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Effect of Fire on the Capacity of Reinforced Concrete Columns

L'effet du feu sur la résistance des colonnes en béton armé

Der Einfluss des Feuers auf die Tragfähigkeit von Stahlbetonstützen

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1. INTRODUCTION

Even without considering the effects of fire the behaviour of reinforced concrete columns is complex. The effects of fire are rarely taken into account in design except by satisfying standard requirements for concrete cover over the reinforcing steel. High temperatures change the stress-strain properties of the concrete and steel as functions of the temperature. Therefore areas of cross sections which are close to exposed surfaces are more effected than areas near the centre. Similarly fire causes non-uniform expansion related to the variation of temperature over the cross section.

The effects of fire can be evaluated theoretically provided that sufficient information about the properties of steel and concrete at high temperatures is available. However the previously accepted basis for evaluating strengths of columns based on section capacity and equivalent pinned end lengths may not be accurate or safe. In fires the "softening" of the stiffnesses of exposed columns can allow substantial redistributions of bending moment which will increase their axial load capacity. Conversely the expansion of columns exposed to fire can result in large increases in axial load as a result of the restraining influence of the remainder of the structure.

This study is based on theoretical predictions for strength of individual columns, column sections and columns in frames.

2. THEORETICAL ANALYSIS

2.1 Material Properties

The material properties which directly affect the capacity and behaviour of reinforced concrete sections subjected to fire are discussed below. Unless otherwise specified these properties are based on information from references 1, 5, 6, 7 and 8.

2.1.1 Concrete Stress-Strain Relationships

The concrete stress-strain curve at normal temperatures is shown as the upper curve in Figure 1(a). A continuous equation for this curve has been found(4) based on test data for 27.5 MN/m^2 strength concrete. Concrete is assumed to have no tensile strength.

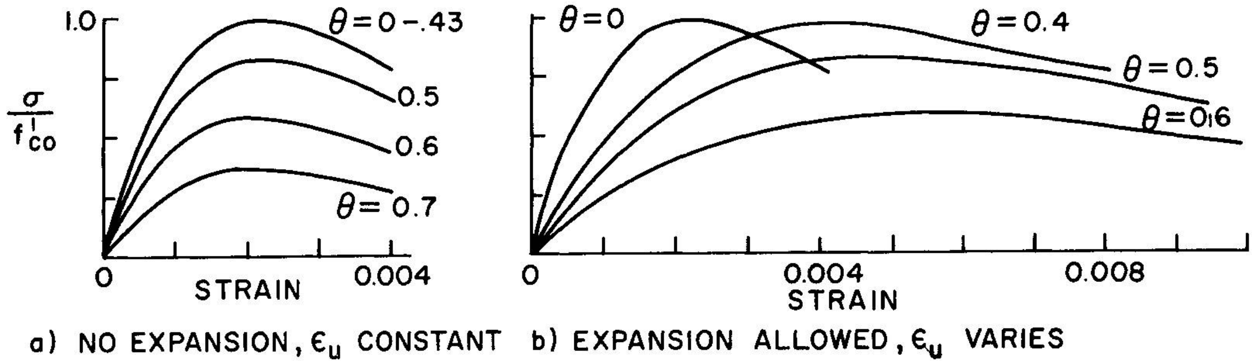


Fig. 1 Concrete Stress-Strain Relations

If the effects of thermal expansion are neglected it was thought reasonable to maintain the failure strain of concrete at a constant value. Therefore it was decided to change the stress for a given strain in proportion to the change in concrete strength, f'_c, which is a function of temperature. The expression for concrete strength as a ratio of the strength, f'_{co}, at normal temperatures is:

$$f'_c/f'_{co} = 1.0 \quad \text{if } \theta < 0.429 \tag{1}$$

$$\text{and } f'_c/f'_{co} = 2.011 - 2.353\theta \quad \text{if } \theta > 0.429 \tag{2}$$

$$\text{where } \theta = \frac{T-20}{1000}$$

The curves corresponding to different values of θ are shown in Figure 1(a).

A more realistic model for the changes in the stress-strain relationship at high temperatures involves increasing the failure strain of the concrete. The equation for failure strains, ε_{cu}, is:

$$\varepsilon_{cu} = 0.002724 + 0.01276\theta > 0.004 \tag{4}$$

The curves in Figure 1(b) show the effect of proportionally modifying the stresses as concrete strength changes and proportionally increasing strains corresponding to a given stress by multiplying the strain by the ratio of failure strains. When these curves are used the effects of thermal expansion should be included.

