	$u = 4,5$	100	270	400	475	570
$h \geqslant 9$	$u = 7,5$	20	140	230	300	400
$h \geqslant 12$	$u = 10,5$	0	50	120	180	270
$h \geqslant 15$	$u = 13,5$	0	20	50	100	180

Intermediate values of u are obtained by linear interpolation.

Figure 31 gives some values of average concrete temperature as a function of the duration in hours of exposure of concrete cast elements to fire.

Average concrete temperature

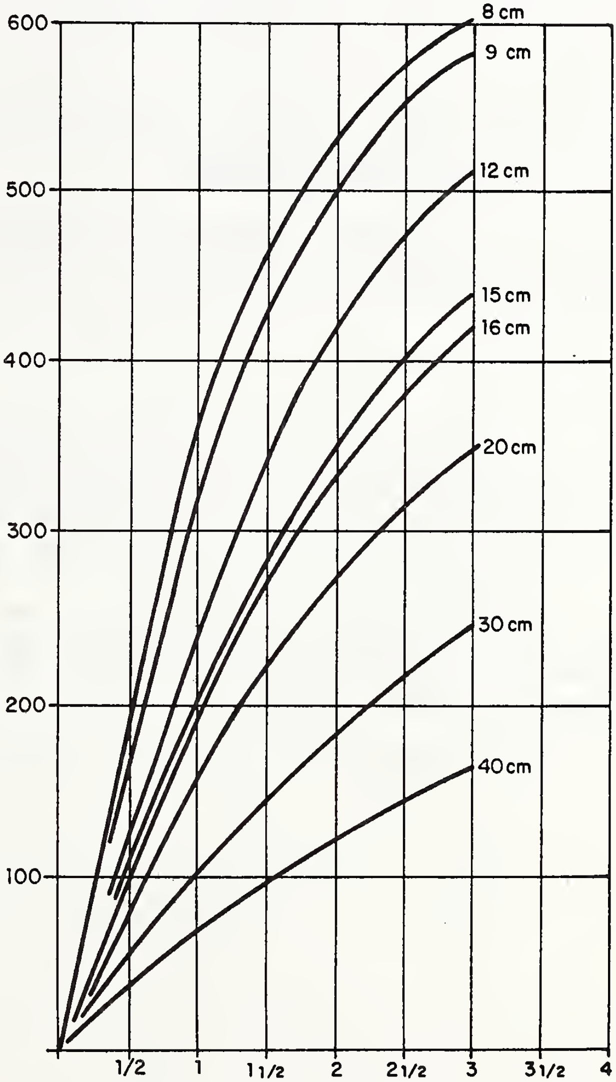


Figure 31

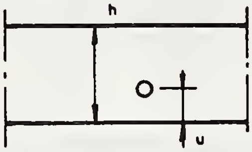
Duration of exposure to fire (in hours)

7.43 Temperature in the Concrete

The temperatures will be calculated by the method given in Chapter IV.

Comment:

The application of this method to several current cases has given:



with $8 \leqslant h \leqslant 40$

Figure 30

7.44. Other Slabs or Preslabs of Reinforced Concrete--Slabs and Preslabs of Prestressed Concrete

If the preceding rules are not applied, cast elements will be calculated in accordance with the provisions of Chapter V using temperatures estimated in accordance with Paragraph 7.43.

Comment:

In the case of preslabs, the rule of derogation of seams (DTU Rules for design and calculation of full-slab floors made of prefabricated concrete preslabs*) is applicable without additional adjustment to the rules applicable in the cold. The minimum thickness for Criterion No. 2 on fireproofness is the same as that given in 7.42.

One will make sure thus that the half-sum of the moments on the support plus that of the span indeed balances the moment induced by the applied loads, and one will verify the length of the bars in accordance with the moment curve that results.

No justification for the shear force is required.

Comment:

When the perpendicular forces induced by the effects of expansion are to be taken into account, justifications in compound bending will be necessary.

The provisions of Paragraph 7.42 (and the associated comment), which are concerned with observing non-fragility conditions, apply equally to the slabs and preslabs discussed in the present Paragraph 7.44.

Comment:

Unless one justifies that the hyperstatically continuous supports are well designed so that rotation of plastic hinges permits adaptation, one will not seek to justify by calculation the arrangement of reinforcing steel which excessively minimizes the intervention of the span bars. Hence it would not be necessary to deviate too far in the unfavorable direction from the limits given in 7.44 for the minimum useful distances and for the maximum involvement of continuity.

The functional justifications for the plastic hinges, in particular, should take into account the elongation characteristics beyond constriction of the reinforcing bars, their adhesion and anchoring in the concrete, the forms of the support sections, and the percentage of steel provided.

7.5. Main and Secondary Beams, Reinforced Concrete or Prestressed Concrete

7.51. Simple Rule

The F degrees (expressed in hours) of the exposure criteria (fire-proofness and fire resistance) are achieved for main and secondary beams if the minimum conditions of the preceeding tables are observed.

---

* In preparation.

Note that:

--e is the thickness of the covering and its finish in cm,

--M_o is the isostatic moment under the loads and overloads according to the provisions of Chapter V,

--M_w and M_e are bending moments balanced by the steel on supports with free length inside the span considered, l_w and l_e

--T is the shear force, once direct transfer has been deducted, and

--τ is the corresponding shearing stress, and is equal to $T/0.875h.b$ (b, or b_o if there is a heel)

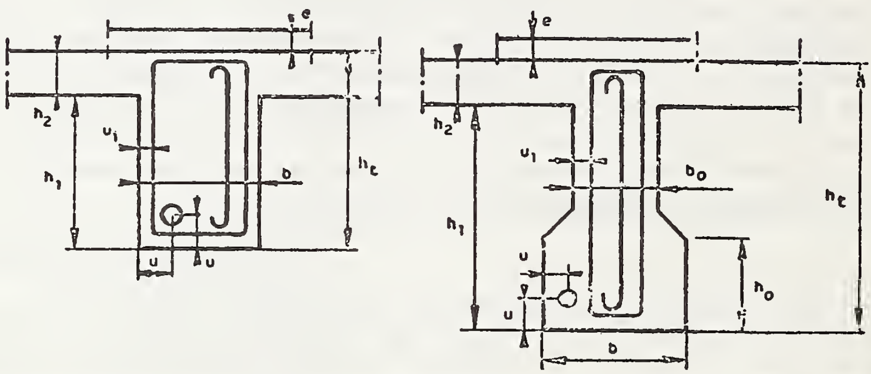


Figure 32

Comment:

The heel beams generally encountered are of very variable dimensions and it is difficult to define simple rules covering all cases.

We find, in particular, that we should justify by calculation beams for higher degrees than 1.5 hr.

The dimensions of the heels, their height and width, can vary, the thickness of the web often being the basic criterion. The distribution of the reinforcing structures may also differ with regard to the number of layers and the number of rods per layer. Then we have to justify by calculation.

In the case of slabs made in two stages and whose lower part has not been made according to the <u>Avis Technique</u> procedure with monitored self-control, the rules for increasing the minimum thickness and reinforcement grid given in Sect. 7.42 are also applicable.

