

RESISTANCE AND STABILITY OF COLUMNS IN FIRE

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RESUME

Cette étude a pour but d'étudier la stabilité et la sécurité pour le renforcement des poteaux en béton armé contre le feu.

La technique de simulation de Monte Carlo est utilisée pour évaluer le risque de fissuration en tenant compte de l'effet de distribution de la température.

Les éléments exposés apparaissent affectés par les variations dont les paramètres peuvent être évalués comme le temps d'exposition, la minceur de la section du poteau et les rapports d'acier et le béton. Une possibilité de réduire cette sensibilité est proposée en cours de cette recherche, qui a été développée au laboratoire de thermique au département de Génie Civil à l'IGCMO, Université de technologie-Houari Boumediene d'Oran.

Abstract

This study deals with the safety of reinforced concrete columns designed for stability against fire. A Monte Carlo simulation technique is used to assess the probability of failure by taking into account the temperature on statistical distributions. The reliability of columns exposed to fire appears particularly affected by variations in the parameters such as the design exposure time, slenderness, and steel ratios. A possibility of reducing this sensitivity is proposed.

KEYWORDS: COLUMNS, STRUCTURAL DESIGN, THERMAL MECHANICS.

RESEARCH SIGNIFICANCE

The research presented herein is motivated by the need for a probabilistic assessment of the risk of collapse of reinforced concrete structures exposed to fire. Earlier research was conducted by Lie et al. Terro Huang and al., and Ellingwood et al., among others, with the aim of predicting the behavior of concrete frames under fire exposure, without including the random nature of phenomena. Some of these results are used herein for perfecting a structural model of concrete columns; these results are also looked at for their experimental and theoretical issues in comparison to the research presented in this study. In addition, a probabilistic analysis is achieved for testing the safety of columns designed in compliance with codes.

INTRODUCTION

The expansion effects in concrete during a fire may be neglected in the structure as a whole as long as the concrete is free to dilate unhindered. In contrast, temperature increase can considerably reduce the mechanical and thermal characteristics of structural reinforced concrete elements. In fact, several

phenomena combine together:

1. The stresses due to temperature gradients ;
2. The possible second order effects increase due to the curvature by the same gradients ;
3. The decrease of material properties (resistance and rigidity loss, irreversible damage); and
4. The possible spalling of concrete, which reduces the concrete resistance area and directly exposes the reinforcement to fire.

Gas Temperature (°C)

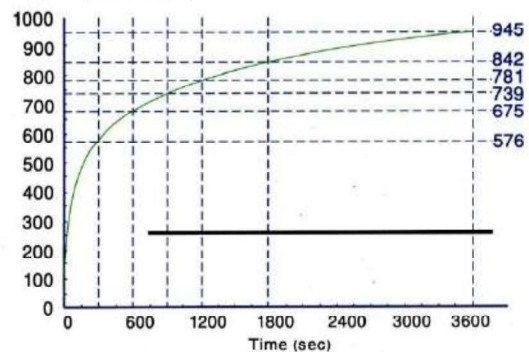


Fig. 1 — Standard curve ISO834.

Most of these phenomena are random by nature. The measurements carried out to define material properties are widely scattered. Nevertheless, for the purpose of design, and in particular, for reinforced concrete columns, recommendations and calculation methods must ensure a minimal, but real, safety margin. The latter is generally provided by taking into account, on an empirical basis, either explicitly or implicitly, the consequent uncertainties in design codes such as CEB code, EC Eurocode, ACI Guides, or FB87 Rules. Nevertheless, improvements in fire design calculation methods for columns have been proposed recently without referring to this random aspect; therefore, there is a need to assess the reliability of elements designed in compliance with these codes and to propose modifications where necessary.

1. THERMAL AND MECHANICAL BEHAVIOR MODEL

The fire resistance of reinforced concrete columns has been calculated in two stages: first by determining the temperature field after thermal loading, then by calculating the load-bearing capacity while taking into account the effects of temperature on the material properties in each point of the structure. The column is subjected to heat exchanges with the surrounding gas in a compartment. The temperature of this gas is assumed to vary over time, according to the formula taken from ISO 834 code.

$$\Theta_f(t) = \Theta_f(t_0) + 345 \log_{10}(8t + 1) \quad (1)$$

Where $\theta_f(t)$ and $\theta_f(t_0)$ are the temperatures expressed in degrees C in the compartment at time t and at initial time t_0 , expressed in min. The curve obtained from this formula (fig.1) is similar to the ASTM E 119 curve and the CAN4-S101 curve.