Use of both of the constitutive relations shown graphically in Figure 1 yield very similar stress distributions and section capacities. However very different deformations result. These differences will have significant effects on the structural analysis.

Creep of the concrete at high temperatures can have a significant effect for long columns and for columns with large eccentricities of loading. No attempt was made to include the effect of creep in this study. The relations for ε_{cu}, however, do contain some creep since they are based on tests carried out over a short period. The effects of temperature history and load history are neglected in the analysis due to lack of information.

2.1.2 Steel Stress-Strain Relationships

An idealized elastic-plastic stress-strain curve for steel having a yield stress of 412.5 MN/m^2 was used in this study. Figure 2 contains the graphical representation of the reduction in yield stress and modulus of elasticity with increased temperature.

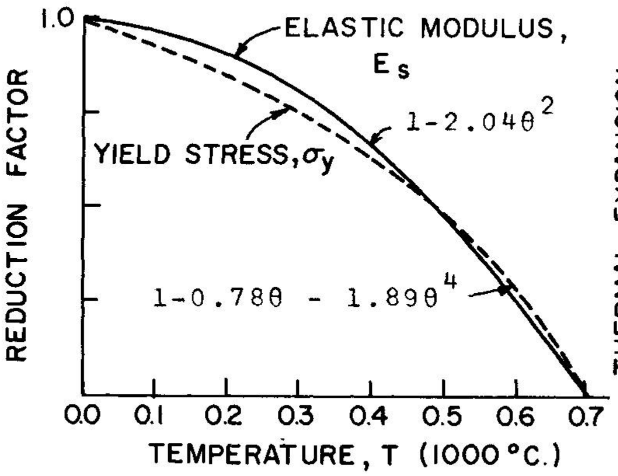


Fig. 2 Effect of Temperature on Steel Stress-Strain Relations

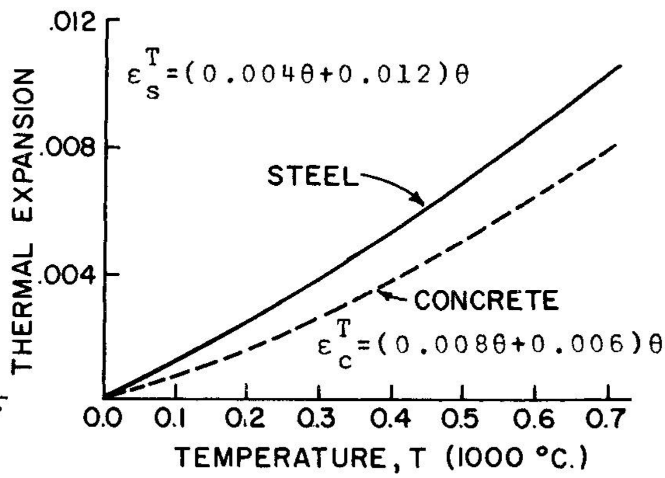


Fig. 3 Thermal Expansion of Concrete and Steel

2.1.3 Thermal Expansion

The coefficients of thermal expansion for concrete and steel are not constant at high temperatures. The curves for thermal expansion are shown in Figure 3.

2.1.4 Temperature Distribution in Cross Sections

The temperature distribution in concrete sections were obtained from T.T. Lie(8) of the National Research Council of Canada. Temperatures at the centres of 2.5 cm square elements of a grid were provided for various sizes of square columns. These columns were assumed to be exposed on all sides to a fire which produces temperatures coinciding with the temperature course for the standard fire test described in ASTM E119-71(3). The temperature in the reinforcing steel was taken to be the same as that for concrete at the same position in the cross section.

2.2 Analysis of Cross Sections

The temperature at the centre of each 2.5 cm square element of the cross section and each reinforcing bar were calculated for any desired period of exposure to the assumed fire. Then for any plane strain distribution on the section the stress at the centre of each element can be found using the constitutive relations for the concrete and steel. The effect of fire on the resistance of the section can be calculated by transforming the area of each element according to the change in elastic modulus for that element.

By cross section mechanics the forces on each element and the moments of these forces about mid depth are summed to give the total internal forces. These are then compared to the applied forces. Successive corrections are made to the magnitude and slope of the plane strain distribution until the calculated internal forces balance the applied forces to within 1 percent. Failure of the section is defined when the internal forces cannot be increased to balance the applied forces.

