Inelastic behaviour of protected steel columns in fire

Inelastic Behaviour of Protected Steel Columns in Fire

Comportement inélastique des poteaux protégés contre l'incendie

Nichtelastisches Verhalten von feuergeschützten Stahlstützen

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1. Introduction

The fire resistance of structural members has been determined in the past by subjecting isolated members to standard fire tests. More recently, calculations based on heat transfer and the structural properties of materials at high temperature have been used to replace the costly and time-consuming standard fire tests. In the past the cost of testing precluded considerations of assuming more realistic situations corresponding to columns in continuous structures subjected to fires representing actual conditions. These more realistic situations can now be studied by computer calculation.

In this work three aspects for protected steel columns are studied: (1) the stress distribution in a column during fire, (2) the interaction of an interior column exposed to fire with the surrounding building structures, and (3) the fire resistance of columns which take into account creep deformation behavior.

This work is a continuation of Ref.1 and the method of analysis regarding determination of stress, strain and deformation of the column are contained therein.

2. Calculation of Temperature Rise

A computer program was written to calculate the temperatures with in a column when the temperature outside four faces was raised in the manner specified for fire tests in JIS A 1304.

Most columns used in buildings have sections less than 15 cm thick. In such cases detailed calculation of temperature distribution in the steel cross section is unnecessary, and one-dimentional model of the heated column can be usefully employed.[2] The model consists of a steel plate having the same cross-sectional and surface areas per unit height as the four sides of the heated column, with the edges and unexposed side perfectly insulated. This model permits use of a one-dimentional numerical procedure.

In calculating the temperatures, furthermore, it was assumed that fire protection panels became divided into a dry zone and a wet zone, within each of which the thermal properties were taken to be constant.

It was also assumed that vaporization occured at atmospheric pressure at the interface between these zones, and that therefore interface moved away from the heated face as drying penetrated into the protection. Typical results, for the temperatures in H cross-section steel columns with 3 or 5 cm thick protection, are shown in Figures 1 and 2.

3. Stresses and Strains in a Column during Fire

To analyse the strain and stress in a column section, the following assumptions were made:

(1) Plane sections remain plane. This assumption is approximately correct for long prismatic members in continuous construction.

(2) The free expansion of steel due to temperature change, ε_T, is assumed as follows (see Fig.3) :

$$\varepsilon_T = 5.04 \times 10^{-9} T^2 + 1.13 \times 10^{-5} T \quad \cdots\cdots\cdots\cdots \quad (1)$$

where T is in degrees Centigrade.

(3) The relationship between steel stress σ, and strain ε, in compression, is assumed as follows (see Fig.4) :

$$\varepsilon E_0(T) = C(T)\sigma - [1 - C(T)]\sigma_y(T) \ln[1 - \sigma/\sigma_y(T)] \quad \cdots\cdots (2)$$

where

$$E_0(T) = 2.15 \times 10^6 [0.745 + 0.137 \ln[-0.01T + 6.3]] \quad (kg/cm^2)$$

$$\sigma_y(T) = 5.0 \times 10^3 [-4.49 \times 10^{-9} T^3 + 3.57 \times 10^{-6} T^2 - 1.41 \times 10^{-3} T + 1.027] \quad (kg/cm^2)$$

$$C(T) = -4.0 \times 10^{-4} T + 1.0$$

In this expression $\sigma_y(T)$ is the steel yield point (SM 58), $E_0(T)$ is the initial tangent modulus and C(T) is a parameter that affects the form of the stress-strain curve as shown in Figure 4, in which T is expressed in degrees Centigrade. Besides these data, the material description must include behavior during unloading and in tension. This is arranged by making the following two assumptions: (1) Behavior in tension is the same as that in compression, and (2) Behavior during unloading from (or reloading to) a previously obtained value of a compressive stress given by equation 2 is linear $d\sigma/d\varepsilon = E_0(T)$, the initial tangent modulus.

(4) To evaluate the creep deformation, it is assumed that the creep strain for SM 58 is related to time t, absolute temperature T′, and current stress σ in the form such that (see Fig.5)

$$\varepsilon_c = [10^{c - a/2.3T'}] \times [[2.37\sigma/5.0]^{b/2.3T' + \alpha}] \, t^n/n \quad \cdots (3)$$

where $c = 20.53$ $b = 1.9 \times 10^4$ $\alpha = -7.25$ $n = 0.35$ $a = 4.5 \times 10^4$

t: minutes σ : kg/mm^2 ε_c : %

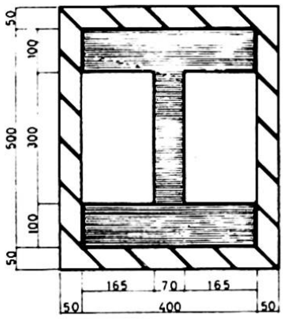
The strain-hardening creep low is applied for the calculation of the nonstationary creep deformation.

