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The Chinese Performance-based Code for Fire-resistance of Steel Structures

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Abstract

In the past two decades, researchers from different countries have conducted series of experimental and theoretical studies to investigate the behaviour of structures in fire. Many new insights, data and calculation methods have been reported, which form the basis for modern interdisciplinary structural fire engineering. Some of those methods are now adopted in quantitative performance-based codes and have been migrated into practice. Mainly based on the achievements in structural fire research at China, the Chinese national code for fire safety of steel structures in buildings has been drafted and approved, and will be released in this year. The code is developed to prevent steel structures subjected to fire from collapsing, ensure safe evacuation of building occupants, and reduce the cost for repairing the damages of the structure caused by fire. This paper presents the main contents of the code, which includes the fire duration requirements of structural components, fundamental requirements on fire safety design of steel components, temperature increasing of atmosphere and structural components in fire, loading effect and capacity of various components in fire, and procedure for fire-resistant check and design of steel components. The analytical approaches employed in the code and their validation works are also presented.

Keywords: Fire-resistance, Steel structures, Code, Safety evaluation, Design

1. Introduction

Fire is a disastrous effect on steel structure since the strength of steel will be greatly reduced with temperature increasing. For the design of steel structures for buildings where fire is likely to happen, fire-resistance has to be considered. The requirements of fire-resistance for steel structures are usually expressed as the fire-resistant duration for various structural components. To satisfy the requirements, the experiments on fire-resistance of steel components with or without fire protection can be conducted. However, there are a number of disadvantages to use experimental approach for fire safety design of steel components. Firstly, it is hard to consider the effects of the various load ratios over the component capacity in reality on the fire-resistance of the component through one or a few experiments. Secondly, it is difficult to simulate the restraint of adjacent structure relative to the considered component in the experiment, which has usually important effects on the fire-resistance of the component. Thirdly, the thermal effect is hard to be considered in the experiment. And finally, the cost of experiments is high. Hence, the advanced approach for fire-resistant design of steel components is based on fire safety check through analysis, which has been employed by BS5950 (1990), EC3 (2002), AS4100 (1990), etc.

The research on behaviour of steel structures subject to fire started from 1990 in China (Li et al., 1999). Since then lots of achievements have been made through theoretical and experimental research. Based on the achievements (Li et al., 1996, 1999; H. S. Zhou, 2004; H. Y. Zhou, 2004), the first code in China on fire safety of steel structures was initiated to compile in 1998 and formally issued in 2000 for steel construction in Shanghai. The analytical approach is adopted in this code, which is validated through experiments and extended to the CECS (China Association for Engineering Construction Standardization) code in 2006 (Chinese standard, 2006). Mainly based on CECS code with adopting the recent achievements in assessing the insulation property of fire protection materials (Li et al., 2012), the national code has been developed (Chinese National code, 2012). The key points of the national code are presented in this paper.

2. Fire Duration Requirements of Structural Components

The aim of the code is to prevent the structure of a building subject to fire from collapsing, ensure safe evacuation of building occupants, and reduce the indirect economical loss due to the failure of the building and the cost for repairing the damages of the structure caused by fire. For this purpose, the requirements on the fire duration of

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Table 1. Fire-resistance duration requirements of structural components (h)

Feature of structural components	Grade of buildings			
	Grade I	Grade II	Grade III	Grade IV
Column and brace	3.00	2.50	2.00	0.50
Girder and truss	2.00	1.50	1.00	0.50
Floor slab	1.50	1.00	0.50	0.25
Roof component	1.50	0.50	-	-
Exit stair component	1.50	1.00	1.00	-

structural components in various buildings are stipulated in this code, as shown in Table 1.

The fire-resistance duration requirement of a structural component is related to the function of the component in the structure and the grade of the building that the structure serves for. The more important is the structural component, the more serious the fire-resistance.