In the case of slabs made in two stages and whose lower part has been made according to the <u>Avis Technique</u> with monitored self-control, the provisions of the <u>Cahiers des Prescriptions Techniques</u> appended to the <u>Avis Technique</u> should be followed.

Comment:

These arrangements result from the present state of our knowledge concerning shattering.

	F	1/2	1	1 1/2	2	3	4 h
Minimum thickness $(h_2 + e)$ cm	Entraxe des poutrelles ⩽ à 2,5 m (a)	5	6	8	10	14	16,5
	Entraxe des poutrelles > à 2,5 m (b)	6	7	9	11	15	17,5
	Poutres croisées espacées au maximum de 2,5 m dans chaque sens (c)	4	5	7	9	13	15,5
$\frac{M_w + M_e}{2 M_0} = 0$	Poutres rectangulaires (d)						
	b minimal en cm (e)	12	16	20	24	32	40
	Poutres à talon (f)						
	b minimal en cm (g)	16	20	24	32	40	50
	b_0 minimal en cm (h)	8	10	12	14	16	18
	h_0 minimal en cm (i)	5	8	12	20	32	50
	Nombre minimal de lits inférieurs (j)	2	2	2	3	3	4
	u (cm): avec b minimal (k)	2,5	4	5,5	6,5	8,0	9,0
	u (cm): et avec un minimum de barres par lit de : (l)	2	2	2	3	4	5
	u (cm): avec b supérieur à la plus grande des 2 valeurs : 1 mètre ou 1,5 h_1 et avec 10 barres par lit (m)	1	2	3	4	5	6
	$\frac{l_w + l_e}{l}$	0	0	0	0	0	0

Key: (a) and (b) Beam spacing on centers; (c) Crossed beams with maximum spacing of 2.5 m in each direction; (d) Rectangular beams; (e) Minimum b in cm; (e) Minimum b_o in cm; (f) Beams with heel; (g) Minimum b in cm; (h) Minimum b_o in cm; (i) Minimum h_o in cm; (j) Minimum number of lower layers; (k) With minimum b; (l) And with a minimum number of bars per layer equal to: (m) With b larger than the greater of two values: 1 meter or 1.5 h_1, and with 10 bars per layer.

Comment:

The minimum thicknesses that enter into the degree of fireproofness depend probably on the water content of the concrete. Therefore it is desirable, whenever possible, to use thicknesses greater than those given in the table.

	F	1/2	1	1 1/2	2	3	4 h
$\frac{M_w + M_e}{2 M_0} \geqslant 0{,}5$	Poutres rectangulaires (a)						
	b minimal en cm (b)	8	11	14	17	23	29
	Poutres à talon (c)						
	b minimal en cm (d)	12	16	20	24	32	40
	b_0 minimal en cm (e)	8	10	12	14	16	18
	h_0 minimal en cm (f)	5	8	12	20	32	50
	Nombre minimal de lits inférieurs (g)	2	2	2	3	3	4
	u (cm) (h) avec *b* minimal	2,5	2,5	3,3	4,0	5,2	6,0
	(i) et avec un minimum de barres par lit de :	2	2	2	3	3	4
	(k) avec *b* supérieur à la plus grande des 2 valeurs : 1 mètre ou 1,5 h_1 et avec 10 barres par lit	1	1,5	2	2	3	4,
	$\frac{l_w + l_e}{l}$	0,25	0,30	0,4	0,5	0,55	0,60
u_1 (cm)	$\tau \leqslant \bar{\sigma}_b$ ou béton précontraint classes I et II (l)			(p) néant			
	$\bar{\sigma}_b < \tau \leqslant \sigma_j$ si tout en cadres (m)	1,5	2	2,5	3	3	3,5
	$\bar{\sigma}_b < \tau \leqslant \sigma_j$ si 40 % au moins en étriers ou épingles (n)			(p) néant			
	$\sigma_j < \tau \leqslant 5\bar{\sigma}_b$ Il faut au moins 40 % en étriers et épingles (o)	1,5	2	2,5	3	3	3,5

Key: (a) Rectangular beams; (b) Minimum b in cm; (c) Beams with heel; (d) minimum b in cm; (e) minimum b_o in cm; (f) Minimum h_o in cm; (g) Minimum number of lower layers; (h) With minimum b; (i) And with a minimum number of bars per layer equal to; (k) with b longer than the greater of the 2 values: 1 meter or 1.5 h_1, and with 10 bars per layer; (l) Or Class I and II prestressed concrete; (m) If all are in frames; (n) If at least 40% are in stirrups and pins; (o) At least 40% must be in stirrups and pins; (p) Zero.

Linear interpolation will be used to define the values of the terms b, u, and $(l_w+l_e)/l$ as a function of intermediate values of the ratio $(M_w+M_e)/2M_o$. To calculate u as a function of a heel width (b) intermediate between the minimum and maximum (1.00 or 1.5 h_1), the same thing is done.

Comment:

When two conditions are required for the same dimension, the most unfavorable should be met. In particular, the condition of web width, b_o, should be compatible with the condition of useful distance, u, when the flexural bars are placed in the web.

The rules and comments in Sect. 7.42 which relate to shattering resistance conditions also apply to the large and small beams considered in Sect. 7.51.

Comment:

One could take into account in calculating M_w and M_e the theoretical capping added by construction on low-continuity supports to take into account the non-calculated moments.

When the reinforcement has two bars per layer these should be as near the center as possible, with the distance between them only that necessary for good covering (3 cm, e.g., for concrete reinforcing steel and 3-5 cm for single wires).

Comment:

It is recommended that,

--The steel be concentrated toward the center of the beam, not placing larger-diameter steels in the corners,

--The number of layers of steel be increased,

--Part of the shear force be balanced by pins or stirrups, and not just frames near the surface of the concrete be used.

--A part of the lower reinforcement be extended on the support.

In the case of joints between small beams or boxes, the simple rules could be applied on the basis of the sections obtained by assuming that there is no joint.

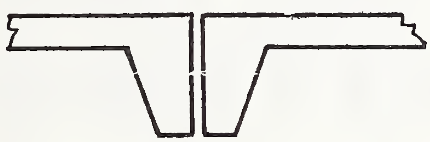


Figure 33

7.52 Temperature of the Concrete

The concrete temperature will be calculated by the general method given in Chapter IV.

Comment:

The use of the general method for particular cases has given the following results:

Temperatures in the heel of beams as a function of their width and the length of time exposed to fire.

These values were represented solely as a function of the width of the heel, since the other characteristics--frame, thickness of the slab, height h--have little effect.

The temperatures indicated were given in the axis of 3x3 cm^2 squares into which the section was cut.

Only the right half of the beam was represented, the other being deduced by symmetry.

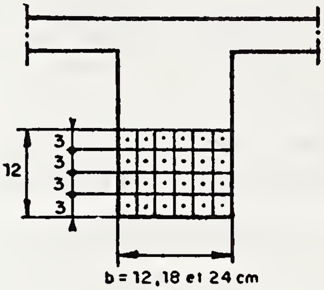


Figure 34. Case of b = 18.

