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A practical approach for fire safety design of fire-resistant steel members

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Abstract. Based on the test data of Nippon Steel Corporation, the formulas for calculating mechanical properties of fire-resistant (FR) steel at elevated temperatures have been established. A practical approach for fire safety design of FR steel members, including axially compressed members, flexural members and eccentrically compressed members, is developed in this paper. Compared with the full-scale specimen experiments and FEM numerical analysis, this practical approach for fire safety design of FR steel members is demonstrated to be effective and precise.

Key words: fire-resistant steel; fire safety; column; beam.

1. Introduction

Steel structures are widely used for buildings due to the advantages of high strength, good ductility and fast fabrication and erection. However, without fire protection steel structures may suffer serious damage or even collapse in a fire catastrophe (Li and Lou 2001). This is because of that the yield strength of steel will be reduced with temperature elevation in steel and at a temperature of around 600 the yield strength of ordinary steel will decline to about one-third of the yield strength at normal temperature. Hence, the load-bearing capacity of steel structures may be greatly reduced during a fire and structures may meet the danger that their capacity may not sustain the external loadings in the fire. Conventional structural steels normally require fire protection measures with paste or panels to hinder the temperature elevation in steel to achieve the desired fire resistance. The application of additional fire protection materials may increase the cost of construction in steel. As an alternative measure of fire-resistance, employing fire-resistant (FR) steel, which requires little or even none of additional fire protection, may lead to cheaper and faster construction (Kelly and Sha 1999). Therefore, many steel manufacturers have developed FR steel for general structures that can better withstand high temperatures compared with ordinary steel, resulting in the increased use of FR steel in structures.

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The conception of FR steel was brought forward by Japanese in the end of 1980s. By adding chromium, molybdenum and other alloying elements, FR steel can maintain high strength level at high temperatures. Initially, Nippon Steel Corporation and Hitachi Metals have developed FR cast steel for general structures which can replace ordinary rolled steel (*Development of Fire Resistant Steel (FR Steel) for Buildings* 1991).

China's Maanshan Steel Co., Baoshan Steel Co. and Wuhan Steel Co. have developed their own FR steel products. At 600°C, these FR steels are able to maintain the strength no less than 2/3 yield strength at room temperature. Their performances at room temperature are as well as conventional steel, and meet all the requirements for steel construction (Wan and Xi 2001, Chen *et al*. 2002).

ThyssenKrupp Stahl in German has developed a new fire-resistant engineering steel named FR30 to market. The steel allows steel structures to be built for the first time without protective coatings and without the usual fire safety measures. Structures made from steel FR30 comply with fire resistance rating F30, which is the minimum requirement for many buildings. FR30 differs from conventional engineering steels in its special alloy composition which enhances toughness and high-temperature strength. With the same load factor the alloy permits significantly higher resistance to fire. All other properties of the new steel are equal to those of conventional engineering steels. FR30 is manufactured and supplied in the form of plate for welding or as finished profiles in thicknesses from 5 to 50 mm and widths up to 3,500 mm (ThyssenKrupp Stahl Trade press 2002).

SAIL (Steel Authority of India Ltd)'s Research & Development Centre for Iron & Steel (RDCIS) has developed FR steel for construction work. The new product is useful in countering fire hazards and enhancing safety of the structures. A new FR steel is also in the final stage of development in the RDCIS laboratory and is expected to be a major breakthrough for the construction industry (Indian Express Newspapers 1998).

FR steels have taken an important step toward providing effective support for architects, civil engineers and builders in the realization of innovative and architecturally ambitious steel building projects. As well as saving costs for the protective cladding elements and coating, the use of FR steel also of course shortens the building process and obviates the need for repeat inspections of fire safety measures and possible reworking.

2. Mechanical properties of FR steel at elevated temperature

The high-temperature mechanical property tests were systematically carried out on SM490-FR steel by Nippon Steel Corporation (NKK 1996, NKK 1995, Sakumoto *et al*. 1992). The test data are used to establish the following mathematical formulations of mechanical properties of FR steel at elevated temperatures for application in fire resistant design of structures using FR steel.

$$\frac{f_{yT}}{f_y} = 1 - (0.001724 \times T_s - 0.034482)^{3.2}/3 \tag{1}$$

$$\frac{E_T}{E} = -2.22 \times 10^{-7} T_s^2 - 2.097 \times 10^{-4} T_s + 1.005 \tag{2}$$

where T_s is the temperature of steel (°C); f_y and f_{yT} are the yield strength of steel at normal temperature and elevated temperature respectively; and E and E_T are the elastic modulus of steel at normal temperature and elevated temperature respectively.

