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Response of Reinforced Concrete Frames to Fire

Comportement au feu des structures en béton armé

Brandverhalten von Bauteilen aus Stahlbeton

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INTRODUCTION

Structural design for fire resistance must provide structural integrity for the level of safety desired in a particular building. Providing such structural integrity requires that geometric space characteristics, building materials, contents, and occupancy, as well as different levels of fire intensity, spread, and damage be considered. For rational design, four categories of fire and corresponding levels of tolerable damage may be identified.

Category		1	2	3	4
Fire	Intensity	Low	Low	High	High
	Duration	Short	Long	Short	Long
Structural Response	Damage Level	Nonstructural damage only.	Some structural damage. No collapse.		Specified endurance - - hours

In order to determine probable damage levels, thermal and structural response to the critical fire environment expected in a particular building must be evaluated by calculating time variations in temperature distribution within the structural elements of the building, as well as deformation and stresses in these elements, and initiation and extent of degradation (cracking, crushing, yielding, or rupture) for different types of fire. Evaluation of structural response should also account for different conditions of restraint by the building system. This is essential for economical and safe design, as such information is needed for selecting trade-offs between various means of fire protection vs. additional structural integrity, and for realistically assessing in-place performance.

Fire endurance ratings based on observed behavior of structural elements under standard test conditions cannot provide the information necessary for a rational design for fire safety. For example, if no collapse for type 2 or 3 fire is ensured, then a lower endurance rating than the present requirement might be acceptable for some structures, leading to a more economical design. Therefore, analytical predictions of thermal and structural responses are needed for an optimum design decision.

Determination of thermal and structural response is possible provided that space characteristics, fire environment, structural system, and material behavior when exposed to a fire environment are suitably modeled. In this paper, the methods and validity of analytical predictions of behavior of reinforced and prestressed concrete elements in fire environments are discussed in relation to observed behavior and to current methods for rating the fire endurance of such structural elements.

ANALYTICAL MODELS

In modeling the fire response of structures, heat flow analysis was separated from structural analysis and two computer programs, FIRES-T and FIRES-RC, were written for solving the separate problems. The details of analytical modeling and numerical methods used in solving the problems have been described elsewhere [1-3]. A brief review and some additional comments on modeling fire environments are included here.

Thermal Analysis - For heat flow analysis, a finite element method [2] coupled with time step integration is used. The problem is solved by satisfying the heat balance equation and a known boundary condition at all nodes. The exposed surface boundary condition is modeled either as a prescribed temperature history at the surface or as a heat flux based on convective and radiative transfer mechanisms from an external heat source. For simplicity, this heat flux, q, is expressed as a sum of convective and radiative terms, q_C and q_R, respectively.

$$q = q_C + q_R = A(T_f - T_s)^N + V\sigma(a\varepsilon_f\theta_f^4 - \varepsilon_s\theta_s^4)$$

where: $T_f = F(t)$ is the time-dependent single-valued temperature of the fire, T_s is the average surface temperature of a small element associated with a particular node, A is the convection coefficient, N is the convection power factor, V is the radiation view factor, a is the surface absorption factor, ε_f and ε_s are emissivities of the fire and the surface, respectively, σ is the Stefan Boltzmann constant, and θ stands for absolute temperature.

This model of the boundary condition is based on the assumption that the heat source can be represented by a turbulent, well-mixed gas having, at any time t, a single value of temperature T_f, and a single value of emissivity ε_f. This model can be viewed as a pseudo-fire in which the effects of temperature gradients, gas flow, fire, load distribution, and enclosure wall radiation characteristics are represented by T_f and ε_f. The boundary condition is further simplified by assuming A, N, a, ε_f, and ε_s as constants throughout the fire duration. In some cases, view factors have been varied for different surfaces of the exposed element, although a value of 1.0 has been used in most cases. Exposure to nonfire conditions on the boundary, such as ambient atmospheric exposure, can be modeled as exposure to another 'pseudo-fire' with appropriate T_f and ε_f.