	12		18			24			
30 mn	350	140	350	110	30	350	110	25	5
	370	160	360	130	50	360	130	50	30
	420	230	410	210	140	410	210	130	110
	550	430	550	410	360	550	410	360	350
60 mn	600	420	660	320	200	560	310	160	90
	630	470	590	370	260	590	360	230	170
	690	560	660	480	390	650	470	360	320
	780	710	760	660	610	760	650	590	570
90 mn	740	610	680	470	360	670	440	280	210
	760	650	710	530	430	700	500	370	300
	810	720	770	630	550	750	610	500	460
	880	820	850	770	730	850	760	705	680
120 mn	820	720	760	580	490	740	540	400	320
	840	760	790	640	560	780	600	480	420
	880	810	840	720	670	830	700	610	560
	940	890	920	850	820	910	830	780	760

One can deduce the temperature of the concrete at any point by linear interpolation from the temperatures shown in the table for neighboring points.

These temperatures were calculated for the running zone of the rain and cross beams. Near the supports of the cross beams on the main beams or walls and of the beams on walls or piers, the temperatures are different because the thermal diffusion is not the same.

In the absence of more precise justification, we assume a decrease in the temperatures indicated for the running parts at a distance of $2h_1$ from the support, the reduction coefficient being a maximum of 0.9, as indicated in the figure.

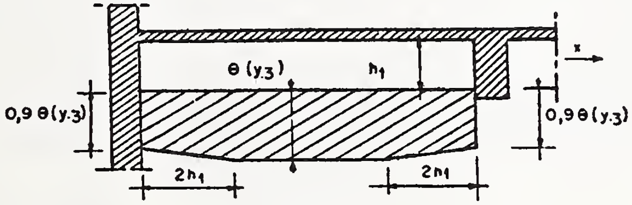
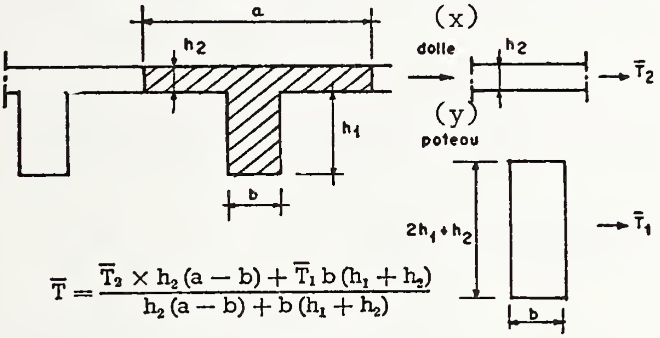


Figure 35

Average Concrete Temperatures. An approximate value on the high side for the average temperature of the concrete will be obtained by calculating the weighted average relative to the cross-sectional areas of a slab and a column.



$$\bar{T} = \frac{\bar{T}_2 \times h_2 (a - b) + \bar{T}_1 b (h_1 + h_2)}{h_2 (a - b) + b (h_1 + h_2)}$$

Figure 36

Key: (x) Slab; (y) Column.

For calculating $\bar{T}_1$ and $\bar{T}_2$ we therefore use the results given for slabs and columns in the earlier sections.

7.53 Other Justifications

Without applying the above rules one could calculate the large and small beams from information given in Chapter V, using the temperature values estimated according to Sect. 7.52.

We know that the half-sum of the ultimate moments on the supports added to the ultimate moment across the span balances the moment created by the applied loads.

Comment:

When the normal forces caused by expansion effects are to be taken into account, verification in compound bending will be necessary.

One will verify the length of the bars according to the moment curve corresponding to the failure diagram. The diagram of shear force will be deduced from it. This gives T_θ on the support once the direct transfer has been deducted.

For the shear force we make the two following verifications:

1. $$\tau_\theta = \frac{T_\theta}{0{,}875\, hb\,(\text{ou } b_0)} \leqslant 2{,}25\ \sigma_{j\theta}$$

[ou = or]

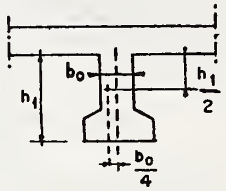


Figure 37

$\sigma_{j\theta}$ corresponds to the temperature θ measured at the point characterized by the distances $b_o/4$ and $h_1/2$, defined in Fig. 37.

2. We verify that the force capable of being resisted by a row of frames, stirrups, or pins (with a temperature θ at each bar calculated at a height $h_1/2$, and thus a stress $\sigma_{arg\theta}/1.15$) remains greater than that found in the cold with service stresses by applying to the latter the coefficient

$$\frac{1}{1.5}\ \frac{T_\theta}{\text{support in the cold}}$$

The rules given in Sect. 7.42, which relate to shattering resistance conditions, also apply to the beams and cross beams indicated in Sect. 7.53.

7.6 Slab Floors

Slab floors will be treated from the standpoint of resistance to fire like cast elements on continuous supports (Sect. 7.4).

7.7 Shell-Floors

7.71 Simple Rules

The F degrees, expressed in hours, of the exposure criteria (fireproofness and resistance to fire) are obtained for shell beams if the following specifications are met:

F (hr)	1/2	1	11/2	2	3	4
Min. thickness (cm)	10	11	13	15	20	25
u for flexural bars only (cm)	1	1,5	2	3	4 5	6

7.72 Temperatures

Depending on the case, and especially on the parts of the structure exposed to the fire, the considerations relative to beams or shells discussed above are to be applied.

7.73 Other Shell Beams

If we do not apply the above rules, we calculate the shell beams from the information in this chapter (VII), using the temperatures estimated from Sects. 7.3 and 7.5.

These shell beams will, however, be more than 8 cm thick, and when they must be fireproof, this thickness will be at least equal to that in Sect. 7.71.

Comment:

The difficulty of defining precise rules is due, partly, to the various possibilities of application of the fire which can, depending on the case, involve only the anchor, the intermediate part of the shell beam, the upper part of this beam, or the whole thing. It is also due to the uncertainties that usually exist with respect to the exact distribution of forces.

For example, one could

--Imagine unloading arches in analyzing the compressive stresses in the concrete and tensile stresses in the steel, taking the temperature into account.

--Imagine a load transfer which would not use a part of the shell exposed to the fire, in the case of shell beams on several levels.

Similarly, one could not take into account the action of the wind in a lintel of a wall with superposed openings for justification with respect to fire, to the extent that the lintels of the upper and lower floors may absorb this neglected force.

7.8 Floors of Hollow Ceramic or Concrete

7.81 Simple Rule

The F degrees, expressed in hours, of the exposure criteria are obtained by adding a thickness of protective coating, which is a function of the extent to which the coating remains in place (adhesion or bonding arrangements).