(5) The column is divided into blocks with 1/20 or 1/40 column length in the axial direction and each block is subdivided further into 20 elements in the cross sectional direction. For each element the strain, stress and material properties are assumed uniform.

(6) Initial stresses and strains in the cross-section before the fire, are determined, assuming that the axis of the column has initially the form of a sine curve which takes into account the various imperfections in a column.

(7) It is assumed that there are no cases in which a steel column subjected to compression will buckle torsionally or locally and the effect of shearing force on the deformation can be neglected. Buckling of the column will occur in the plane of minimum flexural rigidity.

Figure 1 SECTION



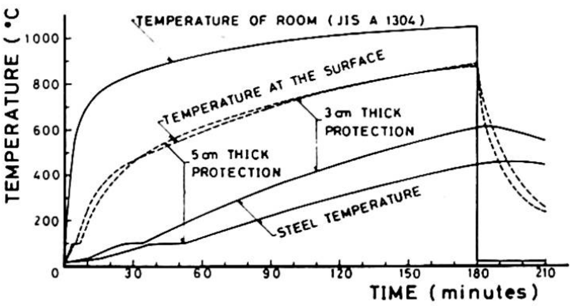
STEEL COLUMN
(SM 58)

Young's modulus	2.15×10^6 Kg/cm²
Strain corresponding to the yield stress	2.50×10^{-1} %
Yield stress	5.00×10^3 Kg/cm² (20°C)

FIRE PROTECTION
(CONCRETE)

Thickness	3.0 or 5.0 cm
Percentage of moisture content	4.0 %
Specific gravity	2340 Kg/m³

Figure 2 CALCULATED TEMPERATURE OF PROTECTED STEEL



FREE THERMAL STRAIN-TEMPERATURE CURVE
Figure 3

$$\varepsilon_T = 5.04 \cdot 10^{-9}\theta^2 + 1.13 \cdot 10^{-5}\theta$$

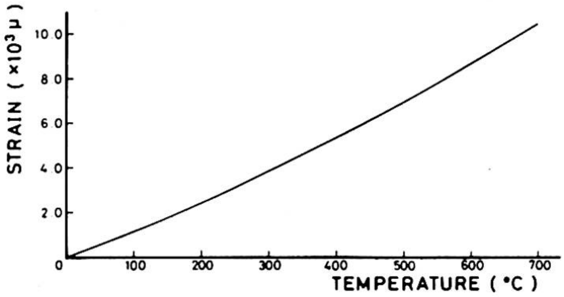


Figure 4 ASSUMED STRESS-STRAIN CURVE
(based on Harada and Furumura's Data)

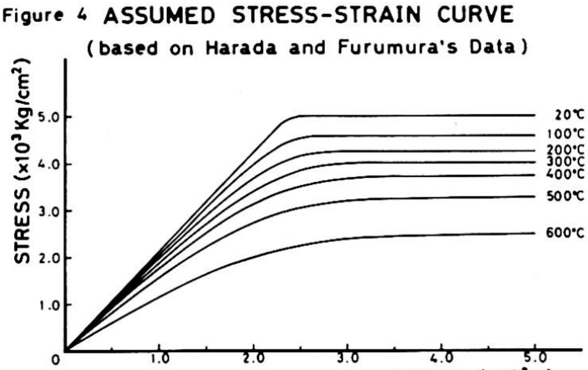
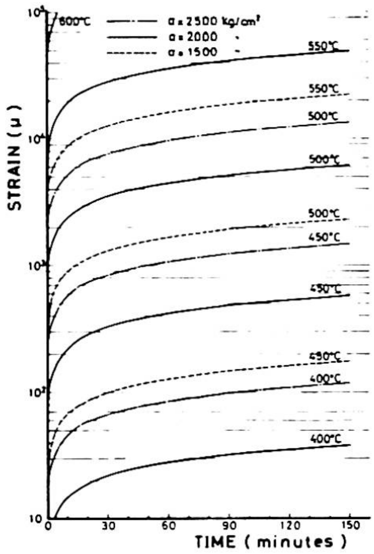
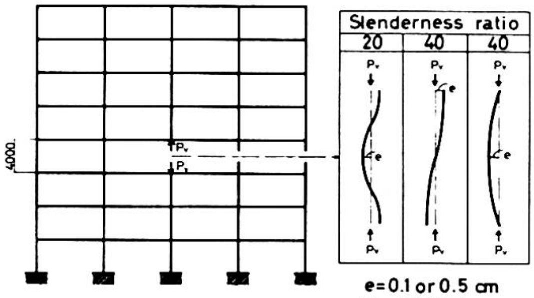