The comparison of the chemical composition between the FR steel and normal steel is listed in Table 1 (Sakumoto *et al*. 1994), and the comparison of the strength between FR steel and normal steel is shown

Table 1 Chemical composition of FR steel and normal steel

Symbol	Chemical composition (WT%)						
	C	Si	Mn	P	S	C_{eq}	P_{CM}
SM490-FR	0.10	0.20	1.11	0.019	0.003	0.42	0.20
	0.16	0.36	1.45	0.019	0.006	0.40	0.25

Note: C_{eq}=C+Mn/6+Si/24+Ni/40+Cr/5+Mo/4+V/14; P_{CM}=C+Si/30+Mn/20+Cu/20+Ni/60+Cr/20/+Mo/15+V/10+5B

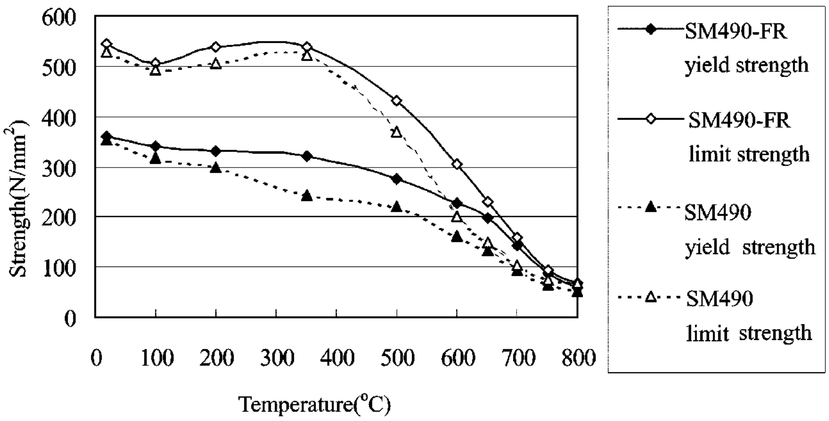

Fig. 1 Comparison of the strength between FR steel and normal steel

in Fig. 1. Different FR steel may have different chemical composition and different properties with temperature elevation. Fortunately, studies reveal that the difference on the properties of FR steels at intermediate temperatures between 20°C and 600°C has negligible effects on the fire-resistant capacity of members of FR steels if the strengths of the steels at 600°C are similar around 2/3 yield strength at room temperature (Ding *et al*. 2004). So, Eq. (1) and Eq. (2) are actually applicable to a range of FR steels with the strength at 600 being about 2/3 yield strength at room temperature for the purpose of studies on the fire-resistant capacity of FR steel members.

3. Elevated temperature during a fire

3.1. Atmosphere temperature

The atmosphere temperature in a fire given by ISO 834 (ECCS 1983) is employed for the purpose of this paper. It is

$$T_g = 20 + 345 \log_{10}(8t + 1) \tag{3}$$

where T_g is the temperature of ambience (°C); t is the fire duration (min).

3.2. Elevated temperature in steel members

Considering the good thermal conductivity of steel, the uniform temperature distribution can be assumed in a FR steel member protected by fire insulation material exposed to fire. With this assumption, the heat transfer equation of the member can be set up as

$$\rho_s \cdot c_s \cdot V \cdot \frac{dT_s}{dt} = \frac{\lambda_i}{d_i} \cdot F_i \cdot (T_g - T_s) \tag{4}$$

where c_s is the specific heat of the insulation material, J/(kg·°C); d_i is the thickness of the insulation material, m; c_s is the specific heat of steel, J/(kg·°C); V is the volume of the member per unit length, m^3/m; ρ_i is the density of the insulation material, kg/m^3; F_i is the area of the inner surface of the insulation material per unit length of the steel member, m^2/m; ρ_s is the density of FR steel, kg/m^3 and λ_i is the thermal conductivity of the insulation material, W/(m·°C).

Eq. (4) can be solved with

$$T_s = \int_0^t \frac{F_i}{\rho_s \cdot c_s \cdot V} \cdot \frac{\lambda_i}{d_i}(T_g - T_s)dt \tag{5}$$

By the results of curve fitting, a simple formula may be obtained from Eq. (5) for predicting the temperature elevation of FR steel members subjected to the standard fire governed by Eq. (3). It is expressed by

$$T_s = (0.102B^{0.6} - 0.4172)t + T_g(0) \tag{6}$$

where $T_g(0)$ is the steel temperature before fire happens, °C and B is a parameter, determined by

$$B = \frac{\lambda_i}{d_i} \cdot \frac{F_i}{V} \tag{7}$$

4. Formulations for fire safety design in practice

4.1. Fire safety check of axially compressed FR steel columns

For the purpose of fire safety design of an axially compressed FR steel column, it is assumed that the initial deformation occurs in the column, which may be expressed as (Fig. 2)(Chinese Code for Structural Design of Steel Construction 1988):

$$y_0 = \delta_0 \sin(\pi x / l) \tag{8}$$

where $\delta_0 = l/1000$ and l is the length of the column.

Being subjected to an axially compressive load, N, the deformation curve of the column may become:

$$y = [\delta_0 / (1 - N/N_{ET})]\sin(\pi x / l) \tag{9}$$

where N_{ET} is the Euler critical load of the column at elevated temperature,

$$N_{ET} = \sigma_{ET} A \tag{10}$$

$$\sigma_{ET} = \pi^2 E_T / \lambda^2 \tag{11}$$

in which A is the gross area of cross-section of the column and λ is the slenderness ratio of the column.