3. Steel Properties at Elevated Temperatures

Because there is no distinct yield strength for steel at elevated temperatures, various effective yield strength of steel at elevated temperatures have been recommended in various codes. Based on the results achieved in Tongji university, the stress under condition of the strain, 1.5%, is adopted as the effective yield strength of steel at elevated temperatures in the Chinese code, which can be expressed with

$$f_{yT}=\eta_T f_y \tag{1}$$

where

$$\eta_T=\begin{cases} 1.0 & 20°\text{C}\le T_s\le 300°\text{C} \\ 1.24\times 10^{-8}T_s^3-2.096\times 10^{-5}T_s & 300°\text{C}<T_s<800°\text{C} \\ \quad +9.228\times 10^{-3}T_s-0.2168 & \\ 0.5-T_s/2000 & 800°\text{C}\le T_s\le 1000°\text{C} \end{cases} \tag{2}$$

here, T_s is the steel temperature; f_{yT} is the effective yield strength of steel at elevated temperature; f_y is the yield strength of steel at ambient temperature; and η_T is the reduction factor of the yield strength of steel at elevated temperature.

The elastic modulus of steel at elevated temperatures can be determined by

$$E_T=\chi_T E \tag{3}$$

$$\chi_T=\begin{cases} \dfrac{7T_s-4780}{6T_s-4760} & 20°\text{C}\le T_s<600°\text{C} \\ \dfrac{1000-T_s}{6T_s-2800} & 600°\text{C}\le T_s\le 1000°\text{C} \end{cases} \tag{4}$$

here, E_T and E are the elastic modulus of steel at elevated and ambient temperature, respectively; and χ_T is the reduction factor of elastic modulus of steel at elevated temperature.

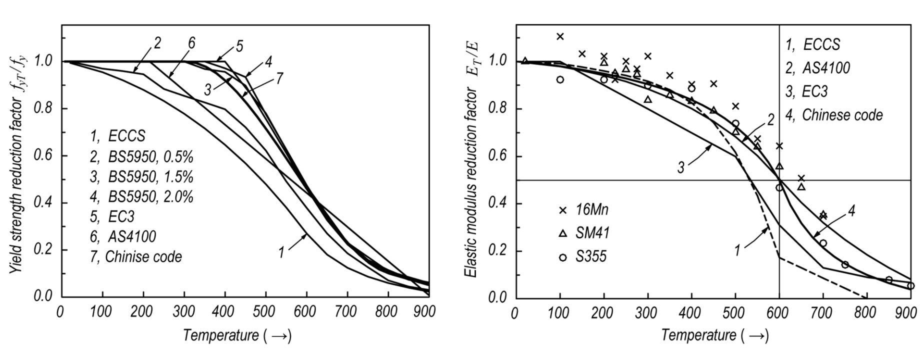
The effective yield strength and elastic modulus at elevated temperatures proposed in various codes are compared in Figure 1(a) and (b), respectively. It can be seen that the yield strength of steel at elevated temperatures used in China code is close to that adopted in BS5950 with strain of 1.5%.

4. Insulation Property of Fire Protection Materials

The insulation property of fire protection materials, which is either intumescent or inorganic, can be assessed by the concept equivalent thermal resistance, calculated by

$$R_i=\frac{5\times 10^{-5}}{\left(\dfrac{T_{crit}-T_0}{t_{crit}}+0.2\right)^2-0.044}\cdot\frac{A_i}{V} \tag{5}$$

where, R_i is the equivalent thermal resistance of the insulation; T_{crit} is the typical critical steel temperature, often taken as 540°C; T_0 is the initial steel temperature, taken as 20°C; t_{crit} is the fire duration time when the steel tempera-



(a) η_T (b) χ_T

Figure 1. Comparisons of the yield strength and elastic modulus reduction factor between various codes.

ture reaches the critical value T_{crit}; and A_i/V is the section factor of the member, in which A_i is interior face area of the fire insulation per unit length and V is volume of the component per unit length.

Intumeacent coatings are reactive fire proofing materials. The behavior of intumescent coatings under heating is very complex and no agreeable model is available to simulate the behavior. The equivalent thermal resistance provide a simple and valid way to assess the fire resistance of intumescent coatings (Li et al., 2012).

5. Fundamental Requirements on the Fire Safety Design of Steel Components

For the fire safety of steel components, it is required that

$$t_d \geq t_m \tag{6}$$

or

$$R_d \geq S_m \tag{7}$$

where t_d and t_m are the actual and required fire-resistant duration of structural components, respectively; R_d is the actual load-bearing capacity of structural components in required fire duration; and S_m is the combined loading effect of structural components in required fire duration.