Comment:

The results of tests on an ordinary plaster coating were used to prepare a document, DTU No. 25.1, "Travaux d'enduit interieur en platre" [Studies on Interior Plaster Coatings], and for floors thicker than 14 cm and with a distance between beams of around 0.6 m or less to show the following values that should be used for this thickness:

F	1/2	1	1-1/2	2
Thickness, cm	0	1.0	1.3	1.6

These numbers can be used for floors with a cast covering of at least 4 cm over the hollow part. For durations of more than 1.5 hr the plaster suggested or special fire-protection plaster is advised since such plaster gives more consistent results, especially with regard to adherence.

7.82 Other Justifications

We can design floors, with or without an additional protective coating, by the methods discussed in Chapter V, the temperatures being determined from the information in Chapter IV.

Considering the problems posed by maintaining, during a fire, the bonds between the concrete, the hollow elements, and the protective coating when there is one, verification of a ceramic or concrete floor with hollow elements can be undertaken only for types of construction for which a similar model has been the object of a satisfactory study.

Comment:

In the absence of protective coatings, the hollow elements should be located in such a way as to reduce the width of the joints between hollow elements and cross beams. Except for special situations, a dimension of 1.5 cm should be considered as the maximum.

The main difficulty is calculation of the temperatures, taking into account the complexity of the formwork and the juxtaposition of different materials.

It would seem possible, in estimating the part played by the empty spaces, to substitute for an actual floor some equivalent floors for determining the temperatures.

For example, we would have:

--Floors with hollow elements whose coating thickness on the underface will be less than e_s will be made comparable to solid slab and cross-beam floors. The volume of the hollow filling will be represented by a thickness distributed equally over the total height of the soffit and the underface of the slab.

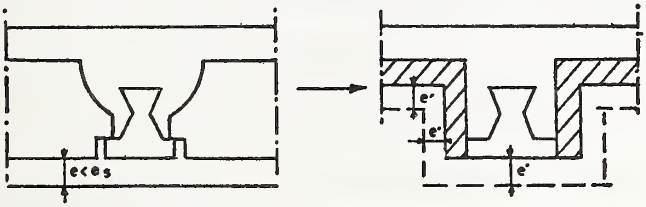


Figure 38

--In the particular case of hollow elements surrounding the beams, the actual thickness on the underface of the latter will be used and the rest of the hollow elements treated as above.

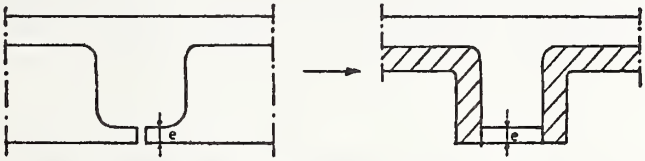


Figure 39

--Floors with hollow elements which have on their surface a protective coating of sufficient thickness e_s are to be considered as a solid slab whose thickness is equal to the total height of the floor (using the actual weight to calculate the forces).

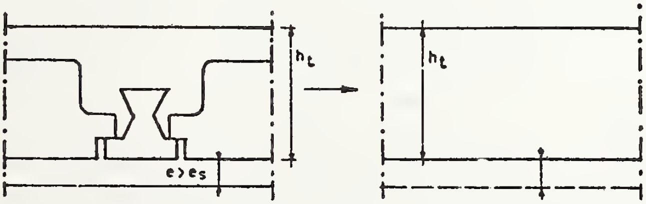


Figure 40

One will assume for ordinary plaster that the adequate thickness, e_s, is 1.0 cm.

The symbol e' is defined in Chapter VIII.

The simple rules for beams and cross-beams or slabs can be applied to this associated formwork.

One can also justify the stability of these floors by calculation, using the methods given in Chapter V.

If the rules for beams and cross-beams are not followed, no justification for the shear force can be provided.

Chapter VIII

ADDITIONAL PROTECTION

One can obtain the desired fire resistance times by adding material that is adapted and suitably bonded to the structure that it is protecting. The binding means or methods of adhesion should be studied in conclusive tests.

Where this addition of materials does not in itself provide adequate justification with respect to the exposure criteria, justification is done by the methods mentioned in Chapter V, the temperature being calculated by the methods in Chapter IV.

Comment:

The implementation of these protective measures should follow completely that described in the official reports of these tests.

The chief difficulty is in the calculation of the temperature because of the juxtaposition of different materials.

To simplify the problem, we can proceed as follows:

We replace a thickness of protective material by a thickness of concrete which has the same thermal protection properties.

A thickness, e , of material with coefficient λ' corresponds to a thickness e' of coefficient λ in such a way that $e' = e/(\lambda'/\lambda)$.

Of course, we have to take into account the special properties of the material concerned and of the behavior of its bond with the concrete in order to apply to the coefficient e' a correction factor.

Without a more precise justification, we have for the following materials, the list of which is not limiting, the values

1 cm of lime or cement mortar: 0.67 cm of concrete

1 cm of vermiculite: 2.5 cm of concrete

1 cm of asbestos cement: 2.5 cm of concrete

1 cm of ordinary plaster: 2.5 cm of concrete

We thus obtain an imaginary contour of the structure studied.

To this contour we apply the rules defined above for finding the value of the temperature of the materials or for observing the simple rules.

A false ceiling can be taken into account if it has been studied in an approved laboratory.

If the false ceiling to be used conforms in its arrangement and the method of attachment to the test conditions, it will be a factor in the fire resistance time of the whole and to the extent that the elements which may be incorporated in it (ducts, lighting fixtures, etc.) have the same characteristics of resistance and insulation.

One could, however take into account a false ceiling only with the agreement of the chief engineer who thus becomes responsible for ensuring its maintenance.

Appendix I.

PROGRAM FOR CALCULATING TEMPERATURES

One of the programs that can be used is described below.

I.1 Breakdown of the Structural Element

The structural element considered is enclosed in a rectangular table of the smallest possible dimensions, observing the following conditions:

The element considered has the hot part below and the cold part above.

The temperatures of the small squares surrounding the element are those of the wall (hot or cold) and are calculated by the program.

The small squares should have the following values:

2 for those of the cold part

1 for those of the hot part

0 for those of the element considered and for those surrounding this element

As a result:

1. The number of rows, N, is always equal to N 1 + 4 where N 1 is the number of squares equal to the element height

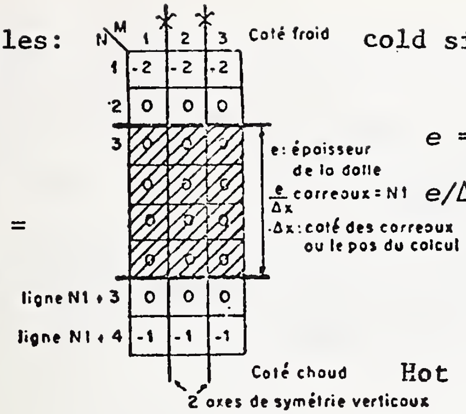
2. The number of columns, N, is equal to

   3 for slabs
   M 1 + 4 for columns
   M 1 + 2 for beams

where M 1 is the largest number of squares across the width of the structural element.

On the other hand, the presence of the 2 axes of vertical symmetry makes the temperatures of the first column equal to those of the second and the temperatures of the last column to those of the next-to-last. They are thus never indicated when the data are entered.

