IV

Fire Resistance of Reinforced Concrete Columns

Résistance au feu des colonnes en béton armé

Feuerwiderstand von Stahlbetonstützen

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In the past, the fire resistance of structural members could only be determined by subjecting them to standard fire tests. Now it is possible to determine the fire resistance of various structural members by calculation.

A numerical procedure has been developed (1) to calculate the fire resistance of square reinforced concrete columns and has been checked with the results of actual fire tests. The procedure has been used to calculate fire resistance under both standard ASTM test conditions (1) and under conditions that more closely resemble those encountered in practice (2).

Under standard test conditions, the fire resistance, expressed in time to failure, is calculated as a function of type of aggregate, column size, cover thickness to steel, amount of steel, eccentricity of loading, design safety factor and equivalent buckling length.

The standard ASTM test conditions (3) assume that the column is free to expand, whereas in fact a column is often restrained by the surrounding structure and thereby attracts more load than assumed. To examine the influence of restraint on the behaviour of the columns, calculations of fire resistance have been made for columns under various degrees of axial restraint.

Another condition in which test and practice differ is the temperature course of the fire to which the column is exposed (in practice fire severities can vary over a wide range). To study this, fire-resistance calculations have been made for columns exposed to heating according to various temperature curves that are characteristic of those of actual fires.

Material Properties

The temperature rise in a column is determined for the heat flow equations as a function of two properties of the concrete, thermal conductivity and thermal capacity. These properties are strongly dependent on the type of aggregate. Quartz aggregate, which has the highest thermal diffusivity of all concretes, was selected for determining these properties for normal weight; crystalline expanded shale aggregate was chosen for lightweight concrete. Values of the properties are given in reference (4).

The most important mechanical properties that determine the strength of reinforced concrete are the compressive strength and modulus of elasticity of the concrete, and the yield strength and modulus of elasticity of the steel reinforcing. Approximate analytical relations, which give these properties as a function of temperature, have been derived for normal weight concrete, lightweight concrete and steel using existing data (5-7). The relationships are shown in Figure 1.

In those parts of the studies that included a period of decaying fire temperature, the following assumptions were made regarding the change of material properties as they cool down. For concrete it was assumed that the material properties are unchanged during cooling and are equal to those corresponding to the maximum temperature reached. For steel it was assumed that the same relationships hold for cooling as well as for heating.

Deformations of concrete and steel due to temperature change were assumed (2) based on existing information. Quartz aggregate concrete was chosen because other aggregates, especially lightweight aggregates, expand less with increasing temperature.

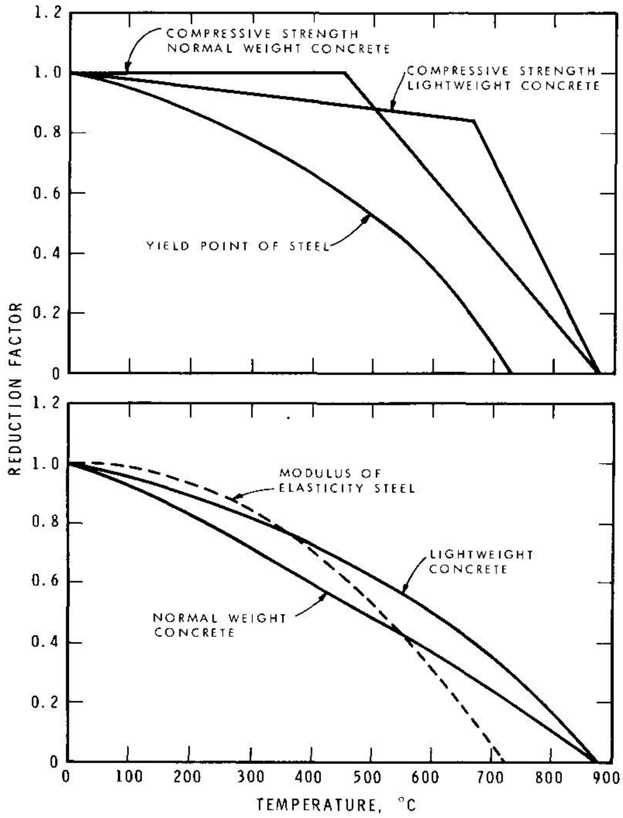

Figure 1: Effect of Temperature on Material Properties

Calculation of Fire Resistance

Column temperatures are calculated by a numerical method (8). The method assumes radiative heat transfer from the fire to the surface of the column. In the column the heat transfer proceeds by conduction. The temperature dependence of the thermal properties of the material are taken into account in the calculation.