6. Fire Curves and Steel Temperature Calculations

6.1. Temperature increasing in normal compartment fire

The standard temperature-time curve recommended by ISO834 is adopted for the gas temperature increasing in normal compartment fire, which is expressed as:

$$T_g(t) - T_g(0) = 345\log_{10}(8t+1) \tag{8}$$

where, $T_g(t)$ is gas temperature at time t; $T_g(0)$ is ambient gas temperature; and t is time in minutes.

6.2. Temperature increasing in hydrocarbon fire

The hydrocarbon temperature time curve is given by

$$T_g(t) - T_g(0) = 1080 \times (1 - 0.325e^{-t/6} - 0.675e^{-2.5t}) \tag{9}$$

6.3. Steel temperature calculation

Under the assumption of uniform temperature distribution in a steel component subjected to fire, the temperature increasing in the steel component in a interval Δt can be determined with:

$$T_s(t+\Delta t) - T_s(t) = \frac{B}{\rho_s \cdot c_s} \cdot [T_g(t) - T_s(t)] \cdot \Delta t \tag{10}$$

Where $T_s(t)$ is the steel temperature at time t; ρ_s is the density of steel; c_s is the specific heat of steel; B is the comprehensive heat transfer coefficient, for components without fire insulation,

$$B = (\alpha_c + \alpha_r)\frac{A}{V} \tag{11}$$

for components with fire insulation,

$$B = \frac{1}{1 + \dfrac{\rho_i c_i d_i A_i}{2\rho_s c_s V}} \cdot \frac{\lambda_i}{d_i} \tag{12}$$

where: α_c is the convective heat transfer coefficient between gas and component, taken as 25 W/(m^2·K) for standard fire; α_r is the radiant heat transfer coefficient between gas and component, which can be determined by

$$\alpha_r = \varepsilon_r \sigma \frac{(T_g + 273)^4 - (T_s + 273)^4}{T_g - T_s} \tag{13}$$

λ_i is thermal conductivity of fire protection material; d_i is thickness of the insulation; c_i is specific heat of the fire protection material; A is surface area of the component per unit length exposed to fire; A_i is interior face area of the fire insulation per unit length; and V is volume of the component per unit length.

When the ISO 834 fire atmosphere temperature increase is employed for structural fire-resistant design, a simplified formula for predicting the temperature increasing in the steel component is proposed in the code. It is expressed as

$$T_s = \left(\sqrt{0.044 + 5.0 \times 10^{-5}\frac{\lambda_i}{d_i}\frac{F_i}{V}} - 0.2\right)t + 20 \tag{14}$$

7. Capacity of Various Components in Fire

With temperature elevation caused by fire, the strength and elastic modulus of steel components will be reduced. The formulas recommended by ECCS (1983) governing the reduction of elastic modulus and yield strength of steel are employed in the code. With the reduced elastic modulus and yield strength of steel under high temperature due to fire, the load bearing capacity of various steel components can be formulated with the same approach as for the situation of normal temperature.

7.1. Axial tension component

The capacity of an axial tension steel component under fire condition is governed by

$$\frac{N}{A_n} \leq \gamma_R f_{yT} \tag{15}$$

where N is the combined axial force effect in the component under fire condition; A_n is the net area of cross section of the component; and γ_R is the safety factor for fire condition.

7.2. Axial compression component

The capacity of an axial compression steel component under fire condition is determined by

Table 2. Coefficient α

Slenderness ratio of components	Temperature of structural components (°C)							
	200	300	400	500	550	570	580	600
≤ 50	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96
100	1.04	1.08	1.12	1.12	1.05	1.00	0.97	0.85
150	1.08	1.14	1.21	1.21	1.11	1.00	0.94	0.74
≥ 200	1.10	1.17	1.25	1.25	1.13	1.00	0.93	0.68

$$\frac{N}{\varphi_T A} \le \gamma_R f_{yT} \tag{16}$$

and

$$\varphi_T = \alpha\varphi \tag{17}$$

where φ is factor of stability for the axially compressed steel component at normal temperature; and α is coefficient, as listed in Table 3.