An iterative procedure is used within each time step to deal with the temperature dependence of material properties and nonlinear thermal boundary conditions. The problem is then linearized about the current temperature distribution within a given iteration. A two-dimensional problem is solved, assuming no heat flow along the long axes of frame members. Member cross-sections can have any shape and may be composed of several materials (concrete, steel, insulation); it is assumed that there is no contact resistance to heat transmission at the interface between these materials. Changes in geometric characteristics associated with structural distress, such as spalling, can be accommodated in solving the heat flow problem, provided that the time of occurrence and extent are defined. When such behavior is indicated in the structural response, the two solutions - heat flow and structural analysis - must be coupled and additional iterative cycles will be required to obtain a solution.

Structural Analysis - A nonlinear direct stiffness formulation coupled with time step integration is used for structural analysis [3]. Within a given time step, an iterative approach is used to find a deformed shape which results in equilibrium between the forces associated with external loads and internal stresses and degradation. The material behavior models for concrete and steel account for dimensional changes caused by temperature differentials, changes in mechanical properties of the material with changes in temperature, degradation of the section through cracking and/or crushing, and increased rates of shrinkage and creep with an increase in temperature. Nonlinear stress-strain laws are used to model the behavior of concrete and steel; these laws are capable of accounting for inelastic deformations associated with unloading. Based on this formulation, a computer program, FIRES-RC, has been developed which is directly coupled to the thermal analysis, FIRES-T.

Geometric discretization of the frame and its elements is shown in Fig. 1. The members are substructured into segments and the cross-sections are further subdivided into subslices by appropriately choosing a finite element mesh. Steel and concrete subslices are treated as uniaxially loaded prisms, so that only uniaxial stress states are considered. Wherever possible, advantage is taken of conditions of symmetry.

VERIFICATION OF ANALYTICAL MODELS

The validity of the simplifications made in the analytical models described above can be judged by comparing analytical results with experimental data.

University of California, Berkeley, Studies [4] - The specimen used in the UCB study was a 12 in. (0.3 m) square prism, 60 in. (1.5 m) long, reinforced with eight No. 5 (15.9 mm diameter) reinforcing steel bars. The specimen was instrumented with thermocouples on both steel and concrete, and with strain gages attached to the steel reinforcing bars. The unloaded specimen was subjected to several cycles of controlled heating in a radiant oven producing approximately uniform surface temperature. An upper limit of 600°F (316°C.) was selected for testing the specimen because reliable measurements of strain above this temperature are difficult to obtain. After heating tests of the unloaded specimen were completed, the specimen and oven were moved into a testing machine and three groups of tests were performed in which the specimen was subjected to: (1) heating, (2) axial compression loading and unloading without heating, and (3) heating under constant compressive load.

Analytical predictions of temperature distribution for a typical cycle are compared with experimental data in Fig. 2 where the influence of varying conductivity on calculated values is shown. The predicted concrete temperatures differed from the observed values, partly due to the approximation of thermal diffusivity values used in the analysis, and partly due to the assumption of constant diffusivity throughout the section. The outer 1-inch layer of concrete, which had undergone higher temperature exposure and moisture loss than the interior, is likely to have had a lower diffusivity than the interior, possibly accounting for the difference between observed and predicted temperature values. Nevertheless, the difference between computed and observed values is not great, and the analytical model for thermal response was therefore considered satisfactory.

Predicted and measured deformations of the prism, subjected to a constant load and a heating and cooling period, are compared in Fig. 3. Good agreement is observed in this case. Deformations during loading and unloading cyclic tests without heating are shown in Fig. 4. The low initial stiffness of the prism and subsequent stiffening with increased compressive load reflect the initially cracked state of the interior concrete portion (a consequence of prior heating), followed by closing of the cracks when a compressive load of about 100 kips was reached. The agreement between predicted and observed values under unheated conditions is somewhat less accurate, attributable to some deficiencies in modeling material properties such as nonlinear stress-strain and fracture behavior of concrete in tension under normal and elevated temperatures, nonlinear characteristics of the unloading portion of the compressive stress-strain relationship of concrete at different temperatures, and high-temperature creep in steel and concrete. Nevertheless, the predicted structural behavior of a reinforced concrete prism loaded in compression and subjected to heating cycles exhibited close agreement with measured values (Figs. 3 and 4).

