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New Developments in Fire Resistance of Concrete Filled Steel Tubes in China

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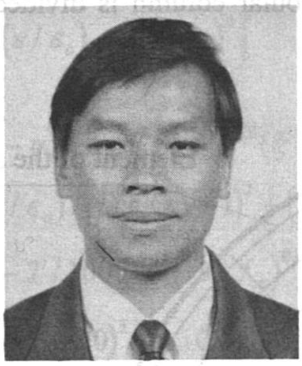

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Summary

Finite element method is applied for the calculations of temperature fields of concrete filled steel tubes under fire. A theoretical model that calculates deformations and strength of column in fire and fire resistance is described in this paper. A comparison of results calculated using this model with the results of tests, there is good agreement. Based on the theoretical model,influence of the changing strength of the materials,dimater,steel ratio and slenderness ratio on the fire resistance is discussed.

1. Introduction

Concrete filled steel tubular columns have been used extensively in China as well as other countries,they have proved to be economical in themselves as well as leading to rapid construction and thus additional cost savings. An important criterion for the design of concrete filled steel tubes,besides the serviceability and critical load bearing capacity,is an adequate fire resistance.

Research to determine the fire resistance of concrete filled steel tubular column has been carried out in several countries in the world[1-7]. In recent years,Harbin University of Civil Engineering and Architectures (HUCEA) has been engaged in research to calculate the fire resistance of concrete filled steel tubular columns with the support of the Chinese Natural Science Foundation. Both theoretical and experimental studies were carried out[8,9]. Most of the HUCEA has been carried out on columns with circular cross-section,and the columns have been subjected to axial compression or eccentric compression loads.

The composite action between the steel and concrete has been considered,which was often neglected by other researchers in the theoretical analysis. A theoretical model that calculates the strength and the fire resistance of the columns is described in this paper,and influence of the changing parameters of the column on the fire resistance are analyzed.

2. Strength of Columns During Fire Exposure

2.1 Division of Cross-Section

To calculate the temperatures, deformations and stresses in the columns and its strength,the cross sectional area of the concrete filled steel tubular column is divided into a number of annular elements,show as in Figure1.

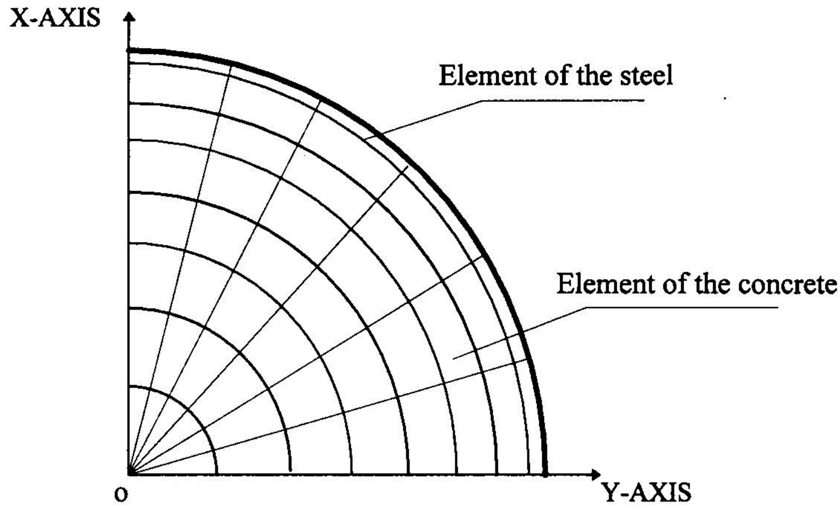


Figure 1 Arrangement of elements in quarter section

2.2 Temperature of Columns During Fire

The column temperatures are calculated by finite element method. The method for deriving the heat transfer equations and calculating temperatures ,with the thermal properties is described in detail in Reference(8) . The temperature of an element in Figure 1 is assumed to be equal to the temperature at its center.