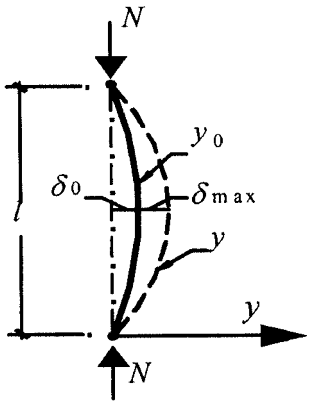

Fig. 2 An axially compressed column with initial deformation

The maximum deformation, occurring at the midpoint of the column, is

$$\delta_{\max} = y|_{x=l/2} = \delta_0/(1 - N/N_{ET}) \tag{12}$$

The maximum stress, locating at the edge of the midpoint section of the column, is

$$\sigma_{\max} = N\delta_{\max}/W + \frac{N}{A} \tag{13}$$

where W is the elastic modulus of the section of the column.

Substituting Eq. (10) and Eq. (12) into Eq. (13) yields

$$\sigma_{\max} = \frac{N}{A}\left[\frac{e_0}{1 - N/(A\sigma_{ET})} + 1\right] \tag{14}$$

where $$e_0 = \delta_0 A/W \tag{15}$$

It is obviously that e_0 is independent of temperature. When the extreme fiber of the midpoint section of the column yields, a plastic hinge will form at the midpoint of the column soon. The column will then lose its stability. Therefore, the critical capacity of the axially compressed column at elevated temperature, N_{crT}, may be determined with Eq. (14) as

$$\sigma_{crT}\left(\frac{e_0}{1 - \sigma_{crT}/\sigma_{ET}} + 1\right) = f_{yT} \tag{16}$$

where $$\sigma_{crT} = \frac{N_{crT}}{A} \tag{17}$$

It can be obtained from Eq. (16) that

$$\sigma_{crT} = \{(1 + e_0)\sigma_{ET} + f_{yT} - \sqrt{[(1 + e_0)\sigma_{ET} + f_{yT}]^2 - 4f_{yT}\sigma_{ET}}\}/2 \tag{18}$$

$$\varphi_T = \frac{\sigma_{crT}}{f_{yT}} = \frac{\{(1 + e_0)\sigma_{ET} + f_{yT} - \sqrt{[(1 + e_0)\sigma_{ET} + f_{yT}]^2 - 4f_{yT}\sigma_{ET}}\}}{2f_{yT}} \tag{19}$$

Similarly, the critical loading capacity of the axially compressed column at normal temperature should be

$$\sigma_{cr} = \{(1+e_0)\sigma_E + f_y - \sqrt{[(1+e_0)\sigma_E + f_y]^2 - 4f_y\sigma_E}\}/2 \tag{20}$$

$$\varphi = \frac{\sigma_{cr}}{f_y} = \frac{\{(1+e_0)\sigma_E + f_y - \sqrt{[(1+e_0)\sigma_E + f_y]^2 - 4f_y\sigma_E}\}}{2f_y} \tag{21}$$

where φ and φ_T are the factor of stability for axially compressed columns at normal and elevated temperature respectively. Normally, φ is given in the Code (Chinese Code for Structural Design of Steel Construction 1998, CEN 1995, BSI 1990).

In the Chinese Code for Structural Design of Steel Construction, the columns are classified into four categories as *a*, *b*, *c* and *d* according to the section pattern and the effects of residual stress. The expressions of e_0 for each sort of columns are as follows:

Sort *a*:
$$e_0 = 0.152\bar{\lambda} - 0.014 \tag{22}$$

Sort *b*:
$$e_0 = 0.300\bar{\lambda} - 0.035 \tag{23}$$

Sort *c*:
$$e_0 = 0.595\bar{\lambda} - 0.094 \qquad \bar{\lambda} \le 1.05 \tag{24a}$$

$$e_0 = 0.302\bar{\lambda} + 0.216 \qquad \bar{\lambda} > 1.05 \tag{24b}$$

Sort *d*:
$$e_0 = 1.081\bar{\lambda} - 0.216 \qquad \bar{\lambda} \le 0.6 \tag{25a}$$

$$e_0 = 0.242\bar{\lambda} + 0.377 \qquad \bar{\lambda} > 0.6 \tag{25b}$$

where
$$\bar{\lambda} = (\lambda/\pi)\sqrt{f_y/E} \tag{26}$$

For obtaining φ_T, we introduce α as

$$\alpha = \varphi_T/\varphi = (\sigma_{crT}\, f_y)/(\sigma_{cr}\, f_y) \tag{27}$$

The values of coefficient α may be obtained through substituting Eq. (1), Eq. (18) and Eq. (20) into Eq. (27) and are listed in Table 2.