Examples:



cold side

Slab

e = slab thickness

[ligne = row]

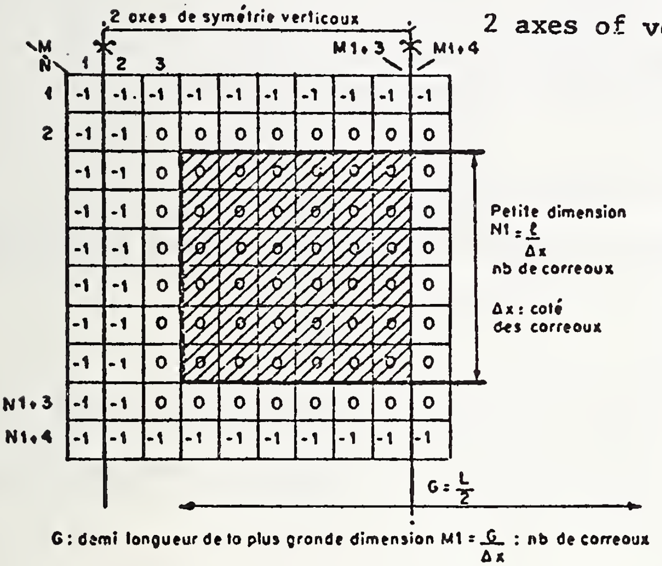
$e/\Delta x$ squares = N 1

Δx = side of squares or step of calculation

Hot side

2 axes of vertical symmetry

Column (surrounded by fire)



2 axes of vertical symmetry

Small dimension

No. of squares

Δx: side of squares

G: half-length of largest dimension

M 1 = $G/\Delta x$: No. of squares

I. 2 Data Entry

1st line λ, c, ρ, Δx, β

where λ (DAO) is the thermal conductivity of concrete at 0^oC, 1.4 kcal/hr in ^oC

c (C) is the specific heat of concrete, 0.22 kcal/kg ^oC

ρ (R_Q) is the bulk density of concrete, 2400 kg/m^3

Δx (DELX) is cell side, cm (integer or not)

β (BETA) is a coefficient involved in the global exchange coefficient of the oven

2nd line L, M, N, ICOD

where L is the number of time intervals at which we want to print out the results
M is the number of columns in the table
N is the number of rows in the table
ICOD = 0 if we do not want to take out the exact results of temperatures in the element but their approximate values
= 1 if we want to take out the temperatures calculated at the center of each cell in a particular zone

3rd line: 1st time requested

4th line: 2nd time requested

L + 2nd line: L-th time requested

Note: The time required is indicated by the computer as a decimal fraction of an hour (15 min = 0.25, 30 min = 0.5, 1 hr = 1, 1 hr and 30 min = 1.5, etc.)

Next line: Two cases are to be distinguished:

If ICOD = 0, we do not want to print the exact temperatures and we go to the next line

If ICOD = 1, write N1, N2, M1, M2

where N1 and N2 are the numbers in the first and last lines of the table of results to be printed
M1 and M2 are the first and last columns of the table of result printed

Next lines:

N lines IN, JD, JF containing each (or N1 +4)

where IN is the number of the line (1 to N1 + 4)
JD and JF are the numbers between the two axes of symmetry of the first and the last cell of line IN or they take zero values. If there is no zero in line IN, write IN, 0, 0.

Next lines:

M - 2 lines corresponding to columns located between the two axes of symmetry

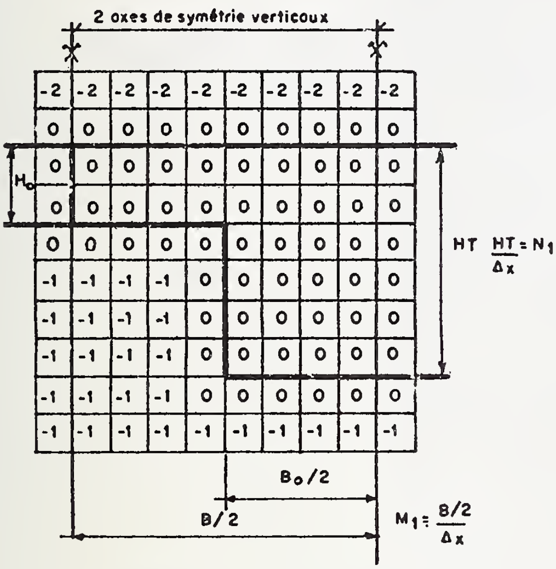
JN, IA

JN is the number of the column from 2 to M - 1

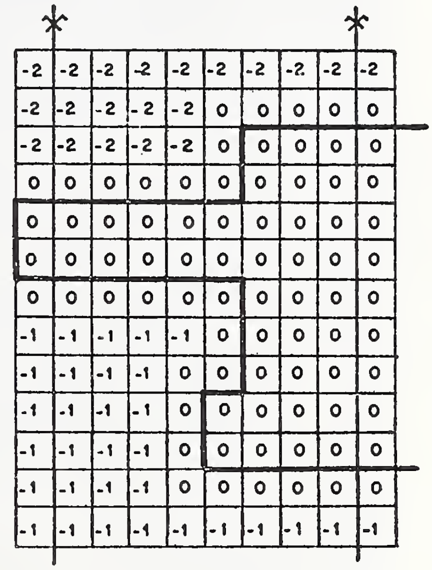
IA is the number of the last square of this column JN or the squares with the value -2

Temperatures	0 à 49	50	51 à 149	150	151 à 249	250	etc.	850 à 949	950	> 950
The machine prints	0	●	1	●	2	●		9	●	●

2 axes of vertical symmetry



c) Beam



d) Beam with heel and rim

I.3 Printout of Results

For each series of results:

--Temperature of the concrete after Δt(DELT) minutes of fire

--AK2 = ; AKK2 = ; DELT =

AKK2 = global coefficient of theoretical exchange permissible in the calculation taking into account the dimension of the squares (2xDA/DELX)

AK2 = global coefficient of exchange of the furnace taken into account in the calculation

If AK2 > AKK2: the program calculates with the value of AKK2 and a slight error can be ignored.

TMB is the average temperature of the concrete
TPF is the average temperature of the cold wall
TPC is the average temperature of the hot wall

Output of additional results ultimately requested: temperature at the center of each square

Output of temperatures of the element considered with the convention given in the table on the previous page.

When the temperature TPF exceeds 140°C, the machine prints the corresponding temperature; likewise when TPFM exceeds 180°C.