2.3 Stress-Strain Relations of the Steel and Concrete

The basic reason that concrete filled steel tubular structures differ from tubular steel structures ,is that there exists transverse confining force between the steel tube and core concrete because of their different transverse deformations. The confining force is passive,the steel and concrete are all in tri-axial stress states. This factor must be considered for the determinations of the stress-strain relations of the steel and core concrete properly.

2.3.1. Stress-Strain Relations of the Steel

Figure 2 shows relations of the stress strength σ_i and strain strength ε_i. Details for the determinations of the parameters in Figure 2 , such as $f_y(T)$, $\varepsilon_y(T)$, $f_u(T)$ and $\varepsilon_u(T)$ are found in Reference (9) and Reference(10).

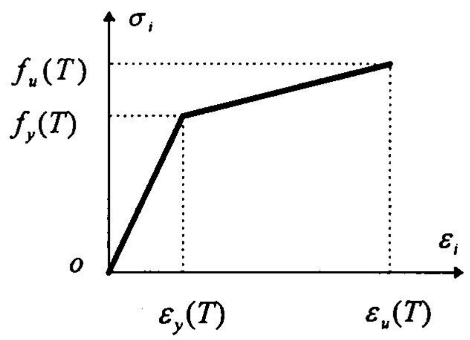


Figure 2 $\sigma_i - \varepsilon_i$ relations of the steel

2.3.2. Stress-Strain Relations of the Concrete

Based on the tests results of concrete filled steel tubular axial short columns under constant high temperature,the relations between longitudinal stress σ_c and longitudinal strain ε of the core concrete has been deduced,i.e.

If $\varepsilon \le \varepsilon_o$, then,

$$\sigma_c = \sigma_o\left[A(\varepsilon/\varepsilon_o) - B(\varepsilon/\varepsilon_o)^2\right] \tag{1a}$$

If $\varepsilon > \varepsilon_o$, then,

$$\begin{aligned} \sigma_c &= \sigma_o(1-q) + \sigma_o q(\varepsilon/\varepsilon_o)^{0.1\xi} (\xi \ge 1.12) \\ \sigma_c &= \sigma_o(\varepsilon/\varepsilon_o)/\left[\beta(\varepsilon/\varepsilon_o - 1)^2 + (\varepsilon/\varepsilon_o)\right](\xi < 1.12) \end{aligned} \tag{1b}$$

where, $\sigma_o = f_{ck}(T)\left[1.194 + (1 - T/1000)^{9.55}(13/f_{ck})^{0.45}(-0.07485\xi^2 + 0.5789\xi)\right]$

$f_{ck}(T) = f_{ck}/\left[1 + 1.1986(T-20)^{3.21} \times 10^{-9}\right]$;

$\varepsilon_o = \varepsilon_{cc}(T) + \left[1400 + 800{(f_{ck} - 20)}/{20}\right]\xi^{0.2}(\mu\varepsilon)$;

$\varepsilon_{cc}(T) = (1 + 0.0015T + 5\times 10^{-6}T^2)(1300 + 14.93 f_{ck})(\mu\varepsilon)$;

$A = 2 - k; B = 1 - k; k = 0.1\xi^{0.745}; q = k/(0.2 + 0.1\xi)$;

$\beta = (2.36 \times 10^{-5})^{\left[0.25 + (\xi - 0.5)^7\right]} f_{ck}^2 \times 5 \times 10^{-4}$;

$\xi = \alpha f_y / f_{ck}; \alpha = A_s / A_c$;

A_s, A_c =sectional area of the steel tube and core concrete respectively;

f_y, f_{ck} = strength of the steel and concrete respectively, $f_{ck} = 0.67 \times f_{cu}$;

f_{cu} = cubic strength of the concrete.

Composite action between the steel and its core concrete have considered in Equation(1).

2.3.3. Composite Stress -Strain Relations of Concrete Filled Steel Tubes

The 'composite' relations of longitudinal stress $\bar{\sigma}$ (=*N/Asc*,where,*N* is the load of axial compression,*Asc=As+Ac*) and longitudinal strain ε of the concrete filled steel tube under compression can be calculated by the stress-strain relations of the steel and concrete,in the calculation,equilibrium equation of force and compatibility condition of deformations should be satisfied. Figure3 shows a typical $\bar{\sigma} - \varepsilon$ curve. Details of the calculation method are found in Reference(9).