The fire safety of axially compressed FR steel columns can then be checked with

$$\frac{N}{\varphi_T \cdot A} \le f_{yT} \tag{28}$$

where
$$\varphi_T = \alpha \cdot \varphi \tag{29}$$

4.2. Fire safety check of FR steel beams

According to elastic theory, the critical moment of a steel beam bending about strong axis may be obtained as Timoshenko and Gere (1961):

Table 2 Coefficient α of axially compressed FR steel columns

	Member temperature °C	100	200	250	300	350	400	450	500	550	600	650	700
Member slenderness ratio λ	30	1.000	1.000	0.999	0.999	0.999	1.000	1.000	1.000	1.001	1.002	1.003	1.005
	40	0.999	0.999	0.998	0.998	0.998	0.999	1.000	1.001	1.003	1.006	1.009	1.013
	50	0.999	0.997	0.996	0.996	0.996	0.997	0.999	1.003	1.007	1.013	1.021	1.030
	60	0.997	0.994	0.992	0.992	0.993	0.995	0.999	1.005	1.014	1.025	1.040	1.058
	70	0.995	0.989	0.987	0.987	0.988	0.991	0.998	1.008	1.023	1.043	1.068	1.099
	80	0.993	0.985	0.982	0.981	0.983	0.988	0.997	1.012	1.034	1.064	1.103	1.152
	90	0.991	0.981	0.977	0.976	0.978	0.984	0.996	1.016	1.044	1.084	1.139	1.211
	100	0.990	0.977	0.973	0.972	0.974	0.982	0.996	1.018	1.053	1.102	1.172	1.268
	110	0.988	0.975	0.970	0.969	0.971	0.979	0.995	1.021	1.060	1.117	1.201	1.319
	120	0.987	0.973	0.968	0.966	0.969	0.978	0.995	1.022	1.065	1.129	1.223	1.364
	130	0.987	0.971	0.966	0.964	0.967	0.977	0.994	1.024	1.069	1.138	1.242	1.400
	140	0.986	0.970	0.965	0.963	0.966	0.976	0.994	1.025	1.072	1.145	1.256	1.431
	150	0.986	0.969	0.964	0.962	0.965	0.975	0.994	1.026	1.075	1.151	1.268	1.456
	200	0.985	0.967	0.961	0.959	0.962	0.973	0.994	1.028	1.082	1.167	1.303	1.532
	250	0.984	0.965	0.959	0.957	0.961	0.972	0.993	1.029	1.086	1.175	1.319	1.569

$$M_{cr} = C_1 \frac{\pi^2 EI_y}{l^2}\left[C_2 a + C_3\beta + \sqrt{(C_2 a + C_3\beta)^2 + \frac{I_W}{I_y}\left(1 + \frac{GI_K l^2}{\pi^2 EI_W}\right)}\right]\beta_b \tag{30}$$

where I_y is the inertia moment about weak axis *y-y* of the beam; I_W is the warping inertia of the beam; I_K is the torsion inertia of the beam; l is the span of the beam; G is the shear modulus; β_b is the equivalent moment coefficient; C_1, C_2, C_3 are coefficients related to load, location and pattern; β is a coefficient related to section pattern of the beam and a is the distance between the acting point of the transverse load and the shear center of the section.

In Eq. (30), the material parameters E and G will change when steel temperature increases, while the other parameters will not change. Hence, Eq. (30) can be modified to calculate the critical moment of steel beam at elevated temperature through simply replacing E and G with E_T and G_T respectively

$$M_{crT} = C_1 \frac{\pi^2 E_T I_y}{l^2}\left[C_2 a + C_3\beta + \sqrt{(C_2 a + C_3\beta)^2 + \frac{I_W}{I_y}\left(1 + \frac{G_T I_K l^2}{\pi^2 E_T I_W}\right)}\right]\beta_b \tag{31}$$

where M_{crT} is the critical moment of steel beam at elevated temperature; G_T is the shear modulus of steel at elevated temperature.

Eq. (30) and Eq. (31) can be simplified as

$$M_{cr} = \varphi_b W f_y \tag{32}$$

$$M_{crT} = \varphi_{bT} W f_{yT} \tag{33}$$

where W is the modulus of the member section; φ_b and φ_{bT} are factor of stability for steel beam at normal and elevated temperature respectively.

Table 3 Coefficient α_b of FR steel beams

T(°C)	50	100	150	200	250	300	350	400
α_b	0.9939	0.9823	0.9712	0.9617	0.9551	0.9529	0.9568	0.9689
T(°C)	450	500	550	600	650	700	750	800
α_b	0.9925	1.0323	1.0962	1.1986	1.3694	1.6816	2.3752	4.9649

Normally φ_b is given in the codes (Chinese Code for Structural Design of Steel Construction 1988, CEN 1995, BSI 1990). For obtaining φ_{bT}, we introduce α_b as

$$\alpha_b = \frac{\varphi_{bT}}{\varphi_b} = \frac{M_{crT} f_y}{M_{cr} f_{yT}} \tag{34}$$

Substituting Eq. (30) and Eq. (31) into Eq. (34) and assuming that the Poisson ratio of steel doesn't change with temperature elevation, then α_b may be obtained as

$$\alpha_b = \frac{E_T}{E} \frac{f_y}{f_{yT}} \tag{35}$$

With Eq. (1), Eq. (2) and Eq. (35), the values of α_b are obtained and listed in Table 3.