2. Program

2.1 Definition of Main Variables of the Program

DELX Δx, side of the cell in cm (usually 1, 2, or 3 cm)
TPF avg. temperature of cold wall
TPC avg. temperature of hot wall
DAO λ_0, conductivity of material at 0°C
DA λ, conductivity of material at t°C
DELT Δt, temperature deviation
C c, specific heat
RO p, bulk density
BETA β, coefficient ($T_m = \beta$.TC)
TC 345 $\log_{10}(8t + 1)$ oven temperature
AK1 global coefficient of exchange of furnace on cold side
AK3 global coefficient of exchange of furnace, calculated
AK2 smallest number between AK3 and 2xDA/DELX
AKK2 2xDA/DELX
TPFM maximum temperature of cold wall
TMB average temperature of concrete (arithmetic mean)

2.2 List of instructions

```
      T6=T6+T1(I,J)
      DF=(T1(I,J)+(T3+T4)/2.)/2.
      IF(DF-TPFM)144,146,146
146   TPFM=DF
144   GO TO 340
200   IBIP=IBIP+1
      SOM=2.*TC*DELX+(DA/AK2-DELX/2.)*(T3+T4)
      T1(I,J)=SOM/2./(DA/AK2+DELX/2.)
      T7=T7+T1(I,J)
340   T5=T5+T1(I,J)
1000  CONTINUE
      DO 6 I=1,N
      T1(I,1)=T1(I,2)
6     T1(I,N)=T1(I,N3)
      IF(ISSP-ISSB)129,129,128
128   TPF=(T6+T9)/(ISSP+ISSB)
129   TPC=(T7+T10)/(IBIP+IBIB)
      IF(TPF-140.)951,900,900
900   DH=DELT
      IF(DH-DH1)902,902,951
902   WRITE(6,901)DH,TPF
901   FORMAT(2X,'TEMPS A PARTIR DUQUEL LA TPF DEPASSE 140=',
     2F5.0,'MINUTES',5X,'TPF=',F5.0,/)
      DH1=DH
951   IF(TPFM-180.)958,959,959
959   DH=DELT
      IF(DH-DH2)961,961,958
961   WRITE(6,903)DH,TPFM
903   FORMAT(2X,'TEMPS A PARTIR DUQUEL LA TPFM DEPASSE 180=',
     2F5.0,'MINUTES',2X,'TPFM=',F5.0,/)
      DH2=DH
958   IF(DELT-R(III))99,600,600
600   WRITE(6,700)DELT
700   FORMAT(/,15X,'TEMPERATURES DU BETON APRES',
     21X,F5.0,'MINUTES DE FEU',//)
      WRITE(6,9)AK2,AKK2,DELT
9     FORMAT(15X,'AK2=',F4.0,'AKK2=',F4.0,'DELT=',F5.0,/)
      WRITE(6,710)TMB,TPF,TPC,TPFM
710   FORMAT(15X,'TMB =',F6.0,//,15X,'TPF =',F5.0,//,
     215X,'TPC =',F6.0,//,15X,'TPFM=',F6.0,/)
      IF(ICOD)711,711,715
715   DO 714 I=N1,N2
      WRITE(6,720)(T1(I,J),J=M1,M2)
720   FORMAT(20(1X,F5.0))
714   CONTINUE
711   DO 27 I=1,N
      DO 26 J=1,M
      IF(T1(I,J))1100,1200,1200
1100  IP(J)=1077952576
      GO TO 26
1200  IF(T1(I,J)-951)1300,1500,1500
1500  IP(J)=1547714524
      GO TO 26
1300  P=T1(I,J)/100.-INT(T1(I,J)/100.)
      IF(P-0.49)1600,1600,1700
1700  IF(P-0.50)1800,1800,1900
1800  IP(J)=1262501952
      GO TO 26
1600  NP=INT(T1(I,J)/100.)
      GO TO 24
1900  NP=INT(T1(I,J)/100.)+1
24    IF(NP-9)1610,1610,1500
1610  IP(J)=-247447423+(NP-1)*16777216
26    CONTINUE
```

2.2 List of instructions (cont'd)

```
      Y3=AMIN1(T1(I-1,J+1),T1(I+1,J+1),T1(I-1,J-1),T1(I+1,J-1))
      IF(Z3)59,60,60
59    IF(T1(I-1,J))100,85,85
60    IF(T1(I,J+1))110,120,120
120   IF(T1(I,J-1))110,140,140
140   TE=T1(I-1,J)
      T3=T1(I+1,J)
      T4=T3
      GO TO 141
85    IF(T1(I,J+1))150,160,160
150   IF(T1(I+1,J))110,180,180
180   TE=T1(I,J+1)
      T3=T1(I,J-1)
      T4=T3
      GO TO 141
160   IF(T1(I+1,J))210,220,220
210   IF(T1(I,J-1))110,240,240
240   TE=T1(I+1,J)
      T3=T1(I-1,J)
      T4=T3
      GO TO 200
220   TE=T1(I,J-1)
      T3=T1(I,J+1)
      T4=T3
      GO TO 141
110   IAS=IAS+1
      GO TO 1000
60    IF(Y3)260,1000,1000
260   IF(T1(I-1,J+1))278,280,280
278   TE=T1(I-1,J+1)
      T3=T1(I,J-1)
      T4=T1(I+1,J)
      GO TO 145
280   IF(T1(I+1,J+1))290,300,300
290   TE=T1(I+1,J+1)
      T3=T1(I-1,J)
      T4=T1(I,J-1)
      GO TO 200
300   IF(T1(I-1,J-1))310,320,320
310   TE=T1(I-1,J-1)
      T3=T1(I,J+1)
      T4=T1(I+1,J)
      GO TO 141
320   TE=T1(I+1,J-1)
      T3=T1(I,J+1)
      T4=T1(I-1,J)
      GO TO 200
141   IF(TE+2.)145,145,200
145   IBSP=IBSP+1
      T1(I,J)=(DA/AK1-DELX/2.)*(T3+T4)/2./(DA/AK1+DELX/2.)
```

2.2 List of instructions (cont'd)

```
      7JOB GO
      DIMENSION T1(50,100),T2(50,100),TP(100),R(8),H(8)
      CALL FDATAS(DAO,C,RO,DELX,BETA)
      CALL IDATAS(L,M,N,ICOD)
      DO 1 LL=1,L
      CALL FDATAS(H(LL))
 1    R(LL)=H(LL)*60
      IF(ICOD)12,12,11
 11   CALL IDATAS(N1,N2,M1,M2)
 12   DELX=DELX/100
      M3=M-1
      DH1=1300
      DH2=1300
      TPFM=0
      DO 10 I=1,N
      DO 10 J=2,M3
 10   T1(I,J)=-1.
      IB=0
      DO 15 L=1,N
      CALL IDATAS(IN,JD,JF)
      IF(JF*JD)15,15,16
 16   IB=IB+JF-JD+1
      DO 15 J=JD,JF
      T1(IN,J)=0.
 15   CONTINUE
      DO 20 J=2,M3
      CALL IDATAS(JN,IA)
      IF(IA)20,20,21
 21   DO 20 I=1,IA
      T1(I,JN)=-2.
 20   CONTINUE
      TPF=0.
      TMC=0
      IT=1
      DA=DAO
      NR=1
      DELT=0.
 50   DELT=600.*C*RO*(DELX)**2/4./DA+DELT
      TS=0.
      T6=0.
      T7=0.
      FAS=0.
      ICSP=0
      IBIP=0
      TF=0.
      AKI=8.*(TPF-TF)**0.3
      AK1=AMAX1(AKI,0.1)
      TC=545.*ALOG10(8.*DELT+1.)
      A=(TPC+273.)/(BETA*TC+273.)
      B=(BETA*TC+273.)/1000.
      CE=(BETA*TC-TPC)/(TC-TPC)
      IF(CE)4,4,2
 4    CE=1.
 2    AK3=5.+40.*CE*(B)**3*(1.+A+A**2+A**3)
      AK2=AMIN1(AK3,(2.*DA)/DELX)
      AKK2=(2.*DA)/DELX
      DO 5 I=1,N
      T1(I,1)=T1(I,2)
 5    T1(I,M)=T1(I,M3)
      IF(NR-1)56,56,54
 54   DO 1000 I=2,N
      DO 1000 J=2,M3
      IF(T1(I,J))1000,58,58
 58   Z3=AMIN1(T1(I-1,J),T1(I,J+1),T1(I+1,J),T1(I,J-1))
```