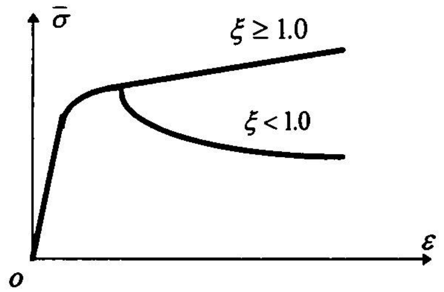
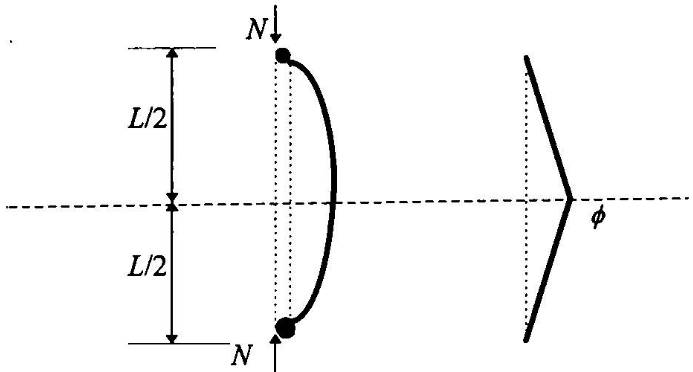


Figure 3 Typical $\bar{\sigma} - \varepsilon$ curve

2.4 Calculation of Strength During Fire

The calculation of the fire resistance of the column involves the calculation of the temperatures of the fire ,to which the column is exposed, the temperature in the column and its deformations and strength during the exposure to fire. Figure 4 shows deflection of the column under load of N exposure to fire, L is the length of the column, ϕ is curvature at midheight of the column.



(a) deflection *(b) curvature*

Figure 4 Deflection and curvature of column exposure to fire

Exposure to fire,the strength of the column decreased with the duration of exposure. A numerical model was worked out for the analysis of the ultimate load,the model allows a differentiated consideration of all physical and geometrical non-linearity.

In this method,for the calculation of column strength, the following assumptions were made: (1).The cross sections remain plane in the fire exposure. (2). The shape of the deformation of the member is regarded as semi-sine curve. (3). Concrete has no tensile strength. (4). the stress of the steel and concrete in compressive zone is equal to that of the concrete filled steel tubes corresponding to the same strain; the steel in tensile zone is in uni-stress state.

Based on assumption(1),the strain of the elements of the steel can be given by:

$$\varepsilon_s = \phi x_{si} + \varepsilon_o - \varepsilon_{sT} \tag{2}$$

and for the element of the concrete can be given by:

$$\varepsilon_c = \phi x_{ci} + \varepsilon_o - \varepsilon_{cT} \tag{3}$$

where, ε_o=axial strain of the column; $\varepsilon_{sT}, \varepsilon_{cT}$=strain of the steel and concrete due to thermal expansion; x_{si}, x_{ci}=horizontal distance from the center of the element of the steel and concrete to a vertical plane through X-AXIS of the column section respectively. The curvature ϕ at midheight of the column,which can be derived from assumption(2) as followings:

$$\phi = (\pi^2 / L^2) u_m \tag{4}$$

where, u_m=deflection at midheight of the column.