7.3. Bending component

The capacity of a bending steel component is governed by

$$\frac{M_x}{\varphi_{bT} W_x} \le \gamma_R f_{yT} \tag{18}$$

and

$$\varphi_{bT} = \begin{cases} \alpha_b\varphi_b & \alpha_b\varphi_b \le 0.6 \\ 1.07 - \dfrac{0.282}{\alpha_b\varphi_b} \le 1.0 & \alpha_b\varphi_b > 0.6 \end{cases} \tag{19}$$

where M_x is the maximum moment in the component under fire condition; W_x is the modulus of the component cross-section; φ_b is the factor of stability for the bending component at normal temperature; φ_{bT} is the factor of stability for the bending component at elevated temperatures; and α_b is a coefficient determined by

$$\alpha_b = 1.150 + 0.154\sin\left(\frac{T_s}{460}\pi - 0.46\pi\right), \quad 0^{\circ}C < T_s \le 500^{\circ}C \tag{20}$$

$$\alpha_b = 1.292 - 0.6565\times10^{-4}(T_s - 500)^2, \quad 500^{\circ}C < T_s \le 600^{\circ}C \tag{21}$$

7.4. Compression and bending component

The capacity of compression and bending steel components under fire condition can be governed by

$$\frac{N}{\varphi_{xT}A} + \frac{\beta_{mx}M_x}{\gamma_x W_x(1-0.8N/N'_{ExT})} + \eta\frac{\beta_{ty}M_y}{\varphi_{byT}W_y} \le \gamma_R f_{yT} \tag{22}$$

and

$$\frac{N}{\varphi_{yT}A} + \eta\frac{\beta_{tx}M_x}{\varphi_{bxT}W_x} + \frac{\beta_{my}M_y}{\gamma_y W_y(1-0.8N/N'_{EyT})} \le \eta_{sT}\gamma_R f \tag{23}$$

with

$$N'_{ExT} = \pi^2 E_{sT}A/(1.1\lambda_x^2) \tag{24}$$

$$N'_{EyT} = \pi^2 E_{sT}A/(1.1\lambda_y^2) \tag{25}$$

where β_m and β_t are equivalent moment factors; $N'_{ExT} = 1.1(\pi^2 E_T A/\lambda_x^2)$ and $N'_{EyT} = 1.1(\pi^2 E_T A/\lambda_y^2)$ are Euler critical load of the component in the bending plane at elevated temperatures, in which λ_x and λ_y are the slenderness of the component and E_T is the elastic modulus of steel at elevated temperatures; γ_x is factor representing the plasticity development, taken as 1.05 for I and box sectional component; φ_{xT} and φ_{yT} are factors of stability for the axially-compressed steel component in the bending plane and out of the bending plane respectively at elevated temperatures; and φ_{bxT} and φ_{byT} are the factors of stability for the bending component about x and y axis at elevated temperatures

7.5. Steel column in frameworks

In a frame subjected to fire, the columns may yield because of the thermal expansion of the adjacent beams, as shown in Figure 2. The stability of the columns can be checked by the following simple expression,

$$\frac{N}{\varphi_T A} \le 0.7\gamma_R f_{yT} \tag{26}$$

7.6. Steel girder in frameworks

The lateral buckling of steel girders in a frame are usually prevented through bracing of floor slabs. Since the girders in a frame is restrained by adjacent components, the axial force in the girder subjected to fire may vary from compression to tension with development of the deflection of the girder, as shown in Figure 3. The load bearing capacity of steel girder in frameworks subjected to fire is checked at the state when the compressive axial force in the girder is reduced to zero, thus

$$M \le \gamma_R f_{yT} W_p \tag{27}$$

where M is the maximum bending moment in the girder; and W_p is the plastic section modulus of the girder.

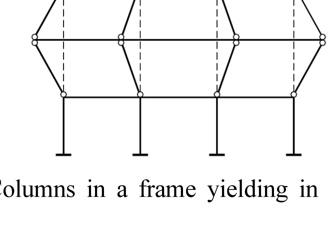


Figure 2. Columns in a frame yielding in fire condition.