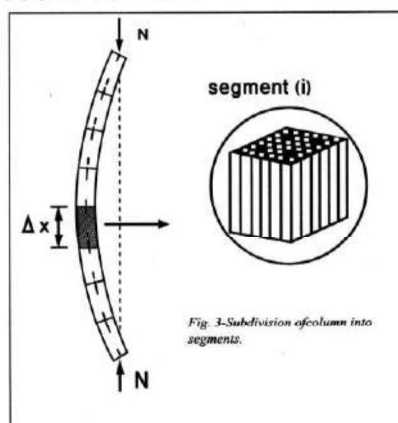


Fig. 2 — Fire resistance calculation procedure.

The exposure time is divided into short intervals, during which the temperature field at each time interval in the structure is determined by use of the CESAR-LCPC finite element program.

A specific computation of thermal variables (CTV) module has been linked to this program to take into account the variations of thermal and mechanical properties of materials in relation to the temperature. The scheme of this procedure is shown in fig.2.

The column is subdivided into segments as described in Fig.3, which are subsequently divided into rectangular finite elements. This model is used for thermal as well as mechanical calculations.

The variations of thermal and mechanical properties of materials occur because of deterioration due to elevated temperatures: water loss in concrete; mineralogical transformations, internal microcracking and modification of steel microstructure. The structural model integrates these variations for the column as a whole through well established relationships based on experimental results. The risk of concrete spalling, however, has not been taken into account.

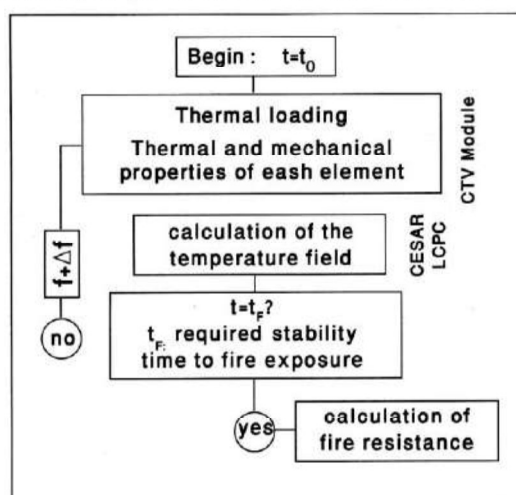


Fig. 3 — Subdivision of column into segments.

The eccentricity –curvature diagrams of segments are computed on the basis of these relationships for each stage of loading assuming that Navier's hypothesis is relevant.

The deformation of the median line of the column is then determined according to boundary and continuity conditions.

The load bearing capacity is finally reached by successive load increases until the equilibrium divergence occurs.

In addition to Navier's hypothesis, the following assumptions are made:

1. The concrete tensile strength is ignored.
2. The shrinkage effect is negligible ;and
3. The variation of length due to thermal expansion is ignored.

The creep effect is accounted for by introducing an additional creep eccentricity, as recommended by the CEB Model Code.

2. VALIDATION OF FIRE RESISTANCE CALCULATION MODE.

To test the validity of the model, the results have been compared with those obtained experimentally and theoretically by Li et al. Other similar comparisons were made with the CEB and the Eurocode EC2 in Reference 14. Columns are assumed to be hinged at both ends uniformly exposed to fire on their four lateral sides, so that the temperature field in across section remains constant along the test column. The thermal loading complies with the standard conditions for temperature rise.