Portland Cement Association Laboratories, Skokie, Illinois, Studies [5, 6] - Two types of prestressed concrete specimens were used in the PCA studies. One group of specimens [5] consisted of slab strips, uniformly loaded over a 12-ft. (3.7 m) simply supported span. In these specimens, aggregate type, concrete cover thickness, size of prestressing strand, and load intensity were varied. In the second group of specimens [6], I-beam specimens with six aggregates were tested using two load intensity levels. Results of the I-beam tests were compared with computed values and generally showed agreement as good as the slab data. The comparison is omitted here due to length limitations on this paper.

The slabs were tested in the PCA floor furnace, and the furnace heating was controlled to meet the standard ASTM E119 time-temperature requirements. The fire temperatures measured by the individual furnace thermocouples showed only small variations from the average, and the average value agreed closely with the standard time-temperature curve. However, during the initial phase of rapid heating, the gas (fire) temperature may differ significantly from the values recorded by shielded, slow response thermocouples. To obtain good agreement between measured and calculated thermal response, it is essential to use a pseudo-fire model reflecting actual conditions as closely as possible. Measurements of temperatures in a wall furnace carried out by Babrauskas [7] using fast response thermocouples have been used to establish a corrected ASTM E119 pseudo-fire time-temperature curve to be used in predicting thermal response during a standard test conducted in accordance with ASTM requirements. The corrected temperature (Fig. 5) is about 500°F (278°C.) higher at 1 minute, 350°F (194°C.) at 3 minutes, 150°F (83°C.) at 6 minutes, and 40°F (22°C.) higher at 12 minutes. No correction is required for times in excess of 24 minutes. These corrections, albeit approximate, provide a much better basis for predicting thermal response during the first 0.5 hour of a standard test.

In modeling thermal boundary conditions for the slabs, it was assumed that the pseudo-fire could be represented by the modified temperature history for a source of $\varepsilon_f = 0.5$. The ambient air was modeled as a pseudo-fire having a constant temperature of 68°F (20°C.) with $\varepsilon_f = \varepsilon_s = a = 1.0$. View factors for all horizontal external surfaces of slabs were taken as 1.0. The vertical sides of the slab strips were insulated so that the horizontal heat flow laterally and longitudinally could be neglected.

Comparisons of calculated and observed temperatures and deflections [8] for a typical prestressed slab (Figs. 6 and 7) indicate good agreement.

CASE STUDY [9]

Encouraged by the reasonably good agreement between analytical predictions and laboratory results, an attempt was made to study the behavior of the sixth story of the Military Personnel Records Center in St. Louis, in which the roof collapsed during a 30-hour fire on July 12-13, 1973 [10,11]. The complex history of the fire, the complex structural system of the building, the lack of accurate records of fire spread, intensity, and structural response, make detailed study of behavior very difficult. Nevertheless, correlation of observed behavior and analytical predictions demonstrated good agreement and provided explanations for observed failures which could not otherwise be explained.

Several observations can be made from the results of the MPRC case study:

1. The deflected shape of the roof after 3 hours of fire exposure (Fig. 5) is shown in Fig. 8. Extrapolating the calculated horizontal displacement of 1.2 in. (30 mm) for one bay, the maximum E-W displacement at the corners of the roof would be about 40 in. (1 m) each. The displacements measured after complete cooling were about 20 in. (0.5 m) each. Considering that some recovery must have taken place during cooling, but that complete recovery could not be achieved because of permanent damage in the slab and the supporting columns, the estimated deflection of 40 in. (1 m) during the fire appears to be reasonable.

2. The relative depression of the slab at the column support is contrary to the normal deflected shape and is primarily due to the very rigid restraint of the slab by the column capitals. The zones of relative depression of the roof slab could be observed on aerial photographs as small ponds of collected water. Locations of these ponds with respect to the structural frame could be established from the photos and correlated well with the locations of calculated depressions over the columns.