During a fire, large temperature gradients, hence large stress gradients, occur in the column cross-section. Figure 2, based on an analysis of the stresses and strains occurring in the column during fire (2), shows how stress distribution changes in a column section during a fire. Early in a fire, very high stresses occur near the outside of the section and cracks appear in the inner area, which is under tensile strain. As the fire progresses, the outer concrete loses its strength, the temperature gradients become less steep, and the extent of cracking in the inner area is reduced. As failure is approached, the cracks in the central area disappear as the reduced effective cross-section becomes stressed to capacity throughout. This demonstrates that fire resistance can be calculated on the basis of ultimate capacity of the cross-section, which assumes full stress redistribution at failure.

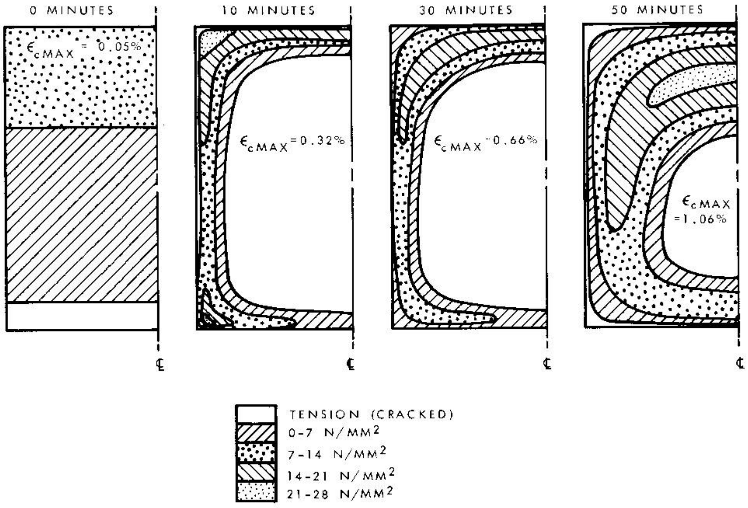

Figure 2: Stress Distribution in Concrete Section During Fire (20 cm Square, 4.9% Steel, 1.25 cm Cover, Calculated Fire Resistance 54 min.)

The strength of reinforced concrete columns at elevated temperatures has been determined by the same method as the ACI method (9) for room temperature, by replacing the relevant material properties by their temperature-dependent values given in Figure 1. To do this the cross-section was divided into small elements, the properties for each element determined for the corresponding temperature, and the effects integrated over the section. Figure 3, a typical interaction diagram of axial and bending strength, shows how fire reduces the section strength with time.

In the case of long columns, the resistance also depends on the buckling load, which is a function of the stiffness. The stiffness is likewise determined by integrating the temperature-dependent stiffness of elements. The stiffness of the concrete, however, is reduced due to cracking and this is approximated by means of a reduction factor. Figure 3 shows how buckling effects during fire reduce the strength of an intermediate column and that of a short column.

In Table I the calculation method is compared to actual test results of axially loaded quartz aggregate columns (10); these tests were carried out under well defined heating and loading conditions. It is seen that the calculation procedure gives a consistent, although somewhat conservative, estimate of the fire resistances of reinforced concrete columns under known load conditions over a considerable range of slenderness ratios. The differences, which are in the order of 10 to 20 per cent, are probably mainly due to conservativeness in the material properties used and in the calculation assumptions.