Internal axial load of the column section at misheight is:

$$N_{in} = 2\sum_{i=1}^{n}(\sigma_{sli} dA_{si} + \sigma_{ci} dA_{ci}) \tag{5}$$

where, $\sigma_{sli}, \sigma_{ci}$=longitudinal stresses of the steel and concrete element respectively; From assumption(3) and (4),in tensile zone of the section, σ_c =0,and σ_{sl} can be determined according to Figure 2; in compressive zone, σ_c can be determined according to equation(1), σ_{sl} can be deduced as followings:

$$\sigma_{sl} = \left[\bar{\sigma}(A_s + A_c) - \sigma_c A_c\right] / A_s = \left[\bar{\sigma}(1+\alpha) - \sigma_c\right] / \alpha \tag{6}$$

Internal bending moment of the column section at midheight is:

$$M_{in} = 2\sum_{i=1}^{n}(\sigma_{sli}x_{si}dA_{si} + \sigma_{cli}x_{ci}dA_{ci}) \tag{7}$$

For any given curvature ϕ, and thus for any given deflection u_m at midheight of the column, the axial strain ε_o is varied until the internal moment at the midsection is in equilibrium with applied moment, which is:

$$M_{in} / N_{in} = e_o + u_m \tag{8}$$

where, e_o =initial load eccentricity , or arbitrary load eccentricity of the column.

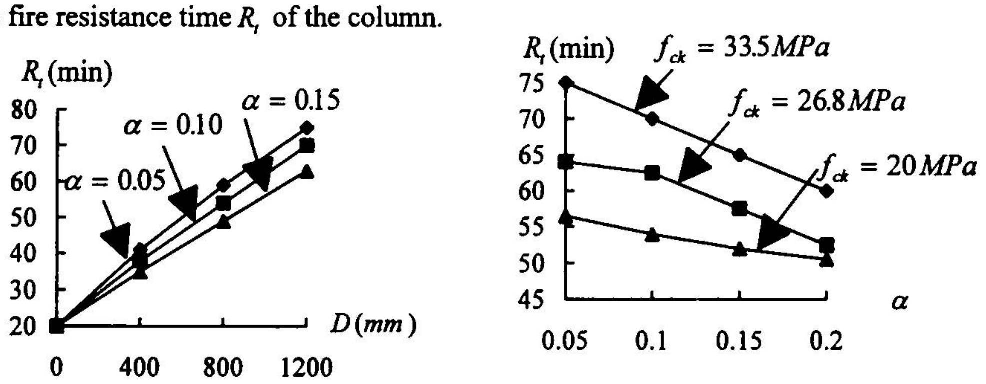
Based on the method introduced above,the strength of column during exposure to fire was calculated,there is good agreement in the trend of deformations between calculated and measured results(more details are found in Reference(9)). A current failure criterion for the column,based on the contraction and the rate of contraction,is that proposed in the ISO-834 Standard,a column is considered to have failed if the column has contracted axially by *0.01L mm* and the rate of contraction has reached *0.003L/min*. A comparison of results calculated using this method with the results of tests is shown in Table 1. There is reasonably good agreement.

TABLE 1. Comparison of Fire Resistance of Tested and Calculated

Column No.	Outer Diameter (*mm*)	Steel Wall Thickness (*mm*)	Steel Strength (*MPa*)	Concrete Strength (*MPa*)	Test Load (*kN*)	Fire Resistance (*min*) Measured[6,7]	Calculated
1	141.3	6.55	350	30	110	55	68
2	168.3	4.78	350	30	218	56	48
3	219.1	4.78	350	30	492	80	69
4	273.0	6.35	350	46.7	1050	188*	194
5	273.0	6.35	350	47	1900	96*	92
6	273.1	5.56	350	30	525	133	112
7	355.6	12.70	350	30	1050	170	168