The fire safety of FR steel beams can then be checked with

$$\frac{M_x}{\varphi_{bT} W_x} \le f_{yT} \tag{36}$$

$$\varphi_{bT} = \alpha_b \varphi_b \tag{37}$$

where M_x is the maximum moment in the beam under fire condition, kN · m; W_x is the elastic modulus of the member section about strong axis x-x of the beam, m^3.

It should be noted that when $\varphi_{bT} > 0.6$, φ_{bT} must be modified as φ'_{bT} by Chinese Code for Structural Design of Steel Construction (1988).

$$\varphi'_{bT} = 1.07 - 0.282/\varphi_{bT} \qquad \varphi'_{bT} \le 1.0 \tag{38}$$

4.3. Fire safety check of eccentrically compressed FR steel columns

The loading capacity of an eccentrically compressed steel column is usually controlled by its overall stability. Various governing equations are provided for this overall stability in various codes (Chinese Code for Structural Design of Steel Construction 1988, CEN 1995, BSI 1990). In Chinese Code for Structural Design of Steel Construction (1998), the eccentrically compressed steel columns are checked in the bending plane and out of the bending plane separately. Reasonably assume that the governing equations for the overall stability of eccentrically compressed steel columns at elevated temperatures are the same in the form as that at normal temperature, then the fire safety of an eccentrically compressed column may be checked with

in the bending plane:
$$\frac{N}{\varphi_{xT} A} + \frac{\beta_m M_x}{\gamma_x W_x (1 - 0.8N/N_{EXT})} \le f_{yT} \tag{39}$$

out of the bending plane:
$$\frac{N}{\varphi_{xT} A} + \frac{\beta_t M_x}{\varphi_{bT} W_x} \le f_{yT} \tag{40}$$

where β_m and β_t are equivalent uniform moment factors; φ_{xT} and φ_{yT} are factors of stability in the bending plane and out of bending plane respectively for axially compressed FR steel member at elevated temperature, which can be obtained by Eq. (29); φ_{bT} is factor of stability for flexural FR steel member at elevated temperature, which can be obtained by Eq. (37); γ_x is plastic coefficient, γ_x=1.05 for hollow square section and *h*-shape section about strong axis, γ_x=1.2 for *h*-shape section about weak axis; N_{EXT} is Euler critical load of the member in the bending plane at elevated temperature, kN;

$$N_{EXT} = \frac{\pi^2 E_T A}{\lambda_x^2} \tag{41}$$

where λ_x is slenderness ratio of the member about axis *x*-*x*.

5. Application and verification

5.1. An axially compressed FR steel column

Y. Sakumoto (etc) of Nippon Steel Corporation, M. Yoshida (etc) of General Building Research Corporation and H. Saito of Chiba University have carried out a series of full-scale loaded heat tests to verify the fire resistance of columns made from FR steel (Fig. 3) (Sakumoto *et al*. 1994). The test specimen was simply supported with cross-section of H-300×300×10×15 and was made from SM490-FR steel. Its length is 3,500 mm (Fig. 4). The useful parameters of the column are: A=117 cm^2,

