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Probabilistic collapse analysis of steel frame structures exposed to fire scenarios*

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Abstract: In this paper, we propose a probabilistic method for analysing the collapse time of steel frame structures in a fire. The method considers the uncertainty of influencing factors. Tornado diagrams are used for sensitivity analysis of random variables. Structural analysis samples are selected by Monte Carlo method, and the collapse times of different structural samples are calculated by fire time history analysis. A collapse time fragility curve is fitted according to the calculated collapse times of the samples. A reliability index of the collapse time is used as a quantitative standard to evaluate the collapse performance of a steel frame in a fire. Finally, this method is applied to analyse the collapse time fragility of an eight-storey 3D steel frame structure under different compartment fire scenarios and fire protection levels. According to the collapse time fragility curve, the effects of the different fire scenarios and protection levels on the collapse resistance of the structure under fire are evaluated.

Key words: Steel frame; Probabilistic collapse analysis; Compartment fire; Monte Carlo method; Fragility curve
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1 Introduction

Steel frame structures are normally used in civil building, but the stiffness and strength of steel degrade significantly at high temperatures (Chen and Jin, 2008). In a fire, the steel structures need to maintain their stability for a specified time to ensure the safe evacuation of occupants. Since the events of 9/11, more attention has been given to the collapse resistance of steel structures in a fire. The collapse time of structures in a fire is also very important for rescue work. Therefore, it is essential to reliably estimate the collapse performance and collapse time of steel frame structures in a fire.

The Cardington full-scale fire test and analysis (Wang, 2000; Quan et al., 2017) showed that the fire resistance of whole structure was obviously different from that of a single component. The interaction between components might influence the behaviour of the whole structure in a fire. Chen et al. (2016) conducted fire tests on two medium-scale steel frames. The influence of a realistic inelastic end constraint on the performance of a steel frame column within the structure was studied. Jiang et al. (2018) experimentally studied the dynamic effect of steel column failure in a planar moment steel frame at high temperature.

Due to the high cost of structure fire tests, finite element analysis is still the main method used for structural fire resistance research. The effects of load ratio, stiffness ratio, and fire location on the load redistribution mechanism of a steel frame were illustrated by Sun et al. (2012) using the computer program Vulcan. The OpenSees software framework redeveloped by Jiang and Usmani (2013) has been used to investigate the progressive collapse resistance of steel frames in a fire.

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Other studies of the collapse of steel structures under fire have been carried out. The behaviour of two steel structures under localized fire was investigated by Agarwal and Varma (2014). The results showed that the gravity column was very important to structural stability in a fire. Jiang and Li (2017b) investigated the load redistribution mechanism of 3D steel frame structures under compartment fire scenarios. They found that the previous sustained load of a buckled column was transmitted more along the short span.

Previous structural collapse analyses were carried out using deterministic parameters. However, uncertainty exists among influencing factors in actual structures. Some studies have been carried out to study the influence of uncertain factors on the fire resistance of structures using probabilistic methods. Structural fragility is the probability that a structure will exceed a specified state under a certain load intensity. The fragility analysis method had been widely used to evaluate the collapse resistance of structures at normal temperatures (Ding et al., 2017; Kumar and Matsagar, 2018). A performance-based earthquake engineering framework was applied by Lange et al. (2014) to study the influence of different fire models on the fire resistance of steel components. A reliability-based design methodology, presented by Guo and Jeffers (2015), was applied to the design of a protected steel column subjected to a parametric fire. Gernay et al. (2019) proposed a fire fragility function to assess structural fire performance. Shrivastava et al. (2019) provided a comprehensive review dedicated to the application of probabilistic methods in structural fire engineering, and emphasized the key factors of compartment fires.

At present, fragility analysis is used mainly to analyse the fire resistance of components. There have been a few studies on the collapse time of whole steel frames under fire. In performance-based fire resistance design, structural collapse is an important criterion. Collapse time is also very important for the safety of rescue workers. Thus, it is important to illustrate the probability of structural collapse time considering the uncertainty of influencing factors. In addition, the collapse resistance of structures under fire should also be quantitatively evaluated using a reliability index.

In this study, we propose a probabilistic method for analysing the collapse time of a steel frame structure. The method considers the uncertainty of material properties and fire protection coatings. Note that in considering the effect of fire protection, to ensure the duration of fire, the idealized standard fire curve was adopted for analysis. The effects of fire uncertainty were not considered in the study. The proposed method is used to analyse the collapse performance of an eight-storey 3D steel frame structure under compartment fire. Firstly, through a sensitivity analysis of random variables, a tornado diagram of the effect of random variables on structure collapse time is drawn, and the degree of influence of the different random variables on structural collapse time is obtained. Secondly, structural analysis samples are selected by the Monte Carlo method, and fire time history analysis is carried out on different structural samples by finite element method. The collapse time of each structural sample is obtained. Finally, structural collapse is taken as the limit state, and the duration of fire is taken as an intensity measure. A collapse time fragility curve is fitted according to the calculated collapse time. The reliability index of the collapse time is used as a quantitative standard to evaluate the collapse performance of the steel frame under fire. At the same time, different compartment fire scenarios and fire protection levels were selected for investigation. According to the collapse fragility curve of the structure under different fire conditions, the influence of different compartment fire scenarios and fire protection levels on the structural collapse probability under fire is evaluated.

2 Collapse probability analysis methodology

A flow chart of the collapse probability analysis method is shown in Fig. 1. The following sections will describe each part of the process in detail.