2.3 Analysis of Individual Columns

Columns with eccentric load applied at pinned ends are analysed to provide information on the PΔ effect. The column is divided into a number of sub-member lengths. The axial deformation and curvature for each sub-member are taken as the average of the calculated values of its end sections. Using an iterative sequence the additional bending moments due to column deflection are included in the analysis.

2.4 Analysis of Frames

For the structural analysis of frames exposed to fire, it is convenient to be able to model section properties in a way which is common to normal analytical methods. For a specific set of forces and duration of fire, the relationships between these forces and the resulting deformations on the section can be represented by equivalent stiffnesses EA and EI. The axial stiffness, EA, of the section is the sum of the stiffness of each of the elements where the modulus of elasticity is the secant modulus for stresses which occur on each element when internal forces balance the applied forces. The flexural stiffness, EI, is found by summing the product of the moment of inertia of each element about the centroid of the section multiplied by the corresponding secant modulus of elasticity.

A computer program for elastic stiffness matrix analysis was modified to permit member stiffnesses to be changed as a result of calculation of the effects of high temperatures. Members selected to be exposed to fire are divided into sub-members to improve the accuracy of the calculation of equivalent stiffnesses for different combinations of axial load and bending moment.

Where thermal expansion is included, the forces due to expansion must be applied at the ends of each affected member. These forces are re-calculated after each iterative cycle for calculation of EA and EI values. Application of these forces to the frame at the ends of the expanding members causes the expected increase in length and depending upon the stiffness of the frame can cause significant redistribution of forces. The structural analysis program is said to have converged when the forces and stiffnesses in every sub-member do not change with successive iterations. Failure is defined by cross section failure as described in section 2.2.

3. ANALYTICAL RESULTS

3.1 Cross Section Capacities

A 40 cm square column with 3.5% reinforcement is used to study the influence of fire on failure load. Figure 4 contains values of predicted capacities for different durations of fire. Load is applied at eccentricities of 0.1h, 0.4h and 0.7h for nominal covers over the reinforcement of 1.90 cm, 3.80 cm, and 6.35 cm (corresponding to depths to the centroid of reinforcing bars of 4.75 cm, 6.65 cm and 9.20 cm).

At like eccentricities and like concrete covers the decreases of cross section capacity with increased exposure to fire are similar for other percentages of steel. The capacities approached the same values at the time when the strength of the steel approached zero. Expansion is neglected in the results shown in Figure 4. Inclusion of expansion gave nearly the same results.