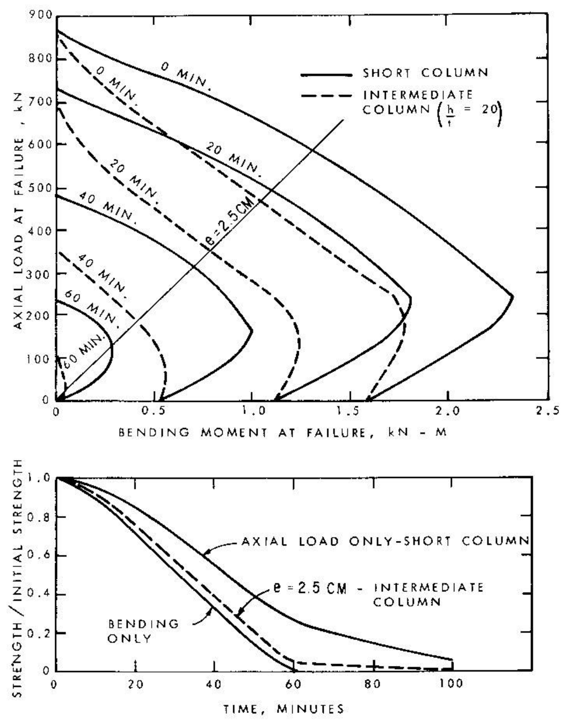

Figure 3: Effect of Fire on Column Strength (15 cm x 15 cm, Normal Weight Concrete 3.5% Reinforcing, 1.25 cm Cover)

Standard Fire Resistance of Square Columns

Figure 4 shows the calculated fire resistance of square columns under ASTM fire test conditions (3) for a standard

TABLE I - COMPARISON OF CALCULATED FIRE RESISTANCES WITH TEST RESULTS

Specimen	Compressive strength of concrete, N/mm^2	Test load, kN	Time to failure, minutes		Ratio time to failure,
			Test	Calculated	Test/Calc.
(i) 15 cm x 15 cm - 4 corner bars 2 cm dia., cover 1.2 cm					
2	38	273	64	57	1.12
3	31	273	69	55	1.25
4	31	273	63	55	1.15
5	31	273	66	55	1.20
(ii) 20 cm x 20 cm - 4 corner bars 2 cm dia., cover 2.9 cm					
3	32	464	107	89	1.20
4	32	464	105	89	1.18
5	43	598	107	88	1.22
6	43	598	107	88	1.22
(iii) 30 cm x 30 cm - 4 corner bars 2.1 cm dia., 4 mid-face bars 2.2 cm dia., cover 3 cm					
11	46	1708	165	147	1.12

case described as follows. The column, designed according to ACI 318-63 (adopted for the National Building Code of Canada), was pin-ended, 3 m long, and subjected to full design dead load plus live load. It was also subjected to the minimum eccentricity specified in ACI-318 (0.1 of the size of the column or 2.5 cm, whichever is the greater); this corresponds to a typical interior lower storey column.

The main parameters affecting fire resistance are column size, cover and type of aggregate. Figure 4 shows that an increase of cover increases the fire resistance, as expected, owing to thermal protection. This increase, however, levels off when the cover reaches 1/4 of column size (t). It is followed by a decrease, caused by a decrease in the moment-resisting lever arm due to a weakening of the outer layers of concrete. For large, lightly reinforced sections and small cover, the influence of the cover is negligible because the concrete carries the entire load at failure.

Other factors that affect the fire resistance of a column are the length of the column, eccentricity (or moment) and the safety factor. Studies (1) showed that an increase of safety factor or a live load less than the full design live load, may significantly increase the fire resistance. The results also show that an increase in eccentricity decreases the fire resistance, especially when the percentage of steel is small (1 to 2%). The decrease, however, is generally not of practical importance because eccentricity in a column under fire will be