The test results used by Lie at al. to validate the analytical model correspond to average values chosen

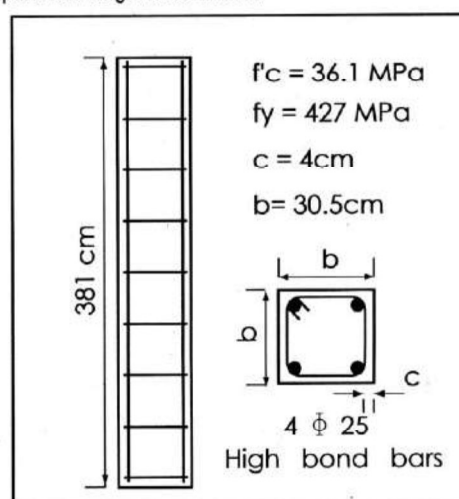


Fig. 4—Characteristics of studied columns.

from a large series of tests carried out by the laboratory, including more than 40 columns. The same experimental results are considered as follows. The mechanical and geometrical parameters of the columns are indicated in fig4. Changes in the temperature versus exposure time for columns surrounded by the fire are presented in Fig5. For the three cases considered (depending on depth u from the concrete surface), the results correlate well with experimental temperatures that are also close to those obtained by the Lie model.

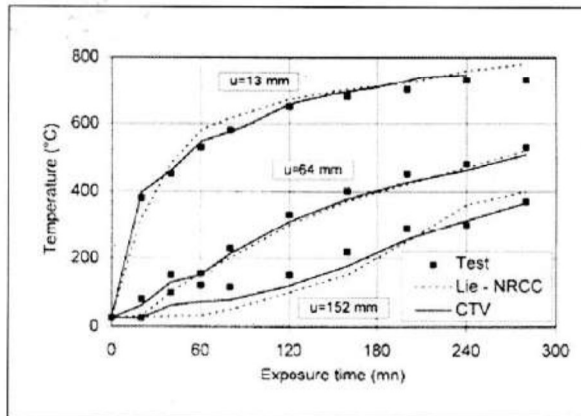


Fig. 5 — Temperature at various points of transverse section.

As can be seen in Fig. 6, the progress of the computed fire resistance versus exposure time is similar for both the Lie model and the CTV model. It can be noted that both models lead to the same fire resistance after 210 min exposure time.

3. RELIABILITY OF COLUMNS EXPOSED TO FIRE

The technical difficulty of assessing the reliability of reinforced concrete columns exposed to fire lies in the large number of variables involved in the calculation of their temperature – dependent strengths and in the need to combine several programs, as previously explained. Under these conditions, the Monte Carlo simulation technique appears to be a powerful and responsive tool.

The failure probability P_f is defined as the probability for the load S applied to the column during fire exposure to be greater than its fire resistance R (bearing capacity) at exposure time.

$$P_f = P(S \geq R) = P(R - S \leq 0) = P(E \leq 0) \quad (2)$$

The standard deviation σE and mean value of the variable E R-C are estimated from a sample of values generated by the Monte Carlo simulation technique.

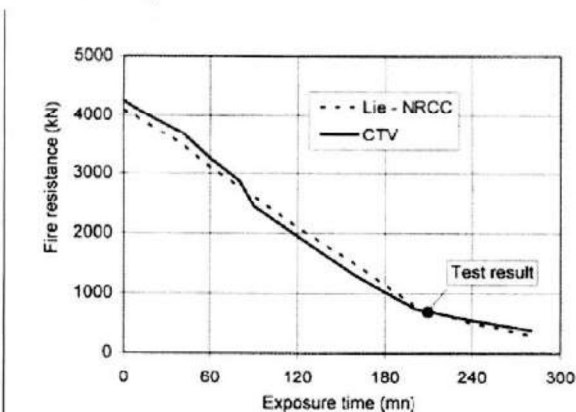


Fig.6 — Comparison of fire resistances.

For the purpose, the variables considered as random in this study are the thermal and mechanical properties of materials at ambient temperature and the geometrical parameters. The corresponding statistical distributions were chosen on the basis of experimental results published in References 7 and 9. Most of the relationships defining the variation of thermal and mechanical properties of materials with temperature are empirical, and therefore, a deviation between foreseen values obtained and real values has to be expected as a model error. Consequently, these relationships have been weighted by random factors (with unity mean). For each column simulated, the random characteristics at ambient temperature are generated at the beginning of the fire exposure, and are updated at every step of the earlier weighted relationships with new set weighting factors.