Figure 5 NONSTATIONARY CREEP

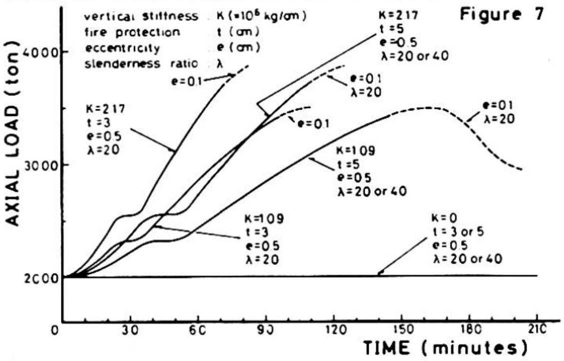


STRUCTURAL ARRANGEMENT OF BUILDING FRAME
Figure 6 AND HEATED COLUMN

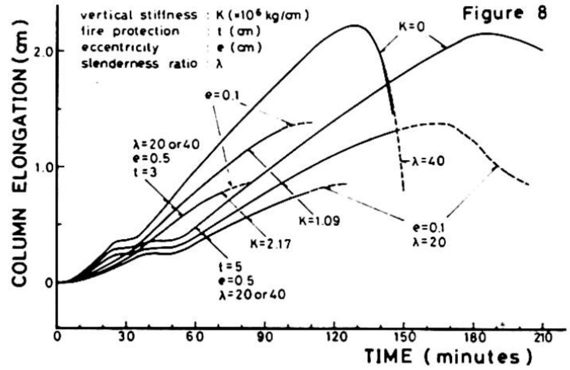


e=0.1 or 0.5 cm
Pv=2000 ton

COLUMN-STRUCTURE INTERACTION DURING FIRE
Figure 7



COLUMN-STRUCTURE INTERACTION DURING FIRE
Figure 8



Based on these assumptions, the distribution of strain and stress is calculated for each 5-minute interval during fire. The method of analysis is to determine, by trial and error, a linear strain distribution in the steel which provides equilibrium of the cross-section.

4. Interaction of a Single Column and Surrounding Structure

A column expands during a fire and, because of the resistance of the surrounding structure to this expansion, more than the initial dead plus live load is attracted to the column.

If every column in a floor were subjected to the same fire, all the columns would expand equally and additional load would not be attracted to any one column. If only one of the floor columns were exposed to fire, the expansion of the column would be resisted by the surrounding structure. The greater the number of slabs above the column, the greater the resistance, and the greater the load attracted to the column.

The correct solution to the interaction problem requires a trial and error determination of moments, curvatures and displacements along the column for each point in time.

5. Results of Calculation

It is clear that a combination of axial load and axial and flexural restraint would arise with the structural arrangement shown in Figure 6.

To calculate lateral deflection, two cases of the flexural restraint for H cross-section steel column are assumed: (1) The column has both ends built in, and (2) The column is built in at the lower end and is free to displace laterally at the upper end but is guided in such a manner that the tangent to the deflection curve remains vertical, and that is equivalent to the case of a column with hinged ends.

The former assumption is considerably in error early in a fire during elastic conditions, but appears to be approximately correct as the column approaches failure when it bends with large inelastic strains at the critical sections. The latter is considered to be corresponded to the case in which every column in a floor would be subjected to the same fire.

Figure 7 shows how the applied load (initial load= 2000 ton/a column) increases by the interaction effect. Figures 8 and 9 to 11 show the corresponding column lengthening and lateral deflection, respectively.

The interaction effect is shown clearly in Fig.7. The curve labelled "K=0", which corresponds to the assumption of no ineraction, serves as a basis for comparison. As the vertical stiffness of the surrounding structure increases from 0 to $1.09\text{X}10^{6}$ kg/cm, which corresponds to 1/5 vertical stiffness of a column with 4 m length, the applied load more increases and the fire resistance is decreased, as seen by comparing column movements. As the vertical stiffness is increased further to $2.17\text{X}10^{6}$ kg/cm, the fire resistance is more decreased. The fire resistance is also effected by the protection thickness.