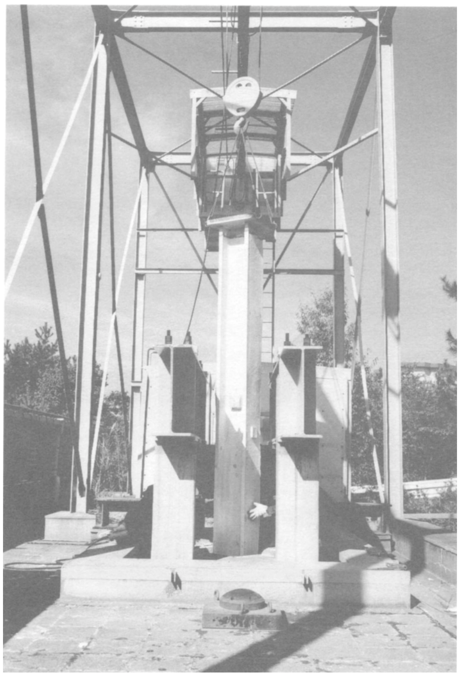

Fig. 3 Testing apparatus and test specimen

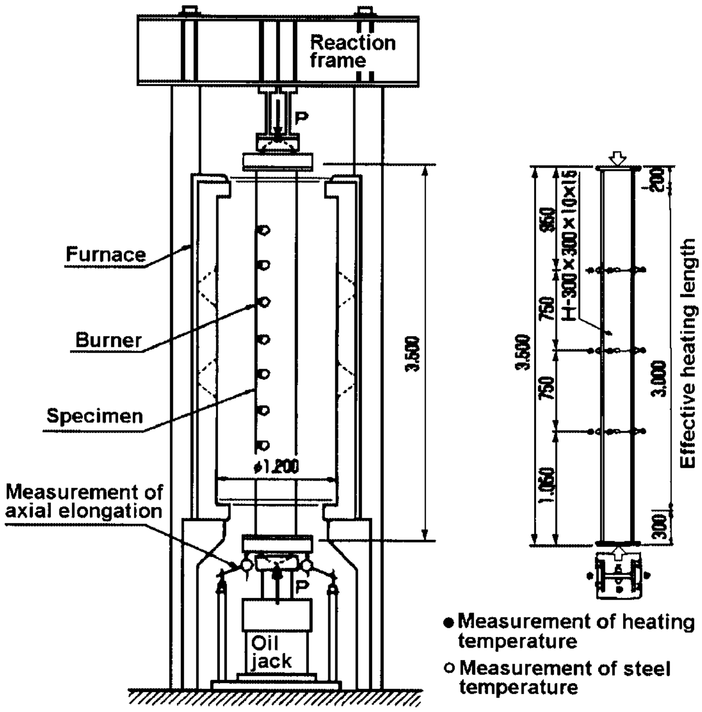

Fig. 4 Testing apparatus, size and shape of the test specimen

I_x=19932.75 cm^4, I_y=6752.25 cm^4, i_x=13.052 cm, i_y=7.597 cm, λ_x=26.815, λ_y=46.072, φ_x=0.9305, φ_y=0.8367. The column was covered by Wet Rock Wool A, and its conductivity is λ_i = 0.13 W/(mK) and the thickness was 12.0 mm. A central compressive load of 2,100 kN was applied to the column. The load capacity of this column is 3483 kN.

To predict the fire duration of the column with the practical approach presented herein before, the critical temperature of the column is firstly calculated. For this purpose, initially assume that α=1.0. Then from Eq. (28) and Eq. (29), it is obtained that

$$\frac{f_{yT}}{f_y} = \frac{N}{\alpha\varphi A f_y} = \frac{N}{\alpha N_u} = \frac{2100}{3483} = 0.6029$$

The steel temperature can then be determined with Eq. (1) as T_s=632.6°C.

Next, it can be determined with Table 1 that α=1.0142. Then

$$\frac{f_{yT}}{f_y} = \frac{N}{\alpha N_u} = \frac{2100}{1.0142 \times 3483} = 0.5945$$

The steel temperature can further be determined with Eq. (1) as T_s=636.7°C.

The last two results of steel temperatures are close to each other, which can be considered to have been convergent. So, the critical temperature of this FR steel column is determined to be 636.7°C.

Secondly, the fire duration time of this FR steel column can be calculated with Eq. (6). With

$$F_i/V = \frac{2h + 4b - 2t}{A} = \frac{2 \times 0.3 + 4 \times 0.3 - 2 \times 0.01}{117 \times 10^{-4}} = 152.14 \text{ m}^{-1}$$

we can obtain that

$$B = \frac{\lambda_i}{d_i} \cdot \frac{F_i}{V} = \frac{0.13}{0.012} \cdot 152.14 = 1648.2$$

and

$$t = \frac{636.8 - 20}{0.102 \times 1648.2^{0.6} - 0.4172} = 74.6 \text{ min}$$

It is obtained from the test that the fire duration time of the column was 77 min, and the average temperature of the column at the moment of failure was 653°C (Sakumoto *et al*. 1994).

Obviously, the results of the fire duration and the critical temperature of the column obtained respectively from the practical approach prediction and measure of test agree with each other very well.

6. A flexural FR steel beam

Y. Sakumoto of Nippon Steel Corporation and I. Nishida of Japan Testing Center for Construction Materials Foundation have carried out a series of full-scale loaded heat tests of FR steel beams (Fig. 5) (Sakumoto and Nishida 1998). The loaded tests were performed in a furnace at the Japan Testing Center for Construction Materials Foundation. The test specimen were simply supported with cross-section of H-400×200×8×13 and were made from SM490-FR steel. The span of one of the specimens is 5,100 mm (Fig. 6). The useful parameters of the beam are: A=81.92 cm^2, I_x=22964.87 cm^4, I_y=1734.93 cm^4, W_x=1148.24 cm^3, i_y=4.602 cm, λ_y=110.82. The stability coefficient of the beam may then be obtained with the Code as φ_b=0.8247.

The beam was covered by Wet Rock Wool A, and its conductivity is λ_i = 0.13W/(mK) and the thickness was 11.3 mm. The load was applied at one-third and two-thirds of the span of the beam. The load