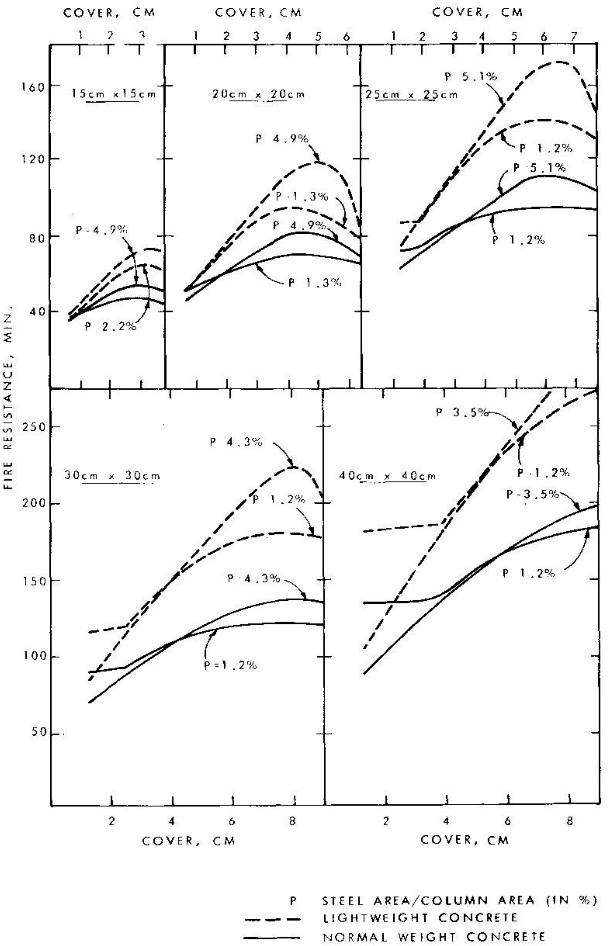

Figure 4: Fire Resistance of Reinforced Concrete Columns as a Function of Cover Thickness to Steel for Various Column and Steel Sizes (Column Length: 3 m; Min Eccentricity: 2.5 cm or 0.1t; Full Live Load)

greatly reduced from the design eccentricity assumed for calculation (1). It was also found that the fire resistance decreases with increase of slenderness ratio; this generally affects only tall slender columns with a small percentage of steel.

The results given in Figure 4 and in Reference 1 for square columns can be expressed in the following simple design rules for minimum column size (t_{min}, cm) and cover* (C_{min}, cm) in terms of required fire resistance (R, hr):

$$
\begin{aligned}
t_{min} &= 10f(R+1) && \text{for normal weight concrete}\\
t_{min} &= 7.5f(R+1) && \text{for lightweight concrete}\\
C_{min} &= 2.5\,R && \text{for } R \le 2 \text{ hr}\\
C_{min} &= 1.25\,R + 2.5 && \text{for } R > 2 \text{ hr}
\end{aligned}
\tag{1}
$$

where f, a factor that takes into account over-design, equivalent buckling length (kh) and percentage of steel (p), is given as follows:

Over-design factor	Values of f		
	kh ≤ 3 m	kh = 6 m	
		t ≤ 30 cm p < 3%	All other cases
1.00	1.0	1.2	1.0
1.25	0.9	1.1	0.9
1.50	0.8	1.0	0.8

* Wire mesh in cover is recommended if cover is greater than 3.75 cm

These results indicate lower ratings on dimensions for normal weight concretes than existing ratings cited in the National Building Code of Canada, and about the same for lightweight concretes. The lower ratings are attributed to a decrease in design safety factor (existing ratings were based on tests carried out in 1920 - 1921), a consideration of minimum eccentricity that occurs in practice, and a more accurate simulation of the heat transfer conditions in a fire.

Interaction With Building Structure

Determination of fire resistance, whether by standard fire test or by the above calculations, is based on the assumption of a pin-ended column with no interaction with the surrounding building structure. Actually, a column expands during fire and, because of the resistance of the surrounding structure to this expansion, more than the assumed dead load plus live load is attracted to the column. It therefore appears, for the single column exposed to fire, that the standard procedure errs on the unsafe side.

Figure 5 shows the results of an interaction study of a column with a typical reinforced concrete building structure. The curves in the top graph show how the applied load, including the effect of interaction, compares with the calculated column strength. The different curves labelled "n slabs" refer to the number of floor slabs above the column that resist expansion of the column; "0 slab" corresponds to no interaction. Curves in the bottom graph show the corresponding column lengthening and lateral deflection of the column at mid-height.