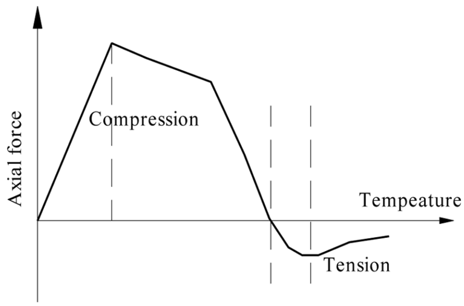


Figure 3. The development of axial force in a restrained girder in fire condition.

7.7. Concrete slab with metal decking

If the deflection of the slab with metal decking is required to be limited to a small value in fire, the fire-resistant duration of the slab can be determined by

$$FR = 114.06 - 26.8\frac{M_{max}}{f_t W} \tag{28}$$

where FR is the fire-resistant duration of the slab with metal decking, in minutes; M_{max} is the maximum bending moment by applied load in unit width of slab; f_t is the tensile design strength of concrete at ambient temperature; W is the section modulus in unit width of the part of concrete where temperature is below 700ºC. The temperature contour of 700ºC at different time in the slab is shown in Figure 4.

If large deflection of slab is allowable, a simplified approach for estimating the fire-resistance of the slab considering membrane action is recommended in the code, according to the research achievement by Bailey et al. (2000) and H. S. Zhou (2004).

7.8. Steel-concrete composite beam

The load bearing capacity of composite beams at elevated temperatures can be checked by

$$M \le M_R^+ \quad \text{for simply supported beam} \tag{29}$$

$$M \le M_R^+ + M_R^- \quad \text{for rotationally restrained beam} \tag{30}$$

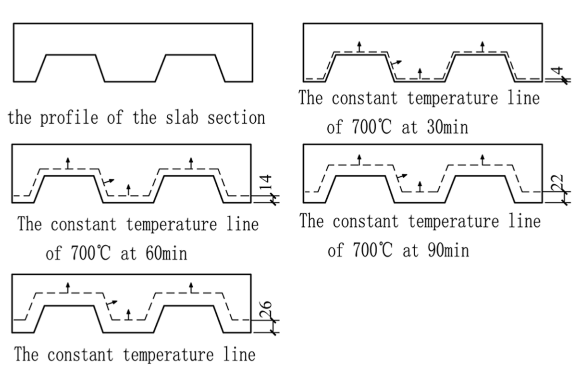


Figure 4. The temperature contour of 700ºC in the slab.

where M is the maximum bending moment in the beam produced by applied load under the condition of simple support, $M = ql^2/8$ for uniformly distributed load; M_R^+ is the sagging bending capacity of the beam at elevated temperatures; M_R^- is the hogging bending capacity of the beam at elevated temperatures.

If the neutral axis is in the concrete, as shown in Figure 5, M_R^+ can be determined by

$$M_R^+ = h_{C1}C_1 - h_{F1}F_1 - h_{F2}F_2 \tag{31}$$

where C_1 is the compressive force in the concrete; F_1 is the tensile force of the top flange of steel beam after it is yield; F_2 is the tensile force of the web and the bottom flange of steel beam after it is yield; h_{C1} is the distance between the position of C_1 and the center of the bottom flange of steel beam; h_{F1} is the distance between the position of F_1 and the center of the bottom flange of steel beam; h_{F2} is the distance between the position of F_2 and the center of the bottom flange of steel beam.

If the neutral axis is in the steel beam, as shown in Figure 6, M_R^+ can be determined by

$$M_R^+ = h_{C1}C_1^{total} + h_{F1}F_1 + h_{F2}^{com}F_2^{com} - h_{F2}^{ten}F_2^{ten} \tag{32}$$

where C_1^{total} is the compressive force when the total area of the concrete is compressed to yielding; $F_2^{com} = 0.5(-C_1 - F_1 - F_2 + F_3)$ is the compressive force of the part of the web in compression; $F_2^{ten} = 0.5(C_1 + F_1 + F_2 - F_3)$ is the tensile force of the part of the web in tension; F_3 is the tensile force of the bottom flange of steel beam after it is yield; h_{F2}^{com} is the distance between the position of F_2^{com} and the center of the bottom flange of steel beam; h_{F2}^{ten} is the distance between the position of F_2^{ten} and the center of the bottom flange of steel beam.