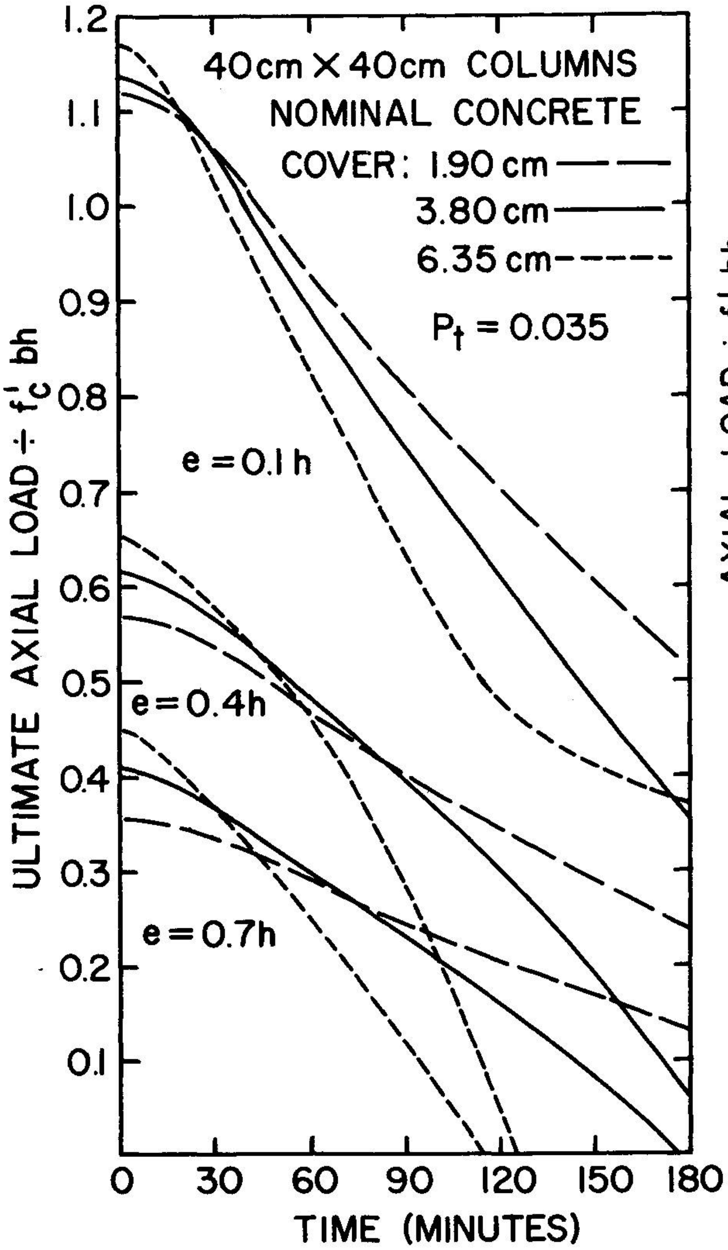
3.2 Individual Column Capacities

Results for pinned end columns for the same eccentricities as above are shown in Figure 5 for different slendernesses. Only the results for 3.5% steel and 3.80 cm nominal cover are shown. The differences in capacity for the two times of exposure to fire found by comparing these results with corresponding section capacities in Figure 4 provide a measure of the secondary moment effect.

3.3 Capacities of Columns in Frames

Several frames have been analysed[9] for various combinations of loading and columns exposed to fire. Figure 6 contains a sketch of a building with design levels of gravity load and wind load. Only columns 1 and 2 are exposed to fire. Analytic results are shown both with and without the effect of thermal expansion. Without expansion, the axial loads remain nearly constant and the moments in columns exposed to fire decrease substantially.

Inclusion of expansion causes some modification to the redistribution of moments. However the main effect is change in axial load on the columns. With expansion the capacity of column 2 is exceeded just prior to 3 hours exposure.



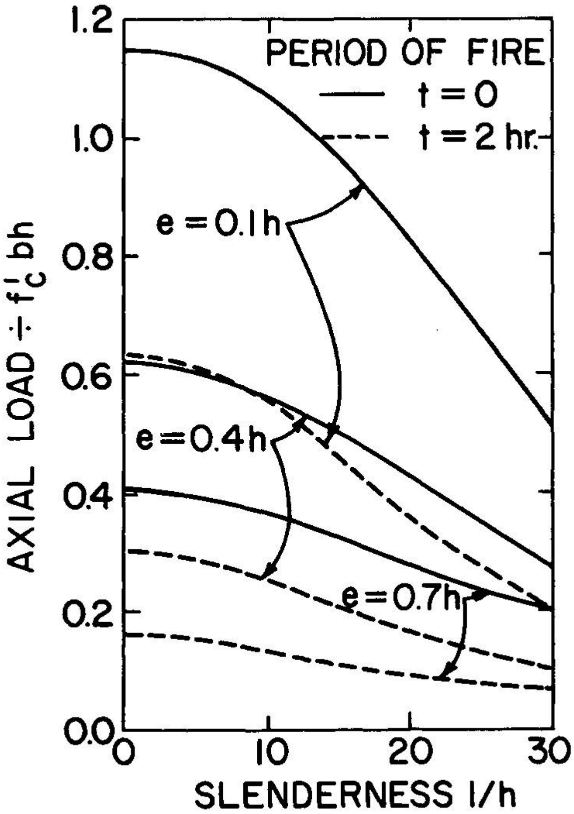


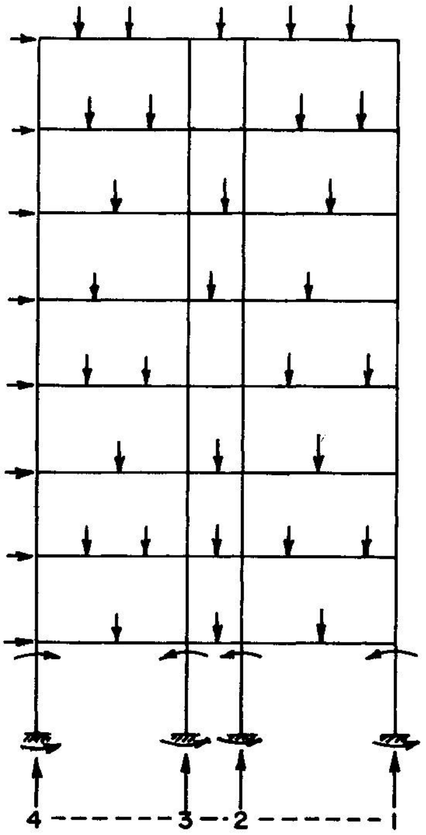
Fig. 5 Effect of Fire on Capacity of Individual Columns