The temperature increase curve is deterministic and defined by Eq. (1). Nevertheless, the global heat transfer coefficient, depending on temperatures in surrounding gas and concrete surfaces, is also recursively recomputed at every step of the exposure time and weighted by a random factor (with unity mean).

Assuming a gaussian distribution of the variable E , the probability of failure index defined by

$$P_f = \Phi(-\beta) \quad (3)$$

where $\Phi(\cdot)$ is the standard normal distribution function. β is the safety index defined by

$$\beta = \frac{\mu E}{\sigma \mu} \quad (4)$$

This assumption is not rejected by Kolmogorov's conformity test for a 90% confidence level. Moreover, the method leads to a stabilized value of P_f for a number of simulations generally lower than 200.

4-RESULTS

In the following outcomes, the fixed parameters are:

1. The type of concrete (calcareous, compressive strength $f_c^I = 25$ MPa);
2. The type of reinforcement (high bond bars, yield strength $f_y = 500$ MPa); and
3. The section sizes (width $b = 40$ cm; depth $h = 30$ cm) and the concrete cover thickness ($c = 4$ cm).

The columns are supposed to be surrounded by fire and have been designed according to the FB 87 rules to highlight the reliability obtained by use of these rules.

Nevertheless, any design code can be tested by the same method.

The parameters to be changed are:

1. The steel ratio ρ (ratio of the total section area of steel to that of concrete);
2. The geometrical slenderness $\lambda G = L_b/h$, L_b being the effective buckling length;
3. The required stability duration under fire exposure t_F taken into account for the design;
4. The proportion eQ of the live load maintained during fire exposure; and

5. The loading ratio $\alpha L = G/(G + Q)$, G being the dead load and Q the nominal live load.

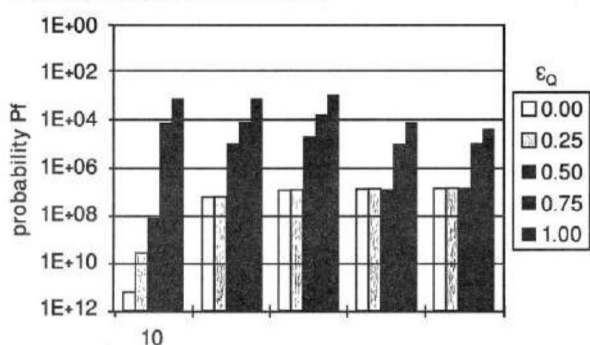


Fig. 7—Values of Pf for $tF = 1h$, $p = 1.00\%$.

Some representative results of this study corresponding to the case where the ratio αL is equal to 2/3 are presented in Fig. 7 to 11. Other results are presented in Reference 14.

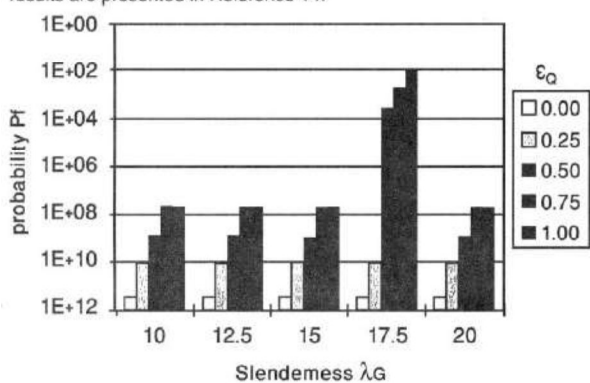


Fig. 8—Values of Pf for $tF = 1h$, $p = 2.45\%$.

The failure probability scale varies from 10^{-12} to 1; however, values less than 10^{-7} are only given herein as a comparison or to underline the effect (as a trend) of various parameters.

The comparison of Fig. 7 and 8, then 9 and 10, show that the reliability increases with the steel ratio (although the design load increases as well).

The risk of failure mainly depends on the live load fraction εQ attained during the fire; this fact is not explicitly taken into account in the design codes. This phenomenon increases with the fire exposure time:

when $tF = 4h$ (Fig. 11), the simulated columns always fail before this time expires, while the total live load continues to be applied.