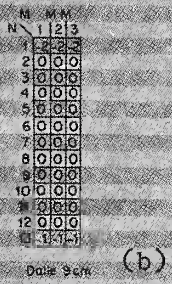
2.2 List of instructions (cont'd)

```
      WRITE(6,830)(LP(J),J=1,M)
830   FORMAT(2X,123A1)
27    CONTINUE
      I1=I1+1
      IF(I1-L)55,55,3000
55    NR=NR+1
56    T8=0.
      T9=0.
      T10=0.
      IBSB=0
      IBIB=0
      DO 2000 I=2,[illegible]
      DO 2000 J=2,M3
      IF(T1(I,J))2000,350,350
350   Z3=AMIN1(T1(I-1,J),T1(I,J+1),T1(I+1,J),T1(I,J-1))
      Y3=AMIN1(T1(I-1,J-1),T1(I+1,J+1),T1(I-1,J+1),T1(I+1,J-1))
      IF(Z3)2000,420,420
420   IF(Y3)2000,430,430
430   D=(T1(I+1,J)+T1(I-1,J)+T1(I,J+1)+T1(I,J-1))[illegible]
      T2(I,J)=D
      T8=T8+T2(I,J)
      IF(J-2)432,432,429
429   IF(J-M3)431,432,432
431   Z1=AMIN1(T1(I-2,J),T1(I+2,J),T1(I,J-2),T1(I,J+2))
      Y1=AMIN1(T1(I-2,J-2),T1(I+2,J-2),T1(I-2,J+2),[illegible]
      IF(Z1+2.)440,440,450
450   IF(Z1+1.)455,455,460
460   IF(Y1+2.)440,440,470
470   IF(Y1+1.)455,455,2000
432   Z2=AMIN1(T1(I-2,J),T1(I+2,J))
      IF(Z2+2.)440,440,433
433   IF(Z2+1.)455,455,2000
440   T9=T9+T2(I,J)
      IBSB=IBSB+1
      GO TO 2000
455   T10=T10+T2(I,J)
      IBIB=IBIB+1
2000  CONTINUE
      TMB=T8/(IB-IBSB-IBIB-IAS[illegible]
      DA=DA0-TMB/1000.
      DO 1650 I=3,[illegible]
      DO 1650 J=2,M3
      IF(T1(I,J))1650,950,950
950   IF(T2(I,J))1650,960,960
960   T1(I,J)=T2(I,J)
1650  CONTINUE
      IF(NR-1)56,56,25
3000  STOP
      END
```

2.3 Examples

2.31. Slab

```
L.0001  /DATA
L.0002  1.4 0.22 2400 1 0.85
L.0003  6 3 13 1
L.0004  0.5
L.0005  1
L.0006  1.5
L.0007  2
L.0008  2.5
L.0009  3
L.0010  2 12 1 3
L.0011  1 0 0
L.0012  2 2 2
L.0013  3 2 2
L.0014  4 2 2
L.0015  5 2 2
L.0016  6 2 2
L.0017  7 2 2
L.0018  8 2 2
L.0019  9 2 2
L.0020  10 2 2
L.0021  11 2 2
L.0022  12 2 2
L.0023  13 0 0
L.0024  2 1
```




```
(a)        TEMPERATURES DU BETON APRES   30.MINUTES DE FEU

           AK2= 69.AKK2=245.DELT=  30.

           TMB =  177.

           TPF =   25.

           TPC =  559.

           TPEM=   25.

 24.   24.   24.
 26.   26.   26.
 35.   35.   35.
 52.   52.   52.
 80.   80.   80.
122.  122.  122.
180.  180.  180.
258.  258.  258.
359.  359.  359.
484.  484.  484.
633.  633.  633.

000
000
000
111
111
111
222
333
444
555
666
```

Key: (a) Concrete temperatures after 30 minutes of fire; (b) Slab 9 cm.

(a) Concrete temperatures after after 61 minutes of fire

```
(a) TEMPERATURES DU BETON APRES   61.MINUTES DE FEU

    AK2= 63.AKK2=213.DELT=  61.

    TMB =  334.

    TPF =  119.

    TPC =  721.

    TPFM=  119.

111.  111.  111.
126.  126.  126.
150.  150.  150.
185.  185.  185.
231.  231.  231.
291.  291.  291.
363.  363.  363.
450.  450.  450.
549.  549.  549.
660.  660.  666.
783.  783.  783.

111
111
111
222
222
333
444
444
555
777
888
```

(b) Time after which the TPF exceeds 140 is 63 minutes (TPF=141); (c) Time after which the TPF exceeds 180 is 64 minutes (TPFM = 180); (d) Concrete temperatures after 90 minutes of fire.

```
(b) TEMPS A PARTIR DUQUEL LA TPF DEPASSE 140=  63.MINUTES   TPF= 141.
(c) TEMPS A PARTIR DUQUEL LA TPFM DEPASSE 180=  64.MINUTES  TPFM= 180.

(d) TEMPERATURES DU BETON APRES   90.MINUTES DE FEU

    AK2= 54.AKK2=194.DELT=  90.

    TMB =  432.

    TPF =  194.

    TPC =  795.

    TPFM=  194.

181.  181.  181.
208.  208.  208.
243.  243.  243.
286.  286.  286.
336.  336.  335.
400.  400.  400.
472.  472.  472.
554.  554.  554.
644.  644.  644.
743.  743.  743.
849.  849.  849.

222
222
222
333
333
444
555
666
666
777
888
```

(a) Concrete temperatures after 120 minutes of fire

```
(a)   TEMPERATURES DU BETON APRES  120.MINUTES DE FEU

      AK2= 46.AKK2=180.DELT= 120.

      TMB =  498.

      TPF =  245.

      TPC =  844.

      TPFM=  245.

226.  226.  226.
264.  264.  264.
307.  307.  307.
356.  356.  356.
412.  412.  412.
475.  475.  475.
545.  545.  545.
622.  622.  622.
706.  706.  706.
796.  796.  796.
891.  891.  891.

222
333
333
444
444
555
555
666
777
888
999
```

(b) Concrete temperatures after 151 minutes of fire

```
(b)   TEMPERATURES DU BETON APRES  151.MINUTES DE FEU

      AK2= 41.AKK2=171.DELT= 151.

      TMB =  545.

      TPF =  279.

      TPC =  879.

      TPFM=  279.

256.  256.  256.
302.  302.  302.
351.  351.  351.
405.  405.  405.
464.  464.  464.
528.  528.  528.
597.  597.  597.
672.  672.  672.
751.  751.  751.
835.  835.  835.
923.  923.  923.

333
333
444
444
555
555
666
777
888
888
999
```

(a) Concrete temperatures after 180 minutes of fire.