Based on the stress distribution for a composite beam in hogging bending region as shown in Figure 7, M_R^- may be given by

$$M_R^- = h_{y2}^{com}F_{y2}^{com} - h_{y2}^{ten}F_{y2}^{ten} \tag{33}$$

where $F_{y2}^{com} = 0.5(F_1 + F_2)$ is the compressive force of the part of the web in compression; $F_{y2}^{ten} = 0.5(-F_1 + F_2)$ is the tensile force of the part of the web in tension; h_{y2}^{com} is the distance between the position of F_{y2}^{com} and the center of the top flange of steel beam; h_{y2}^{ten} is the distance between the position of F_{y2}^{ten} and the center of the top flange of steel beam; e_3 is the distance between the position of

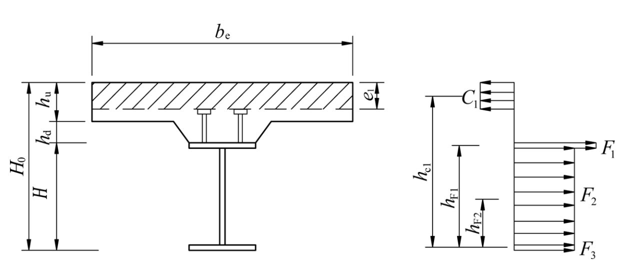


Figure 5. The profile of a composite beam and the stress distribution when the neutral axis is in the concrete.

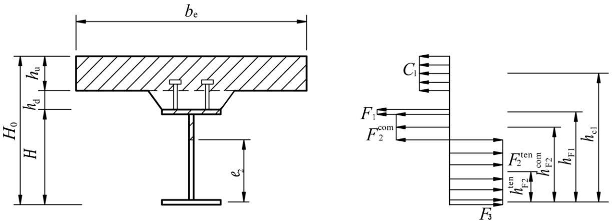


Figure 6. The profile of a composite beam and the stress distribution when the neutral axis is in the steel beam.

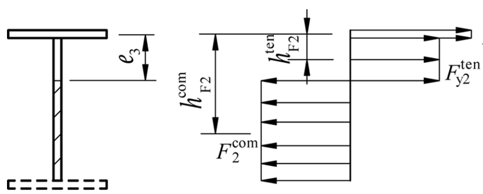


Figure 7. The stress distribution for a composite beam in hogging bending region.

the neutral axis and the center of the top flange of steel beam.

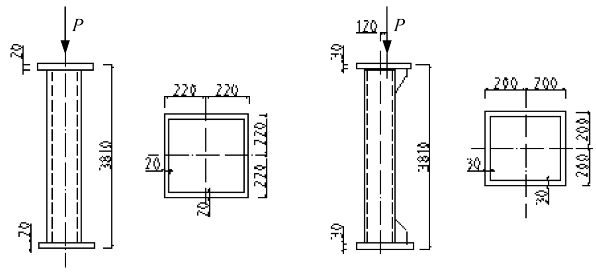
8. Experimental Validation

In order to validate the effectiveness of the approach for the fire safety design of steel components presented hereinabove, a series of experiments have been carried out in Tongji University (Li & Jiang, et al., 2000) on steel columns, steel beams and steel-concrete composite beams subject to fire.

8.1. Experiments on the axially compressed and eccentrically compressed steel columns

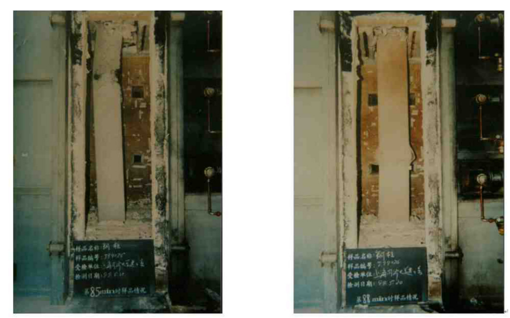
The purpose of the experiments is to validate the fire safety design method employed by the Code for axially and eccentrically compressed steel columns. The axially compressed specimen is a box column welded by steel plate with a thickness of 20 mm, and the eccentrically compressed specimen is also a box column welded by steel plate with a thickness of 30 mm, as shown in Figure 8. The height of the columns is 3810 mm, but only a segment of 3000 mm of the columns were enclosed in the stove. The slenderness ratio of the axially compressed one is 22.2 and that of the eccentric column is 25.2. The offsetting of the load for the eccentrically compressed column is 120 mm. The column specimens were made of Q235 steel, the yielding strength and Young's modulus of which are 255 MPa and 1.85×105 MPa, respectively. The specimens were protected by fireproof paint with a thickness of 10 mm, and the thermal conductivity of 0.12 W/(m·K). Figure 8 Sketches of specimens of columnsshows some pictures of the tested column.