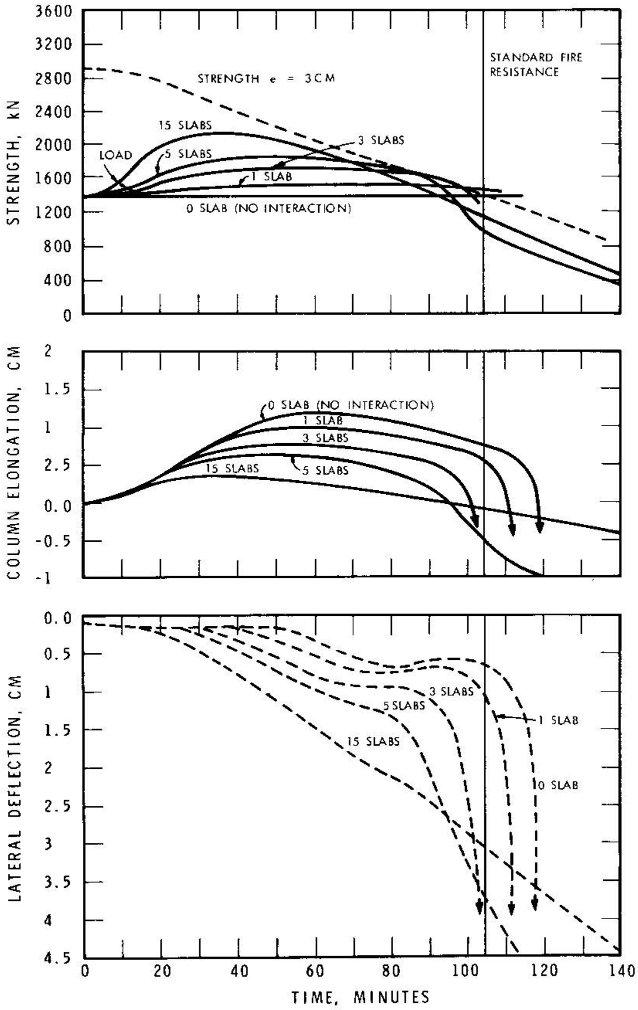

Figure 5: Column-Structure Interaction During Fire (30 cm x 30 cm Square Column, 4.3% Steel, 3.75 cm Cover, 3 m long.)

As the vertical stiffness of the surrounding structure increases from 0 to 3 storeys (or slabs), the applied load increases and the fire resistance is decreased, as seen by comparing column movements. The decrease, however, is not large and the resulting fire resistance is never much less than that calculated from Equation (1). This is because a reduction in end moments due to increased column flexibility compensates for the increased column load due to interaction.

As the vertical stiffness is increased further to 5 storeys, however, a change takes place. What happens is that the increased applied load causes lateral bending to take place, followed by chord shortening between the column ends and relief of the applied load, so that failure does not take place. Eventually, the column becomes shorter than it was initially and, owing to load transfer to adjacent supports, the fire resistance is increased beyond that for an isolated column. For very stiff resistance (15 storeys), the column bends or buckles quite early, but the column does not fail. If the structure can transfer the total load to other supports, the column will never fail.

The results in Figure 5, therefore, indicate that interaction effects with the surrounding structure due to column expansion are not likely to reduce the fire resistance of isolated columns, and in some cases may increase it substantially. Another more critical interaction effect, revealed in the recent St. Louis fire, however, is the effect of large lateral expansion of the slab above the fire floor causing shear failure of columns. This requires further investigation.

Fire Resistance Under Actual Fire Conditions

The standard fire temperature curve initially rises at a prescribed rate and then drops suddenly to ambient temperature (3). In actual fires the temperature reached in the period of temperature rise can be much higher or lower than the standard temperatures. In addition, instead of a sudden drop to ambient temperature, there is, in actual practice, a gradual decrease of the fire temperature after the period of temperature rise.

A wide variety of temperature courses are possible, depending on the fire load, ventilation and other characteristics of the enclosure (1). The most severe are the ventilation controlled fires, i.e., the rate of burning of the combustible materials in the enclosure is determined by the dimensions of the openings through which the air necessary for combustion can be supplied. Formulae describing the temperature course of ventilation-controlled fires (11) have been adopted to calculate fire resistance of reinforced concrete columns (2).

As shown in Figure 6, two major parameters determine the temperature course of ventilation-controlled fires -- the window opening and the fire load (Q), as defined in Figure 6. The larger the opening factor (F) the greater the temperature rise and the shorter the duration. The fire load affects only the duration of the fire -- the higher the fire load the longer the duration of the fire. In studies at DBR/NRC (2), three characteristic temperature curves, corresponding to F = 0.02, F = 0.05 (standard case), and F = 0.1, were chosen.