4. PROPOSALS TO IMPROVE DESIGN METHODS

The reliability of reinforced concrete columns varies considerably with respect to various parameters. The tool that has been perfected in this study could allow these rules to be changed. An example is given in the case of the FB 87 rules.

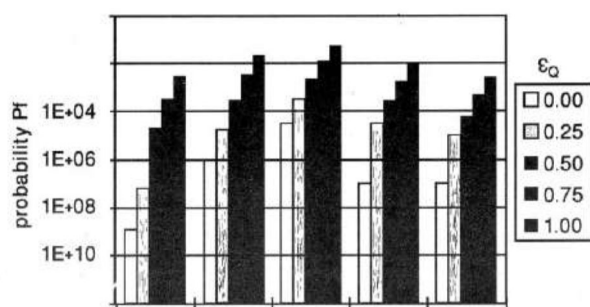


Fig. 9—Values of Pf for $tF = 2h$, $p = 1.00\%$.

The design formula of the ultimate load takes into account the reduction factors ϕ_c and ϕ_s of concrete and steel strengths, respectively, according to the mean temperatures in the materials.

There is no evidence that the column behavior only depends on these reduction factors and means temperatures because of the strain gradients resulting from second order effects. Moreover, the reduction factor ϕ_c does not vary in a linear way with temperature, which could partially explain the observed anomalies.

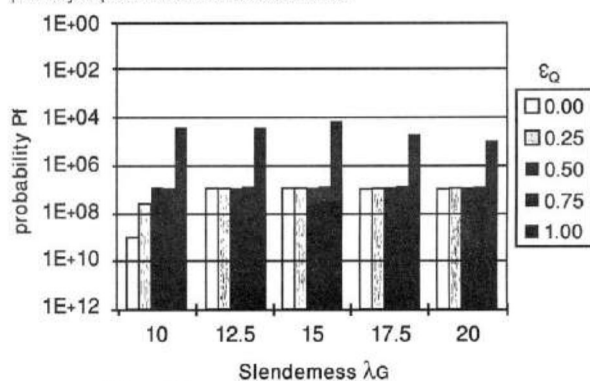


Fig. 10—Values of Pf for $tF = 2h$, $p = 2.45\%$.

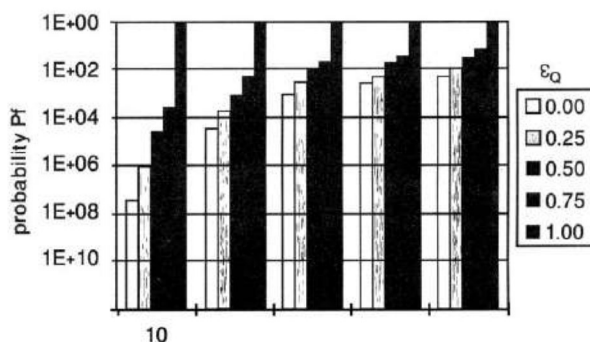


Fig. 11—Values of Pf for $tF = 4h$, $p = 2.45\%$.

Taking into account the effective distribution of temperatures and the corresponding values of ϕ_c in the concrete cross section would improve the existing formula. This calculation needs numerical tools whose use remains impractical for designers. We propose a simplified method. The concrete cross section is subdivided into three zones, B1, B2, and B3, as described in Fig. 12, for which the corresponding reduction factors ϕ_{c1} , ϕ_{c2} , and ϕ_{c3} must be defined. The distances h_1 and h_2 are defined as follows