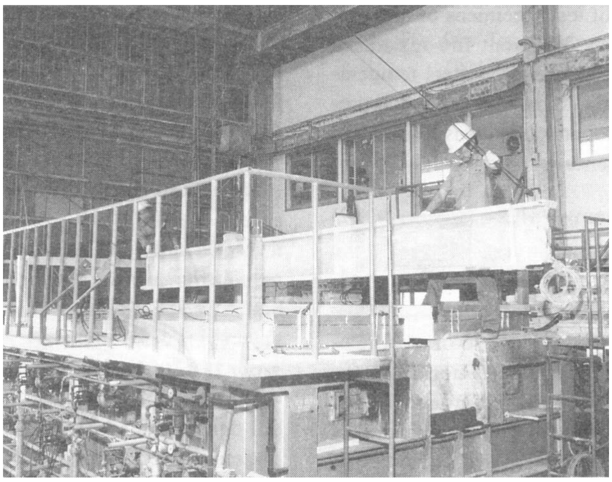

Fig. 5 Testing apparatus and test specimen

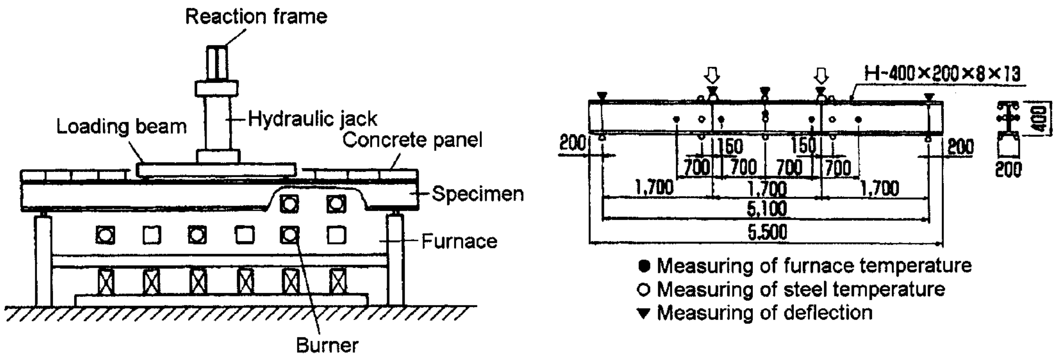

Fig. 6 Testing apparatus, size and shape of the test specimen

at each loading point is 137.5 kN and the load-bearing capacity of the beam at the loading point is 200 kN.

To predict the fire duration of the beam with the practical approach presented hereinbefore, firstly determine the critical temperature of this FR steel beam. For this purpose, initially assume that α_b=1.0. Then, from Eq. (36) and Eq. (37) it is obtained that

$$\frac{f_{yT}}{f_y} = \frac{M}{\alpha_b \varphi_b f_y W} = \frac{M}{M_u} = \frac{137.5}{200} = 0.6875$$

The steel temperature can then be determined with Eq. (1) as T_s=588.45°C.

Next, it can be determined with Table 2 that α_b=1.175. Then

$$\varphi_{bT} = \alpha_b \varphi_b = 1.175\times0.8247=0.969$$

$$\varphi'_{bT} = 1.07-0.282/\varphi_{bT}=0.779$$

$$\frac{f_{yT}}{f_y} = \frac{M}{\alpha_{bT} f_y W} = \frac{0.6875\times0.728}{0.779} = 0.643$$

The steel temperature can further be determined with Eq. (1) as T_s=612.85°C.

It can further be determined with Table 2 that α_b=1.243, then

$$\varphi_{bT} = \alpha_b \varphi_b = 1.243\times0.8247=1.0246$$

$$\varphi'_{bT} = 1.07-0.282/\varphi_{bT} = 0.795$$

$$\frac{f_{yT}}{f_y} = \frac{M}{\alpha_{bT} f_y W} = \frac{0.6875\times0.728}{0.795} = 0.63$$

$$T_s = 619.4^{\circ}\mathrm{C}$$

Continue the above procedure, we have the results of the new iteration as α_b=1.265

$$\varphi_{bT} = \alpha_b \varphi_b = 1.265\times0.8247=1.043$$

$$\varphi'_{bT} = 1.07-0.282/\varphi_{bT}=0.8$$

$$\frac{f_{yT}}{f_y} = \frac{M}{\alpha_{bT} f_y W} = \frac{0.6875 \times 0.728}{0.8} = 0.626$$

$$T_s = 621.3^{\circ}\mathrm{C}$$

The last two results of steel temperatures are close to each other, which can be considered to have been convergent. So, the critical temperature of this FR steel beam is determined to be 621.3°C.

Secondly, the fire duration time of this FR steel beam can be calculated with Eq. (7). With

$$F_i/V = \frac{2h + 3b - 2t}{A} = \frac{2 \times 0.4 + 3 \times 0.2 - 2 \times 0.008}{81.92 \times 10^{-4}} = 168.95 \ \mathrm{m}^{-1}$$

we can obtain that

$$B = \frac{\lambda_i}{d_i} \cdot \frac{F_i}{V} = \frac{0.13}{0.0113} \cdot 168.95 = 1943.7$$

and

$$t = \frac{621.85 - 20}{0.102 \times 1943.7^{0.6} - 0.4172} = 65.6 \ \mathrm{min}$$

It is obtained from the test that the fire duration time of the beam was 64 min, and the average temperature of the beam at the moment of failure was 613°C (Sakumoto and Nishida 1998).

Obviously, the results of the fire duration and the critical temperature of the beam obtained respectively from the practical approach prediction and measure of test agree with each other very well.