The load on the specimens was kept constant while the temperature in the stove increased according to ISO834 standard. The load for axially compressed column specimen was 4116 kN and that for eccentrically compressed column specimen was 3528 kN. Figure 10 shows the variation of the axial deformation of the specimens with the lasting time of fire exposures. The fire-resistant duration of the axially compressed column and the eccentrically compressed column were measured 88 minutes and 85.2 minutes, respectively.



(a) axially compressed column (b) eccentricall compressed column

Figure 8. Sketches of specimens of columns.



(a) Axially compressed column (b) Eccentrically compressed column

Figure 9. Specimens of columns in the stove.

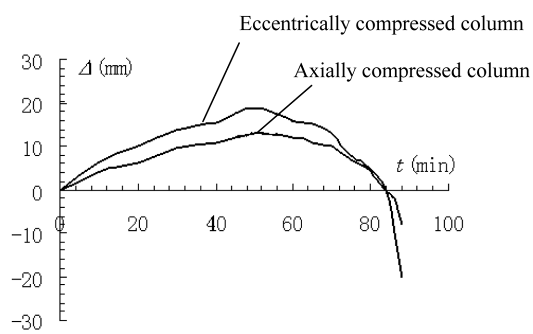


Figure 10. The variation of axial deformation with time.

8.2. Experiments on the axially restrained steel beam

The purpose of the experiments is to validate the fire safety design method employed by the code for bending beams with thermal effects due to restraints. A beam specimen welded by three plates is shown in Figure 11. The steel for the specimen is Q235, the yielding strength of which is 225 MPa and the Young's modulus is 1.85 MPa. The specimen was protected by fireproof paint with a thickness of 15 mm and the thermal conductivity of 0.102 W/(m·K).

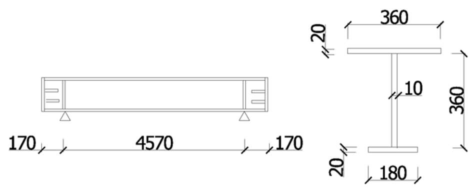
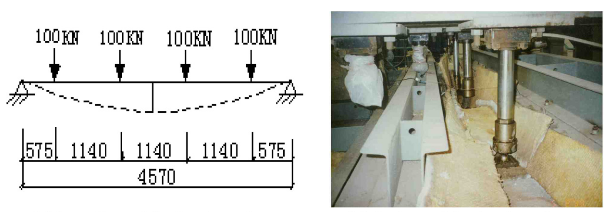


Figure 11. Configuration of the beam.

Figure 12. The configuration of furnace for the experiment on beam.



(a) Location of the loads on beam (b) A view of the loads on beam

Figure 13. Loads on the beam specimen.

Figure 12 shows the furnace for the fire-resistant experiment on the beam. The ends of the beam were restrained by a truss. The truss is heavily insulated by ceramic materials with a thickness of 40 mm. So the axial expansion of the beam is restrained when heated and the thermal effects on the beam was simulated. Four point loads were applied on the beam, as shown in Figure 13.

The ISO834 standard fire was also adopted for the beam specimen. The axial force in the beam due to the restraint of the truss was measured to be 1140 kN when the beam failed. The measured fire-resistance duration of the beam was 54 minutes.

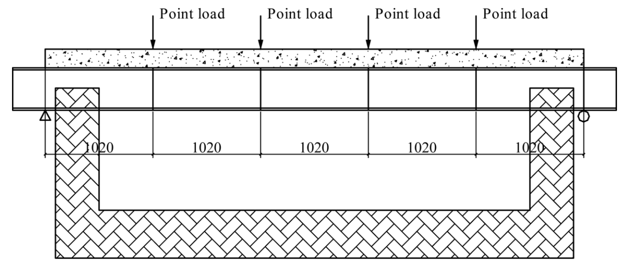


Figure 14. Testing assembly.