3. Calculated bending moments in the roof slab showed a sign reversal in the center region of the bay so that steel reinforcement was required under fire exposure in the top of the slab, while for service load conditions, top steel was provided in the end-quarters only. Absence of top steel in the middle portion of the bay would indicate the likelihood of failure in the vicinity of the top steel cut-off. This was fully supported by observations, as large portions of the roof slab seemed to have ripped along the lines where the top steel was discontinued.

4. Calculated moments and shears in the exterior columns under fire exposure increased dramatically. The maximum moment increased more than two-fold as compared to maximum moments under service conditions, and the shear in the column increased three-fold. Moments and shears also increased in the interior columns, but the increases were less pronounced. Calculated moments reached the ultimate, but did not exceed it significantly. A few moment failures were observed in columns, but in most cases, the columns failed in shear. Calculations showed that internal cracking of concrete reduced the effective (uncracked) area to about 18 percent of gross area and thus reduced shear capacity greatly. The amount of lateral ties in the columns was nominal and thus did not contribute to shear resistance. While the shear capacity of the uncracked columns would have been sufficient to resist the increase in shear forces, the extensive degradation of the interior core reduced the shear capacity to such an extent that on the basis of calculation, shear failures were estimated at about 2-1/4 hours. Shear failures observed after the fire support the general prediction of a dominant shear failure mechanism in the columns.

RESPONSE OF COLUMNS TO FIRE [12]

A pilot study to explore the effects of fire characteristics and of structural restraint on the response of reinforced concrete columns in a multistory frame building was carried out. To provide a realistic basis for the study, a typical 12-story reinforced concrete building was selected, and responses of the basement and 11th floor columns were determined analytically. The fire environments were characterized by two time-temperature curves, assigning two emissivities for each. Column behavior during the 1-hr. fire exposure was studied using the computer programs FIRES-T and FIRES-RC.

Axial restraint stiffness was modeled by springs at the upper and lower ends of each column; the spring constants were calculated using linear elastic behavior of the surrounding structure and were assumed to be constant throughout the fire.