6.1. An eccentrically compressed FR steel column

Consider that an eccentrically compressed FR steel column is simply supported with cross-section of H-300×300×10×15 and is made from SM490-FR steel. Its length is 3,500 mm. The useful parameters of the column are: A=117 cm^2, I_x=19932.75 cm^4, I_y=6752.25 cm^4, i_x=13.052 cm, i_y=7.597 cm, W_x=1328.85 cm^3, λ_x=26.815, λ_y=46.072, φ_x=0.9305, φ_y=0.8367, β_{tx}=1, φ_b=1.0. The column was covered by Wet Rock Wool A, and its conductivity is λ_i=0.13W/(mK) and the thickness was 12.0 mm. An eccentrically compressive load of 1,200kN was applied to the column. The eccentricity of the load e=120 mm. The load capacity of this column is 1767.5 kN. If this column is compressed axially, its load capacity is 3483 kN.

To predict the fire duration of the column with the practical approach presented hereinbefore, firstly determine the critical temperature of this FR steel column. For this purpose, initially assume that α 1.0 and α_b=1.0. Then, from Eq. (40) it is obtained that

$$\frac{f_{yT}}{f_y} = \frac{N}{\alpha\varphi A f_y} + \frac{\beta_{tx} M_x}{\alpha_b \varphi_b W_x f_y} = \frac{N}{\alpha N_u} + \frac{M_x}{\alpha_b M_{xu}} = \frac{1200}{1767.5} = 0.679$$

$$\frac{N}{N_u} = \frac{1200}{3843} = 0.312$$

$$\frac{M_x}{M_{xu}} = 0.679 - 0.312 = 0.367$$

The steel temperature can then be determined with Eq. (1) as T=593.3°C.

Next, it can be determined with Table 1 that α=1.01. And it can be determined with Table 2 that α_b>1, so φ_{bT}=1. Then

$$\frac{f_{yT}}{f_y} = \frac{N}{\alpha N_u} + \frac{M_x}{\alpha_b M_{xu}} = \frac{0.312}{1.01} + 0.367 = 0.676$$

The steel temperature can further be determined with Eq. (1) as T=594.9°C.

The last two results of steel temperatures are close to each other, which can be considered to have been convergent. So, the critical temperature of this FR steel column is determined to be 594.9°C.

Secondly, the fire duration time of this FR steel column can be calculated with Eq. (7). With

$$F_i/V = \frac{2h + 4b - 2t}{A} = \frac{2 \times 0.3 + 4 \times 0.3 - 2 \times 0.01}{117 \times 10^{-4}} = 152.14 \text{ m}^{-1}$$

we can obtain that

$$B = \frac{\lambda_i}{d_i} \cdot \frac{F_i}{V} = \frac{0.13}{0.012} \cdot 152.14 = 1648.2$$

and

$$t = \frac{595 - 20}{0.102 \times 1648.2^{0.6} - 0.4172} = 69.54 \text{ min}$$

7. Conclusions

Based on the high-temperature mechanical properties of FR steel and the limit state of load-bearing capacity, a practical approach suitable for fire safety design of FR steel members in practice is developed in this paper. The formulations of the approach for fire safety check of axially compressed columns, eccentrically compressed columns and beams at elevated temperatures are similar to that of the corresponding members at normal temperatures. This is of the benefits for practical engineers to easily understand the concepts behind the approach and to apply the approach in practice. Prediction of the experimental results demonstrated the effectiveness of the proposed approach.

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Notation

T_s : temperature of steel (°C)
f_y : yield strength of steel at normal temperature
f_{yT} : yield strength of steel at elevated temperature
E : elastic modulus of steel at normal temperature
E_T : elastic modulus of steel at elevated temperature
T_g : temperature of ambience (°C)
T : fire duration (min)
c_i : specific heat of the insulation material, J/(kg·°C)
d_i : thickness of the insulation material, m
c_s : specific heat of steel, J/(kg·°C)
V : volume of the member per unit length, m^3/m
ρ_i : density of the insulation material, kg/m^3
F_i : area of the inner surface of the insulation material per unit length of the steel member, m^2/m
ρ_s : density of FR steel, kg/m^3
λ_i : thermal conductivity of the insulation material, W/(m·°C)
$T_g(0)$: steel temperature before fire happens,°C
l : length of the member
N_{ET} : Euler critical load of the column at elevated temperature

A	: gross area of cross-section of the column
λ	: slenderness ratio of the column
W	: modulus of the member section
N_{crT}	: critical capacity of the axially compressed column at elevated temperature
I_y	: inertia moment about weak axis y-y of the beam
I_W	: warping inertia of the beam
I_K	: torsion inertia of the beam
G	: shear modulus
β_b	: equivalent moment coefficient
C_1, C_2, C_3	: coefficient related to load, location and pattern
β	: coefficient related to section pattern of the beam
a	: distance between the acting point of the transverse load and the shear center of the section
M_{crT}	: critical moment of steel beam at elevated temperature
G_T	: shear modulus of steel at elevated temperature
φ_b	: factor of stability for steel beam at normal temperature
φ_{bT}	: factor of stability for steel beam at elevated temperature
M_x	: maximum moment in the beam under fire condition, kN·m
W_x	: modulus of the section about strong axis x-x of the beam, m^3
β_m, β_t	: equivalent uniform moment factors
γ_x	: plastic coefficient, γ_x=1.05 for hollow square section and h-shape section about strong axis, γ_x=1.2 for h-shape section about weak axis

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