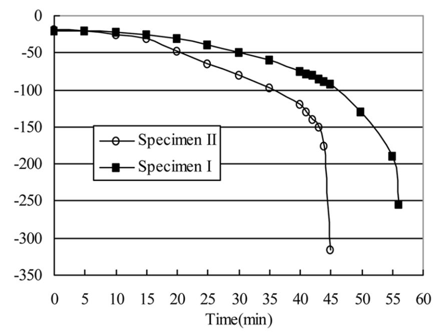
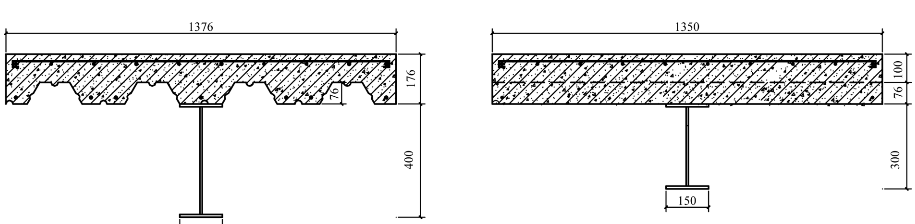


Figure 16. The development of deflections of composite beam specimens.

8.3. Experiments on simply supported composite beams

In order to investigate the behaviour of composite beams subjected to fire, experiments on two specimens of steel-concrete composite beams were conducted. The spans of the both specimens are 5.1 m. The distribution of applied load is shown in Figure 14. The yield strengths of the steel beam is 250 MPa. The compressive strength of the concrete is 33.3 MPa. The beam is protected by fireproof paint with 10 mm in thickness.

The profiles of the two specimens are shown in Figure 15. For specimen I, the cross-sectional size of the steel beam is H400X150X8X12, and the width of the concrete slab is 1376 mm. Each point load applied on specimen I is 100 kN. For specimen II, the cross-sectional size of the steel beam is H300X150X8X8, and the width of the concrete slab is 1350 mm. Each point load applied on specimen II is 75 kN. The developments of deflections of the two specimens are shown in Figure 16. It is found that composite beams are damaged in fire with a transversal crack at mid-span. The measured fire-resistant durations



(a) Specimen I (b) Specimen II

Figure 15. The profile of the specimens.

Table 3. The comparison between measurement and prediction for fire-resistance duration of specimens

Members	Predicted (min)	Measured (min)	Relative error
Axiallycompressed column	94	88	6.82%
Eccentrically compressed column	90	85.2	5.63%
Axiallyrestrained beam	53.8	54	0.4%

Table 4. The comparison between measurement and prediction for the capacity supporting the point load on composite beam specimens

Members	Predicted (kN)	Measured (kN)	Relative error
Specimen I	103	100	3%
Specimen II	78	75	4%

of the two specimens of composite beams were 55 and 45 min, respectively.

8.4. Comparison between prediction and measurement

The fire-resistance duration of the axially compressed column, the eccentrically compressed column and the restrained beams can be predicted through the approach proposed in the Chinese code. The comparison between the results obtained by the experimental measurement and the Code prediction is shown in Table 3. It can be seen that the fire-resistance of steel components can be satisfactorily predicted with the Code.

To validate the approach for evaluating the capacity of steel-concrete composite beams in fire presented in the code, the limit capacity for the point load applied on the two composite beam specimens described hereinabove are predicted with the approach in the code and compared with the experimental results, as shown in Table 4.

9. Concluding Remarks

The main content of Chinese national code on Fire Safety Design of Steel Structures is presented in this paper. The principle of the code is to meet the requirements of fire-resistance on the basis of limit state of steel structures under fire condition. The analytical approach is employed in the code for the fire safety of various steel components through checking the load-bearing capacity of the components exposed to a fire in the time of required fire resistant duration. The effects of the load level, thermal action and structural restraints on the fire-resistance of steel components can be considered. The effectiveness of the approach adopted by the Code is validated through experiments.

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