$$h_1 = \frac{h}{3} \quad (5)$$

$$h_1 = \frac{b}{3} \quad (6)$$

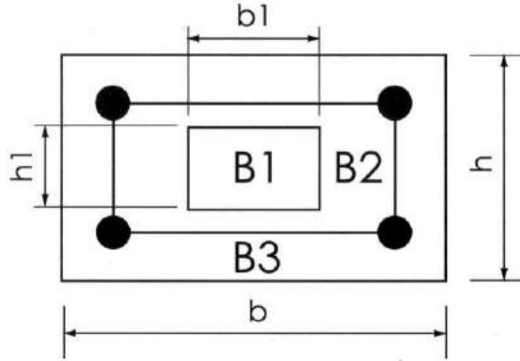


Fig. 12— Dividing section into three zones

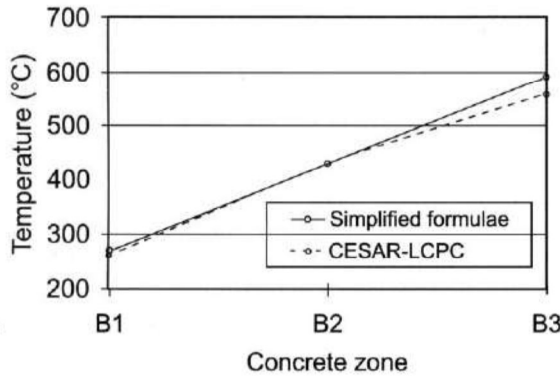


Fig. 13— Comparison between CESAR-LCPC and simplified model.

for a thick cross section, that is, $b \leq 1.5h$

$$b = b - \frac{2h}{3} \quad (7)$$

for a thick cross section, that is, $b > 1.5h$. The convention $b > h$ is assumed.

The temperature θ_{B1} , θ_{B2} , and θ_{B3} in these zones are then estimated from

$$\theta_{B1} = \frac{1}{3} (0.8\theta_{fire} + \theta_c) \quad (8)$$

$$\theta_{B3} = \frac{1}{2} (0.8\theta_{fire} + \theta_{ms}) \quad (9)$$

$$\theta_{B2} = \frac{1}{2} (\theta_{B1} + \theta_{B3}) \quad (10)$$

Where θ_{fire} is the temperature of the surrounding gas calculated according to the temperature increase curve, and θ_{ms} and θ_c are the mean temperature and the temperature at the center of the concrete cross section, respectively, possibly given by tabulated data. The temperature obtained with these formulae are slightly different from those calculated using the CESAR-LCPC program, as can be seen in

Fig.13.

The load-bearing capacity of a column can then be estimated considering for each zone, B1, B2, and B3, the effective concrete strength $\phi_{c1}f'_c$, $\phi_{c2}f'_c$ and $\phi_{c3}f'_c$ respectively.

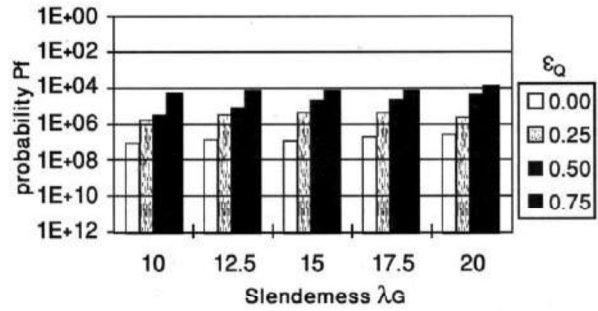


Fig. 14— Value of Pf for $tF = 4h$, $p = 1.00\%$

For design purpose, a simplified formula derived from FB 87 rules is proposed as follows

$$N_{u'} = \alpha' \left(\frac{f'_c \sum A_{ci} \phi_{ci}}{1.3 \times 0.9} + f_y \sum A_{si} \phi_{si} \right)$$

Where α' is the area of the concrete zone θ_{B1} , θ_{ci} is the corresponding reduction factor for the strength of concrete at temperature θ_{B1} and A_{si} is the area of the reinforcing bar (i) and ϕ_{si} is the corresponding reduction factor. The factor α' is defined as

$$\alpha' = \frac{1}{1 + 0.2 \left(\frac{\lambda}{35} \right)^2} \text{ if } \lambda \leq 50 \quad (12)$$

$$\alpha' = 0.71 \left(\frac{50}{\lambda} \right)^2 \text{ if } 50 \leq \lambda \leq 70 \quad (13)$$

Where λ is the slenderness ratio stated by $\lambda = Lb/r$, where r is the radius of gyration, that is $\lambda = \sqrt{12} \lambda_G$ for a rectangular cross section.