Figures 12 to 14 show the variations of the strain, stress and tangential modulus in the extreme element on the convex side of the critical section. Figure 15 and 16 show the variations of the curvature distribution of the columns.

From the numerical results, the column is not greatly affected by creep until the steel temperature reaches 450°C. After 450°C,

COLUMN-STRUCTURE INTERACTION DURING FIRE
Figure 9

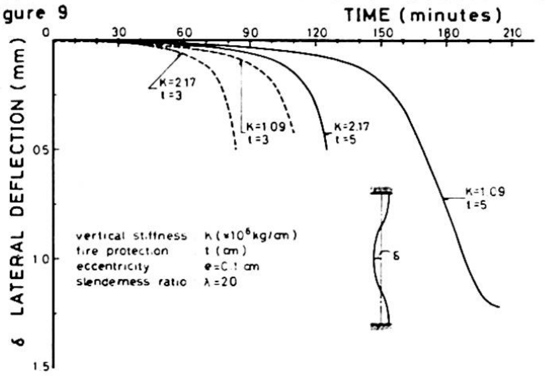
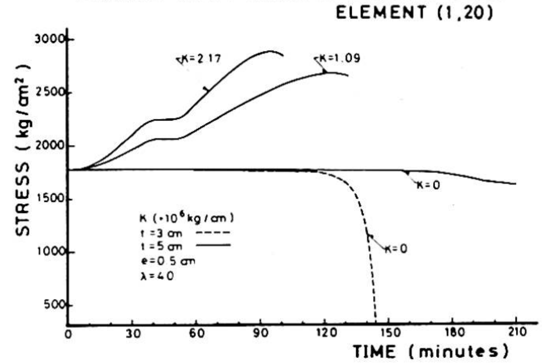


Figure 13 STRESS-TIME CURVES DURING FIRE
ELEMENT (1,20)



COLUMN-STRUCTURE INTERACTION DURING FIRE
Figure 10

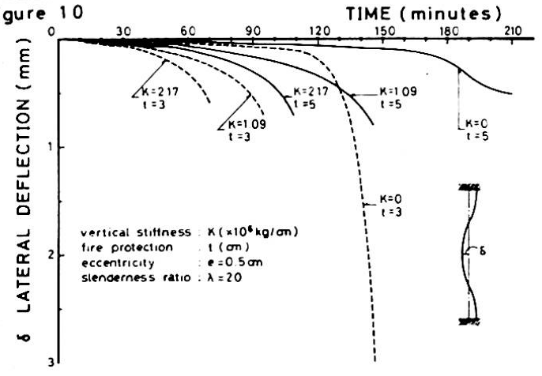
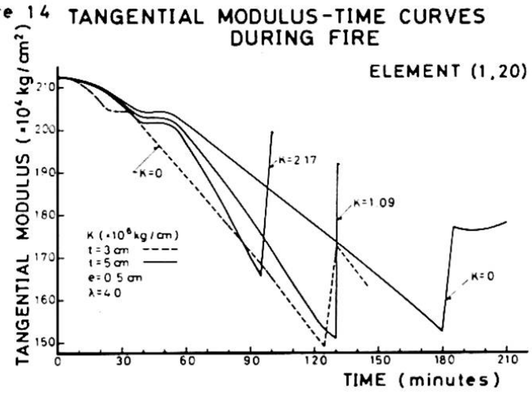
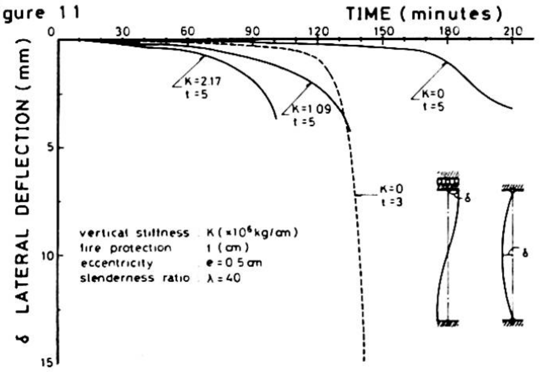


Figure 14 TANGENTIAL MODULUS-TIME CURVES DURING FIRE
ELEMENT (1,20)