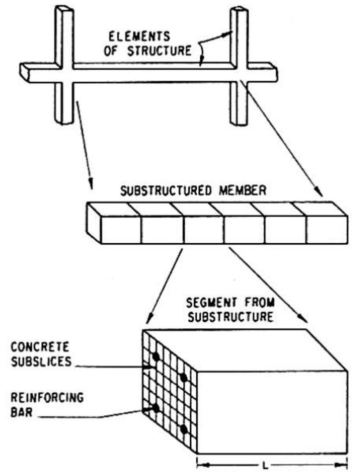

FIG. 1 GEOMETRIC IDEALIZATION

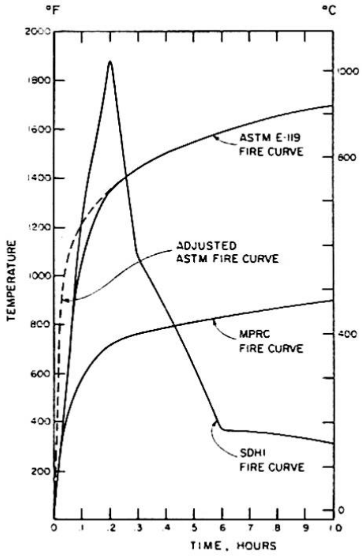

FIG. 5 FIRE CURVES

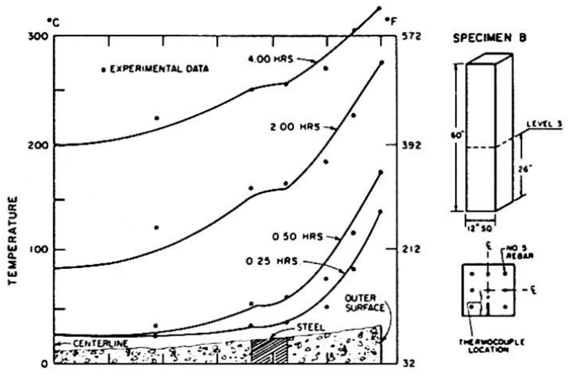

FIG. 2 COMPARISON OF EXPERIMENTAL DATA TO
CALCULATED TEMPERATURES

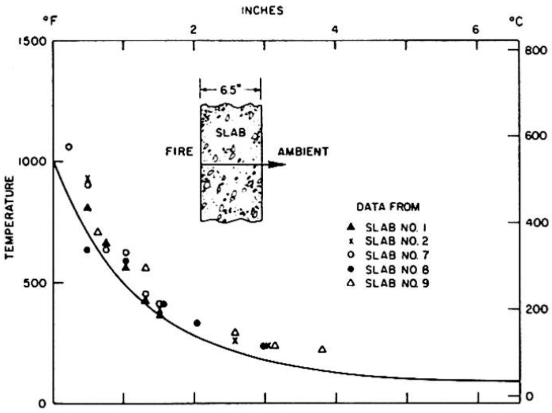

FIG. 6 COMPARISON OF EXPERIMENTAL DATA TO CALCULATED
TEMPERATURES - 30 MINUTE EXPOSURE

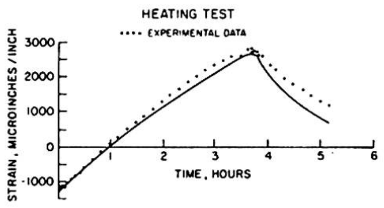

FIG. 3 COMPARISON OF EXPERIMENTAL DATA
TO CALCULATED STRAINS

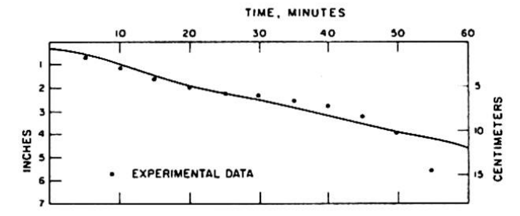

FIG. 7 COMPARISON OF EXPERIMENTAL DATA TO
CALCULATED DEFLECTIONS

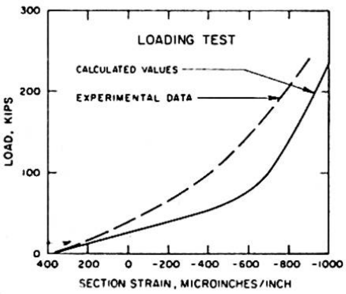

FIG. 4 COMPARISON OF EXPERIMENTAL
DATA TO CALCULATED STRAINS

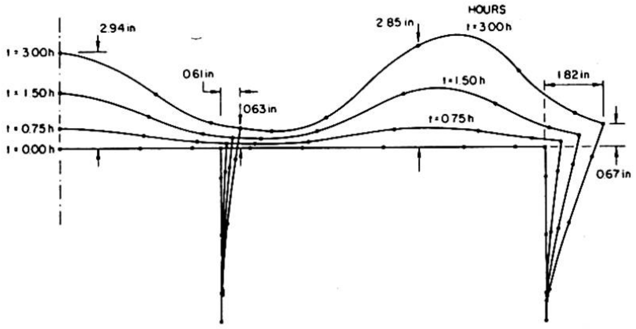

FIG. 8 FRAME DEFORMATIONS

Characteristics of the columns and thermal response in terms of maximum steel temperatures after 1-hr. fire exposure are summarized in the table below.

Column Location	Column Size in x in	Concrete Cover in.	Steel Reinf. Ratio	Initial Axial Load kips	Axial Restraint Stiffness kips/inch		Type of Fire & Emissivity				Maximum Steel Temperature °F (°C)			
							ASTM		SDHI Fig. 5		ASTM		SDHI	
					K_U	K_L	A9	A3	S9	S3	A9	A3	S9	S3
Basement	20 x 20	1.5	0.032	670	365	∞	0.9	0.3	0.9	0.3	761(405)	522(272)	401(205)	263(128)
11th Fl.	20 x 20	1.5	0.012	100	65	2000	0.9	0.3	0.9	0.3	637(336)	421(216)	368(187)	223(106)

Structural response in terms of relative deflections, steel and concrete load, and section degradation are summarized in the following table.