Some results obtained by applying the suggested formulas are presented in fig.14 and 15, in the case of $tF = 4h$. The steel ratio and the buckling length have practically no longer effect on reliability. Some similar results are obtained for $tF = 2h$ and for $tF = 1h$. It is not possible at this stage to specify the incidence of this method on the initial cost of column construction; it depends on parameters such as required stability time tF and ratio εQ of live load maintained during this time. In fact, it is necessary first to specify level to be reached, which may also depend the parameters. Nevertheless, such decisions can be made only the appropriate authorities. This calculation method could be help the decisionmaking process by defining the calculation values of different random variables or by modifying certain partial safety factors, for example, by means of a process test in different combinations.

CONCLUSION

The first aim of this study was to perfect a calculation method to determine the resistance of reinforced concrete columns exposed to fire. The model chosen is based on a segment method which, as well as being acknowledged to be reliable and realistic, can calculate very rapidly. It associates the CESAR-LCPC finite element program for the thermal study and a specific program for the structural behavior computing the instability load.

The reliability of reinforced concrete columns, designed on the basis of codes, was then estimated by means of Monte Carlo simulations. This reliability characterizes the capacity of these columns to support their nominal load during a set time t_F of fire exposure.

The failure probability of columns, before the required time t_F of fire stability is over, varies considerably in certain cases according to the following parameters; buckling length, steel ratio, and time t_F taken into account in the design. For $t_F = 4h$, premature failure is very often the rule. The present study is in keeping with the field of reliability analysis of engineering structures subjected to thermal and mechanical loading. It could be a helpful decision-making tool for drawing up or improving calculation codes.

We suggest, as an example, a modification on the current verification method for the concrete columns outlined by the FB87 rules. This modification, which is very simple to apply, enables chosen reliability levels to be obtained. They could also be easily adjusted to comply with other safety requirements.

The results presented herein only deal with one of the problems and only concern as defined in study (concrete cross section and cover thickness, and boundary conditions). To scope the proposals, it will be necessary to examine the influence of other parameters such as concrete cover thickness, cross section sizes, and links at the ends. The case of fire exposure on one side, which is sometimes more harmful, will have to be considered. In addition, the risks of concrete spalling, especially in the case of high-strength concrete, need to be examined. Research currently in progress in different laboratories should yield the additional data that is needed.

CONVERSION FACTORS

1m	=	39.37in
1MN	=	224.82kips
1MPa	=	0.145 kip/in

NOTATIONS

A_{cl}	=	Area of concrete zone B
A_{si}	=	Area of reinforced bar(i)
b	=	Width of columns cross section
b_1	=	Width of central concrete zone of column cross section.
c	=	Concrete over.
f'_c	=	Compressive strength of concrete cylinders
f_y	=	Yield strength of steel
G	=	Nominal dead load
h	=	Overall depth of column cross section.
h_1	=	Depth of central concrete zone of column cross section.
L_b	=	Effective buckling length
P_f	=	Failure probability

Q	=	Nominal live load
R	=	Resistance (bearing capacity)
S	=	Load applied to column
t_F	=	Required stability duration under fire exposure
u	=	Depth from concrete surface
α_L	=	Loading ratio
β	=	Safety index
ε_Q	=	Proportion of live load maintained during fire exposure
$\Phi_{(.)}$	=	Standard normal distribution function
ϕ_c	=	Reduction normal distribution function
$\theta_{c1}, \theta_{c2}, \text{ and } \theta_{c3}$	=	Reduction factor of concrete strength
ϕ_c	=	Reduction factors due to temperature
$\theta_{B1}, \theta_{B2}, \text{ and } \theta_{B3}$	=	Reduction factor of steel strength, bar(i)
λ	=	Slenderness
λ_G	=	Geometrical slenderness
$\phi_{c1}, \phi_{c2}, \text{ and } \phi_{c3}$	=	Temperatures (degrees C) in zone B ₁ , B ₂ and B ₃
θ_{fire}	=	Temperature at center of concrete cross section (degrees C)
$\theta(t)$	=	Temperature (degrees C) in compartment at time t
$\theta(t_0)$	=	Temperature (degrees C) in compartment at time t_0
θ_{ms}	=	Mean temperature in steel bars
ρ	=	Steel ratio

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