COLUMN-STRUCTURE INTERACTION DURING FIRE
Figure 11



CURVATURE DIAGRAMS due to BENDING MOMENT
Figure 15

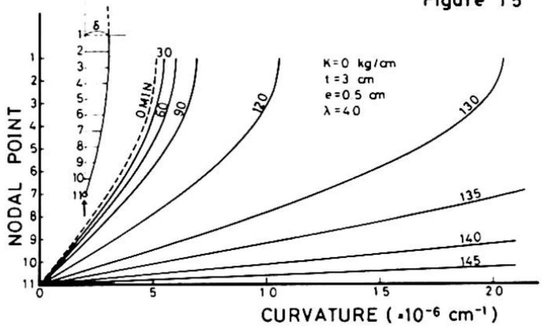
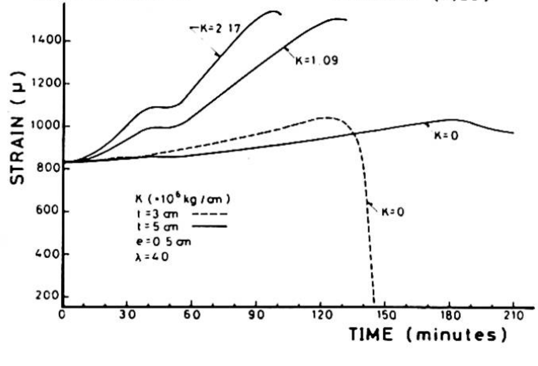
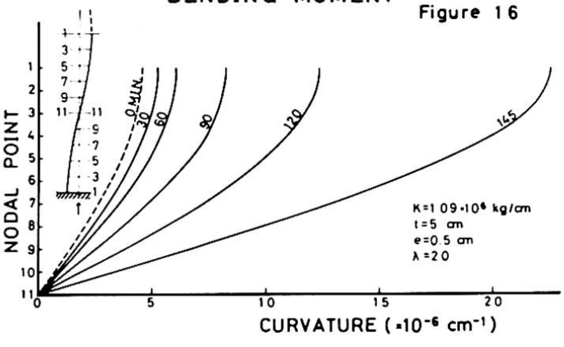


Figure 12 STRAIN-TIME CURVES DURING FIRE (due to stress)
ELEMENT (1,20)



CURVATURE DIAGRAMS due to BENDING MOMENT
Figure 16



the creep rate increases rapidly and the column causes chord shortening, followed by lateral bending and relief of the applied load.

For very stiff vertical resistance the column buckles quite early, and the greater the initial deflection, the earlier the buckling of the column.

6. Conclusions

This study represents an initial attempt to evaluate in detail the behavior of a steel frame structure in a fire.

The creep deformation of columns in transient heating processes is of considerable interest to engineers working in the field of fire protection.

A numerical technique has been described in Ref.(1). The technique utilized a creep model was originally suggested by England[3] and Dougill[4] and expanded by the authers to suit certain practical requirements. The calculations that have been undertaken have been based on an idealized model of material behavior and it is doubtful whether the numerical values obtained have any significance in themselves. However, some aspects being important to understand modes of behavior of columns in fire have been found out and the information obtained from such detailed analysis is very useful in the future development of a simple and rational procedure for fire safety design of high rise steel structures.

7. References

(1) Furumura,F., and Konishi,T.,"Inelastic Behavior of Reinforced Concrete Wall Exposed to High Temperatures", Report of the Research Laboratory of Engineering Materials,No.1,Tokyo,1976, pp.31-47.

(2) Stanzak,W.W., and Lie,T.T.; ASCE, Journal of the Structural Division,May.1973, pp.837-852.

(3) England,G.L., and Ross,A.D.; Magszine of Concrete Research, V.14,No.40,Mar.1962, pp.5-12.

(4) Dougill,J.W.; Magazine of Concrete Research,V.24,No.80,Sep. 1972, pp.139-148.

SUMMARY

The study shows the inelastic behaviour of protected steel columns under fire action. It is shown that the effect of creep on the buckling strength is very important at temperatures more than 450° C.

RESUME

L'étude s'occupe du comportement inélastique des poteaux métalliques protégés contre l'incendie. Il est montré que l'influence du fluage est très importante pour les températures supérieures à 450° C.

ZUSAMMENFASSUNG

Das nichtelastische Verhalten von geschützten Stahlstützen bei Brandbeanspruchung wird untersucht. Es wird gezeigt, dass dem Kriechen bei Temperaturen über 450° C grosse Bedeutung zukommt.