Column Location	Type of Fire	Deflection 1-hr % of initial[1]	Steel Load 1-hr % of initial	Concrete Load,1-hr % of initial	Initial Time of Crushing hrs[2]	Crushed Area,1-hr % of total	Initial Time of Cracking hrs	Cracked Area,1-hr % of total	Initial Time of Yielding hrs[3]	Concrete Area,1-hr % of total[4]	Flexural Stiffness 1-hr % of initial
Basement	A9	-397	435	42	0.25	34	0.45	50	0.50	16	30
	S9	-81	385	39	0.20	20	0.40	49	0.50	31	54
	A3	-250	353	54	0.55	20	0.70	54	0.75	26	25
	S3	+51	321	49	NC	1	0.65	19	NY	80	70
11-th Story	A9	-6413	-237	166	0.40	20	0.20	65	NY	15	21
	S9	-1875	+1033	18	0.20	19	0.15	74	NY	7	21
	A3	-5788	-2000	316	NC	3	0.35	80	NY	17	20
	S3	-275	+1128	12	NC	0	0.20	89	NY	11	32

1 minus sign indicates change from initial shortening to elongation or change in load from compression to tension.

2 crushing of entire 1 in. (2.54 cm.) thick peripheral layer; NC signifies that within 1 hr. there was no full crushing of the outer layer, although partial crushing (e.g. at corners) may have taken place.

3 NY signifies that no yielding of steel reinforcement within 1 hr. fire duration has taken place.

4 effective concrete area remaining after cracking and crushing.

Geometrically, the two columns (basement and 11th floor) differ only in the amount of steel reinforcement in each. The fire endurance rating obtained from a standard fire exposure test would be the same for both columns. Yet, as shown by the tables above, the thermal response differs greatly with type of fire, and the structural response differs greatly with both type of fire and amount of axial restraint.

CONCLUSIONS

The studies carried out to-date indicate that for reliable prediction of response, pseudo-fire characteristics should include emissivity in addition to realistic time-temperature models.

The structural response of reinforced concrete structures is sensitive to the following characteristics: variations in thermal coefficients of expansion, stress-strain relationships and creep in both tension and compression, inelastic deformation associated with unloading, and fracture (cracking, crushing, rupture).

In reinforced and prestressed concrete, cracking of interior concrete due to thermal gradients greatly reduces strength and stiffness of the element exposed to fire. This phenomenon is strongly influenced by pseudo-fire characteristics such as rate and duration of heating, peak temperature, and rate of cooling.

Four categories of fire with corresponding levels of tolerable damage have been suggested for a more rational design of structures for fire resistance. Standard tests of fire resistance do not provide sufficient information for such design. Information regarding loss of strength and stiffness, as well as the magnitude of fire-induced forces and deformations in structures for different fire conditions, must be considered in design.

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SUMMARY - Mathematical models developed for predicting thermal and structural response of reinforced and prestressed concrete frames in fire environments are substantiated by laboratory tests and case studies. Suggestions for a more rational design of structures for fire resistance are included.

RESUME - Des modèles mathématiques ont été étudiés afin de prédire le comportement thermique et structural de cadres en béton armé et précontraint dans un incendie. Ils ont été établis à partir d'essais en laboratoire et de cas réels d'incendie. Des propostions sont faites pour un dimensionnement plus rationnel des structures devant résister au feu.

ZUSAMMENFASSUNG - Mathematische Modelle wurden entwickelt, um das thermische und strukturelle Brandverhalten von Rahmen aus Stahl- und Spannbeton vorauszusagen. Sie wurden aufgrund von Laboruntersuchungen und wirklichen Brandfällen aufgestellt. Eine rationellere Berechnung der brandwiderstehenden Tragwerke wird vorgeschlagen.