Fig. 4 Effect of Fire on Cross Section Capacity

4. CONCLUSIONS

The capacity of column cross sections is most affected by concrete cover over the reinforcement. Except for small eccentricities, no capacity remains at 3 hours exposure for the standard nominal cover of 3.80 cm. For all cases the 40 cm square section retains less than half of the original capacity after 3 hours exposure.

Analyses of individual long columns show the influence of increased secondary bending resulting from reduced stiffness due to fire. If the forces on a column can be assumed to be unaffected by thermal expansion or by changes in the stiffnesses of members, capacities can be adequately predicted using this type of analysis. It is suggested that the above assumptions are not normally valid.



Column Force		Period of Exposure to Fire (hrs)						
			Without Expansion			With Expansion		
		0	1/2	2	3	1	2	3
1	P	25.5	25.5	25.7	26.0	23.8	24.8	
	M_L	4.5	4.3	2.8	1.7	1.6	1.3	
	M_u	4.9	4.7	3.2	2.0	3.1	2.2	
2	P	36.0	35.9	34.5	32.5	51.2	44.4	failure
	M_L	1.3	1.2	0.8	0.5	0.7	0.7	
	M_u	1.3	1.2	0.4	0.2	1.5	0.7	
3	P	34.3	34.4	35.7	37.7	18.5	25.0	
	M_L	4.4	4.5	5.4	6.2	4.4	5.3	
	M_u	4.9	4.9	5.4	5.6	9.0	8.2	
4	P	27.3	27.3	27.0	26.8	29.7	28.8	
	M_L	0.1	0.1	1.0	1.9	1.9	0.4	
	M_u	3.7	3.6	3.1	2.7	3.3	2.8	

P = (*Vertical Reaction* $\div f'_c bh$) x100
M_L = (*Base Moment* $\div f'_c bh^2$) x100
M_u = (*Upper Column Moment* $\div f'_c bh^2$) x100

Fig. 6 Column Forces in a Building Exposed to Fire

The ability of columns in frames to support the structure are dependent on the restraining effect of the structure, the loading combinations and the portion exposed to fire. In many cases moments in exposed columns decrease thereby increasing the axial load capacity. However expansion causes substantial increases in axial load on some exposed columns which increases the likelihood of failure.

It is the authors' opinion that analyses or standard tests(3) of individual columns exposed to fire can only provide an approximate and not necessarily safe estimate of the effect of fire. Design provisions must be based on consideration of the behaviour of columns in structures.

It is important to note that much more research is required to better define the affect of fire on the material properties.

5. NOTATION

The notation used corresponds to that specified in the Introductory Report for this Symposium. Additional symbols are defined where they first appear in the text.

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SUMMARY

Numerical analyses have shown that strengths of individual reinforced concrete columns are dramatically reduced when exposed to fire. However the redistribution of forces in frames is shown to partially offset the loss of strength. This evidence and analyses of the effects of thermal expansion raise questions about the validity of present day evaluations of columns exposed to fire.

RESUME

Les résultats d'analyses numériques ont révélé que la résistance de colonnes individuelles en béton armé est de façon critique lorsque celles-ci sont exposées au feu. Cependant la redistribution de forces dans la structure compense partiellement cette perte de résistance. Ce phénomène ainsi que l'analyse d'effets de dilatation thermique conduisent à certaines questions sur la validité du calcul actuel des colonnes exposées au feu.

ZUSAMMENFASSUNG

Numerische Berechnungen haben gezeigt, dass eine kritische Verminderung der Tragfähigkeit von einzelnen bewehrten Betonsäulen stattfindet, sobald diese dem Feuer ausgesetzt werden. Jedoch wird dieser Verlust an Tragfähigkeit bis zu einem gewissen Grade kompensiert durch eine Umlagerung der Kräfte in Rahmentragwerken. Diese Tatsache und Untersuchungen über den Einfluss der Wärmedehnungen stellen die üblichen Vorschriften für Bemessung bewehrter Betonsäulen in Frage.