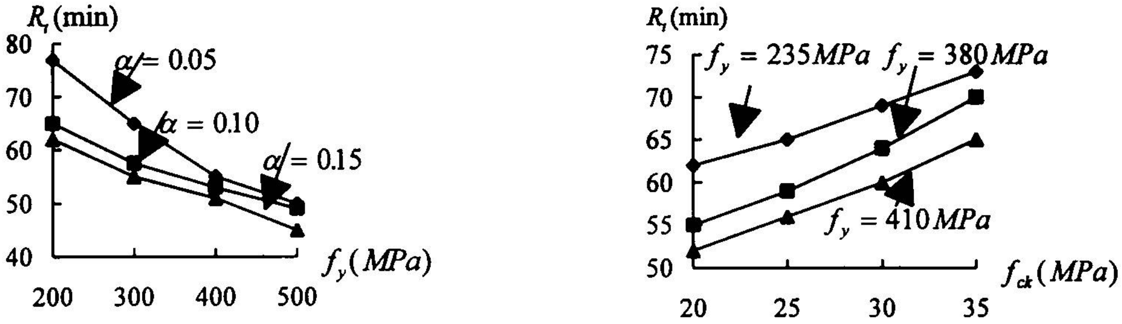
* There are reinforced bars in the core concrete[7].

By the use of the analytical method,influence of the changing steel ratio(α) ,strength of the steel(f_y) and of concrete(f_{ck}), diameter(D) of the column on the fire resistance were analyzed(see Figure 5),in the calculations the applied load of the column have been designed according to 'Designing Code of Concrete Filled Steel Tube of China(DL5400-97)',the initial arbitrary load eccentricity is taken as $0.001L$. Slenderness ratio(λ=4L/D) has almost no influence on the fire resistance time R_t of the column.



($\lambda = 20; f_y = 380MPa; f_{ck} = 26.8MPa$) (a)

($\lambda = 20; f_y = 380MPa; D = 1000mm$) (b)



($\lambda = 20; D = 1000mm; f_{ck} = 26.8MPa$) ($\lambda = 20; \alpha = 0.10; D = 1000mm$)

(c) (d)

Figure 5 Influences of the parameters on the fire resistance time

3. Conclusion

Based on the analytical results of this study,the following conclusion can be drawn:
(1).The analytical method introduced in this paper is capable of predicting the fire resistance of the concrete filled steel tubular columns.
(2).There is composite action between the steel and concrete under fire. The action has been considered in the model for the calculations of the fire resistance of concrete filled steel tubular columns introduced in this paper.
(3). Based on the results of calculation,any desired fire resistance time can be obtained depend on the type and the thickness of the isolating material.

References

1.Design Manual for SHS Concrete Filled Columns,British Steel Corporation,Corby Technical Center,U.K.,1984.
2.Johannes Falke,Comparison of Simple Calculation Methods for the Fire Design of Composite Columns and Beams,Proc. of an Engineering Foundation Confer. on Steel-Concrete Composite Structures,New York,the Structural Division of the ASCE,1993,pp.226-241.
3.Timo Inha,A Simple Method for the Structural Fire Design of Composite Structures,Proc. of an Engineering Foundation Confer. on Steel-Concrete Composite Structures,New York,the Structural Division of the ASCE,1993,pp.210-223.
4.Klingsh,W.,and Wittbecker,F.W.,Fire Resistance of Hollow Section Composite Columns of Small Sections,Bergische University,Wuppertal,Germany,1988.
5.T.T.Lie,and D.C.Stringer,Calculation of the Fire Resistance of Hollow Structural Section Columns Filled with Plain Concrete,Can. J. Civ. Eng.21,1994,pp.382-385.
6.T.T.Lie,and M. Chabot,A Method to Predict the Fire Resistance of Circular Concrete Filled Hollow Steel Columna,J. of Fire Prot. Engr.,2(4),1990,pp.111-126.
7.T.T.Lie,Fire Resistance of Circular Concrete Filled with Bar-Reinforced Concrete,J. of the Struct.Engr.,Vol.120,No.5.M94,pp.1489-1509.
8.Han Lin-hai,Finite Element Analysis of Thermal Field of Concrete Filled Steel Tubes under Fire,J. of Harbin University of Civ. Eng. and Architecture,Vol. 30,N0.2,1997.
9 Han Linhai,Fire Resistance of Concrete Filled Steel Tubular Beam-Columns,Internal Report of Chinese Nation Natural Science Foundation,Beijing,1996.
10.Han Lin-hai and Zhong Shan-tong,Mechnics of Concrete Filled Steel Tubes,Dalian University of Technology Press,1996.