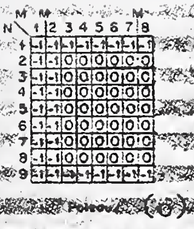
(a)

```
TEMPERATURES DU BETON APRES  180.MINUTES DE FEU

AK2= 37.AKK2=165.DELT= 180.

TMB =  577.

TPF =  300.

TPC =  905.

TPFM=  300.

275.  275.  275.
326.  326.  326.
380.  380.  380.
438.  438.  438.
499.  499.  499.
564.  564.  564.
633.  633.  633.
706.  706.  706.
783.  783.  783.
864.  864.  864.
947.  947.  947.

333
333
444
444
555
666
666
777
888
999
999
```

Note: The examples given do not correspond to likely dimensions for reinforced concrete structures. They are intended to show the use of the program.

2.32 Column

```
L.0001  /DATA
L.0002  1.4 0.22 2400 1 0.65
L.0003  3 7 9 1
L.0004  0.5
L.0005  1
L.0006  1.5
L.0007  2 3 5 6
L.0008  1 0 0
L.0009  2 3 6
L.0010  3 3 6
L.0011  4 3 6
L.0012  5 3 6
L.0013  6 3 6
L.0014  7 3 6
L.0015  8 3 6
L.0016  9 0 0
L.0017  2 0
L.0018  3 0
L.0019  4 0
L.0020  5 0
L.0021  6 0
```



(a)

```
TEMPERATURES DU BETON APRES   31 MINUTES DE FEU

AK2= 29.AKK2=151.DELT= 752.

TMB = 693.

TPF =   0.

TPC = 699.

TPFM =   0.

  0.  727.  700.  695.
727.  687.  659.  644.
709.  662.  627.  609.
702.  653.  616.  597.
708.  662.  627.  609.
727.  687.  659.  644.
  0.  727.  700.  695.

 07777
 77766
 77666
 77666
 77666
 77766
 07777
```

(b)

```
TEMPERATURES DU BETON APRES   60 MINUTES DE FEU

AK2=  8.AKK2=126.DELT=  60.

TMB =  771.

TPF =    0.

TPC =  738.

TPFM =   0.

  0.  804.  794.  789.
804.  788.  776.  770.
795.  777.  765.  758.
792.  774.  761.  755.
795.  777.  765.  758.
804.  788.  776.  770.
  0.  804.  794.  789.

 03888
 88888
 88888
 88888
 88888
 88888
 08888
```

Key: (a) Concrete temperatures after 31 minutes of fire; (b) Concrete temperatures after 60 minutes of fire; (c) Column.

(a) Concrete temperatures after 92 minutes of fire.

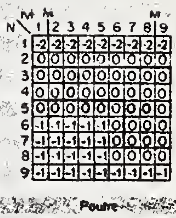
(a)

```
TEMPERATURES DU BETON APRES   92.MINUTES DE FEU

AK2= 92.AKK2= 92.DEET= 92.

TMB = 938.

TPF = 0.

TPC = 972.

TPFM= 0.

  0.  939.  988.  988.
 955.  966.  950.  943.
 939.  952.  926.  913.
 906.  947.  918.  903.
 938.  952.  926.  913.
 556.  966.  950.  943.
  0.  988.  988.  988.

 0****
 ***99
 **999
 *9999
 **999
 ***99
 0****

STOP   00000
```

2.33. Beam

(b) Time after which TPF exceeds 140 is 12 minutes (TPF = 147); (c) Time after which TPF exceeds 180 is 13 minutes (TPFM = 191); (d) Concrete temperature after 30 minutes of fire; (e) Beam.

```
L.0001  /DATA
L.0002  1.4 0.22 2400 1 0.55
L.0003  3 9 9 1
L.0004  0.5
L.0005  1
L.0006  1.5
L.0007  2 6 6 5
L.0008  1 0 0
L.0009  2 2 8
L.0010  3 2 8
L.0011  4 2 8
L.0012  5 2 8
L.0013  6 6 8
L.0014  7 6 8
L.0015  8 6 8
L.0016  9 0 0
L.0017  2 1
L.0018  3 1
L.0019  4 1
L.0020  5 1
L.0021  6 1
L.0022  7 1
L.0023  8 1
```



(e)

(b)

```
TEMPS A PARTIR DUQUEL LA TPF DEPASSE 140 = 12 MINUTES   TPF = 147.
```

(c)

```
TEMPS A PARTIR DUQUEL LA TPFM DEPASSE 180 = 13 MINUTES   TPFM = 191.
```

(d)

```
TEMPERATURES DU BETON APRES   30 MINUTES DE FEU

AK2= 54.AKK2=277.DEET= 30.

TMB = 514.

TPF = 390.

TPC = 837.

TPFM= 420.

 352.  329.  314.
 414.  388.  370.
 515.  470.  442.
 673.  569.  525.
 720.  641.  601.
 758.  700.  671.
   0.  758.  742.

 444444333
 555444444
 665555544
 777777655
      7666
      8777
      0877
```

(a)

```
          TEMPERATURES DU BETON APRES   61.MINUTES DE FEU

          AKZ= 41.AKKZ=151.DELT=  61.

          THB =  644.

          TPF =  491.

          TPC =  758.

          TPFM=  503.

441.  430.  422.
537.  524.  514.
648.  622.  606.
788.  720.  692.
848.  788.  761.
875.  836.  816.
  0.  875.  864.

555544444
666655555
777776666
888888777
    8888
    9888
    0999
```

(b)

```
          TEMPERATURES DU BETON APRES   90.MINUTES DE FEU

          AKZ= 30.AKKZ=141.DELT=  90.

          THB =  693.

          TPF =  521.

          TPC =  815.

          TPFM=  530.

466.  456.  450.
576.  564.  556.
697.  673.  659.
845.  779.  752.
910.  852.  826.
937.  900.  882.
  0.  937.  927.

555555544
666666665
777777777
888888888
    9988
    9999
    0999
```

Key: (a) Concrete temperatures after 61 minutes of fire; (b) Concrete temperatures after 90 minutes of fire.

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16. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)*

This method provides a means for predicting, by calculation, the resistance to fire of a reinforced or prestressed concrete element of construction, in accordance with 1959 French directives.

The method is useful in allowing builders to design structures which show the degree of fire resistance required by the various French construction regulations in force. According to those regulations, only a test furnishes legal proof of fire resistance.

French researchers hope this first step will lead to the acceptance in France of fire resistance calculations as legal proof of satisfactory resistance.

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