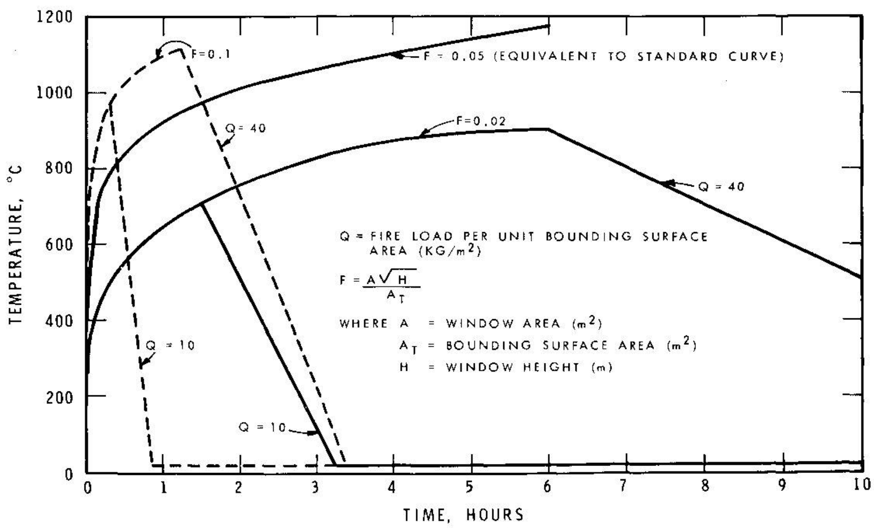

Figure 6: Characteristic Temperature Curves For Various Fire Loads Q and Opening Factors F (Heavy Bounding Walls)

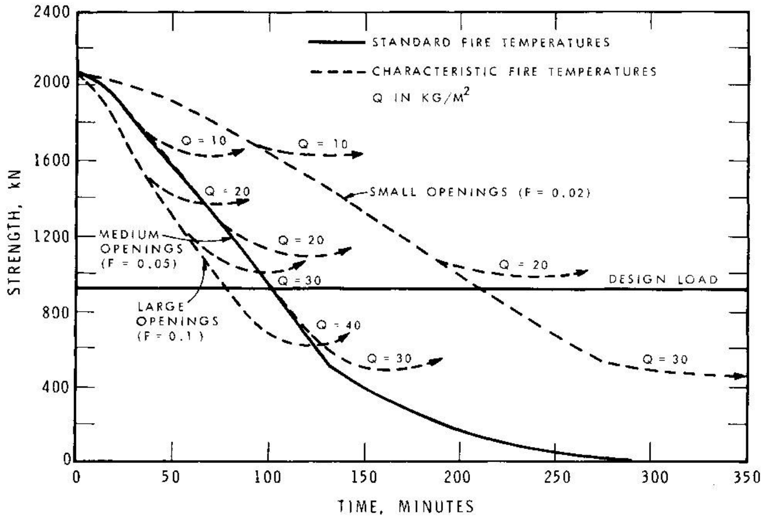

Figure 7: Strength-Time Curves for 30 cm x 30 cm Column (2.2% Steel 3.75 cm Cover)

Figure 7 compares, for a typical case, column strength vs time for different characteristic temperature curves with that for the standard curve. The curves show that for a fire load less than a critical value, Q_{crit}, no failure of the column takes place; in other words below a certain critical value of the fire load there is insufficient combustible content to heat the column to failure. Above the critical fire load, i.e., when $Q > Q_{crit}$, the time to failure very quickly approaches a constant value, R, obtained by assuming an unlimited fire load. The time to failure, R, is a function of the opening factor: the greater the openings, the greater the rate of heat rise, and the shorter the time to failure. On the other hand, Q_{crit} increases with an increase of opening factor since more of the heat escapes the fire compartment.

Figure 8 shows the relationship between Q_{crit}, R, and dimensions for a cover of 3.75 cm. The major variables affecting critical fire load were found (2) to be column size, cover and opening factor; percentage of steel has only a minor effect. The following approximate relationship between critical fire load, Q_{crit}, and fire resistance based on the standard fire test (time to failure for F = 0.05), R_s, was determined from the results for normal weight square concrete columns:

$$Q_{crit} = 4 + 10\,R_s \quad \text{(small, medium openings)}$$
$$= 4 + 16\,R_s \quad \text{(large openings)} \qquad (2)$$

This relationship is helpful in assessing Code fire resistance requirements since Q_{crit} can be related to mean fire load for a given occupancy, Q_m, as follows:

$$Q_{crit} = k\,Q_m \qquad (3)$$

where k is a design safety factor. The design fire safety factor is related to the risk of failure which can be determined as a function of the basic requirements of life safety and economic loss (5); for example, a greater value

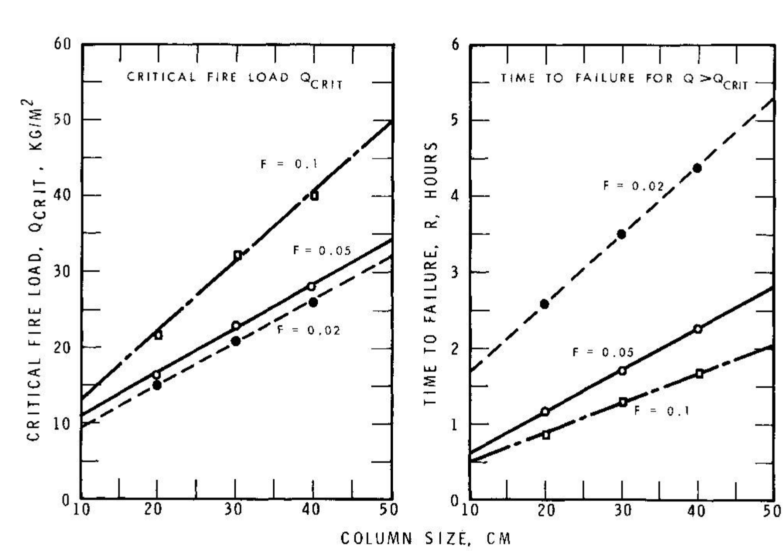

Figure 8: Fire Resistance Parameters For Square Columns (3.75 cm Cover)

of k is required for tall buildings for reasons of life safety and economic loss than for low buildings. Thus, by using Equations (2) and (3) the required fire resistance, R_s, can be related to the basic requirements of life safety and economic loss.

Summary and Conclusions

The fire resistance of square, reinforced concrete columns has been studied under both standard ASTM test conditions and under conditions that more closely resemble those met in practice. Fire resistances are determined by numerical calculation based on the thermal and structural properties of materials at high temperatures. Comparisons with furnace tests of columns show that the calculated fire resistances are consistent with those measured but are about 10 to 20 minutes less, the difference being attributable mainly to conservativeness in the material properties used.

Under standard ASTM test conditions, the fire resistance in time to failure is calculated as a function of type of aggregate, column size, cover, percentage of steel, eccentricity of load, design safety factor and buckling length. Based on calculated results, simple design rules are given (Equation (1)).

The standard ASTM test conditions assume that a column is free to expand, whereas in fact a column is restrained by the surrounding structure and thereby attracts more load than assumed. A numerical study of a restrained column during fire (Fig. 5) indicates, however, that the assumption of no restraint is generally conservative.

Based on the use of more realistic fire temperature curves than the standard ASTM curve, fire resistance was calculated in terms of critical fire load (amount of combustibles) below which no failure takes place, and time to failure if the fire load is greater than critical. Although the critical fire load increases with increased ventilation, the time to failure decreases considerably with increased ventilation. Equations (2) and (3) provide an approximate relationship between standard fire resistance of concrete columns and critical fire load; this is useful in relating Code fire resistance requirements to the fundamental requirements of life safety and economic loss.

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SUMMARY

The fire resistance of square, reinforced concrete columns has been studied under both standard fire test conditions and under conditions that more closely resemble those encountered in practice, including column-structure interaction effects and realistic fire conditions. Based on calculated results, simple design rules are given for the fire resistance as a function of type of aggregate, column size, cover thickness to steel, amount of steel, design safety factor and equivalent buckling strength.

RESUME

On a procédé à l'étude de la résistance au feu des colonnes en béton armé soumises d'une part aux conditions généralement définies par les normes, d'autre part à des conditions qui correspondent mieux à celles rencontrées dans la pratique, en tenant compte des effets d'interaction entre la colonne et la structure, et des conditions d'incendie réalistes. En se basant sur les résultats du calcul, on indique des règles de dimensionnement simples donnant la résistance au feu en fonction du type d'agrégats, de la taille de la colonne, du recouvrement des aciers d'armature, du pourcentage d'armature, du facteur de sécurité et de la résistance équivalente au flambement.

ZUSAMMENFASSUNG

Der Feuerwiderstand quadratischer Stahlbetonstützen wurde sowohl für normierten Brandverlauf, als auch für praxisnahe Bedingungen unter Einschluss der Wechselwirkung zwischen Stützen und Tragwerk und realistischer Brandbedingungen untersucht. Gestützt auf die Berechnungsergebnisse werden einfache Bemessungsregeln gegeben für den Feuerwiderstand in Abhängigkeit der Zuschlagstoffe, der Stützengrösse, der Stahlüberdeckung, des Bewehrungsgehalts, des rechnerischen Sicherheitsfaktors und der äquivalenten Knickfestigkeit.