2.1 Structural analysis method and collapse criteria

In this study, the explicit dynamic analysis software LS-DYNA was used to analyse the fire time history of a steel frame structure. The beams and columns of the steel structure are modelled by the Hughes-Liu beam element. The constitutive relation of steel at high temperature is derived from Eurocode3 (EC3) (CEN, 2005). The constitutive relation of concrete at high temperature is derived from Eurocode2 (EC2) (CEN, 2004). On the cross section of the beam element, 21 integral points are defined to satisfy the calculation accuracy. A layered composite shell element is used to model the floor slabs of the structure. For the layered composite shell element, the integration point is located in the middle of each layer. The trapezoid rule is applied to the integration of the shell element. In the analysis, the fracture of the shell element is determined by the effective plastic strain.

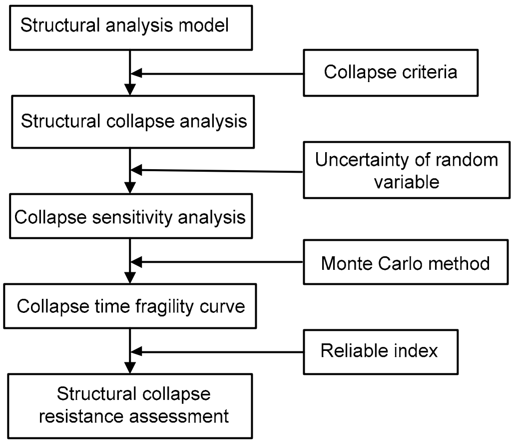

Fig. 1 A flow chart of the collapse probability analysis method

In the fire time history analysis, the loads are applied to the structure in three load steps. Firstly, a vertical load is applied to the floor slabs. Secondly, the vertical load is kept constant to eliminate dynamic effects of the load. Thirdly, the corresponding temperature is applied to the heated components while the vertical load is kept constant. This analysis method has previously been validated and applied (Ren, 2020). Therefore, this fire time history analysis method was used in this study.

The study was aimed mainly at the collapse of a steel frame structure in a fire. The regulations on the collapse of steel structures in GSA2003 (GSA, 2003) were adopted. For a steel frame structure, a rotation of a beam of up to 12° is used as the collapse criterion. Considering the catenary mechanism of a steel beam at high temperature, it is reasonable to use 12° as the collapse criterion for a steel fame under fire. Therefore, the time when the rotation of the frame beam reached 12° was taken as the structural collapse time in this study.

2.2 Structural collapse time sensitivity analysis

Due to the randomness of different design parameters, tornado diagram analysis (TDA) can be used to assess the sensitivity of the structural collapse time to random variables. To draw the tornado diagram, the nominal collapse time should first be obtained by a deterministic analysis. In this deterministic analysis, the random variables are set to their nominal values, and the nominal collapse time calculated by fire time history analysis. Then, the upper and lower limits of the random variables are taken, and the effects of each random variable on the upper and lower limits of the collapse time are calculated by fire history analysis. When the collapse time of the structure is calculated, only one random variable changes and the remaining variables remain as nominal values. In this study, the upper and lower limits of random variables were taken as (nominal value±mean square deviation). The absolute variation of collapse time caused by the change of random variable can be used to measure collapse time sensitivity. According to the absolute change of collapse time, the random variables are arranged in descending order according to the relative magnitude of their effect. The variable at the top of tornado diagram has the greatest influence on the structural collapse time.

2.3 Structural collapse time fragility curve

The probability of collapse of a structure under a certain load strength is called the structural collapse fragility. A collapse fragility curve is used to define the structural collapse probability at different intensity measure (IM) values. In this study, the fire duration t was used as the IM, and the collapse of the structure under fire was chosen as the limit state. Since the relation between random variables and collapse time cannot be written directly, the fragility curve should be fitted by Monte Carlo method. In this study, the structural analysis samples were selected by the Latin hypercube method. The collapse time of each structural sample under fire is calculated by fire time history analysis, and the logarithmic mean value $\mu_{\ln}$ and logarithmic standard deviation $\sigma_{\ln}$ of collapse time are calculated as follows:

$$\mu_{\ln} = \frac{\sum_{i=1}^{N} \ln t_{\mathrm{cp},i}}{N}, \tag{1}$$

$$\sigma_{\ln} = \sqrt{\frac{\sum_{i=1}^{N} (\ln t_{\mathrm{cp},i} - \mu_{\ln})^2}{N-1}}, \tag{2}$$

where N is the number of structural samples, and $t_{\mathrm{cp},i}$ is the collapse time of the ith sample.

According to the logarithmic mean and logarithmic standard deviation, the collapse fragility curve of the structure under fire can be fitted by the following equation (Porter et al., 2007):

$$P_{\mathrm{cp}}(\mathrm{IM}=t) = \Phi\left(\frac{\ln t - \mu_{\ln}}{\sigma_{\ln}}\right), \tag{3}$$

where $P_{\mathrm{cp}}(\mathrm{IM}=t)$ is the collapse probability of the structure under fire duration t; $\Phi(\cdot)$ is the standard normal cumulative distribution function.

2.4 Reliability index of structural collapse time in a fire

From the perspective of collapse time, the limit state equation of structural collapse time under fire can be written as

$$Z = t_{\mathrm{cp}} - t_{\mathrm{dm}}, \tag{4}$$

where t_{cp} is the structural collapse time and t_{dm} is the design demand time.

According to the limit state equation, the reliability index β of the structural collapse time in a fire can be calculated as

$$\beta = \frac{\mu_{\mathrm{cp}} - \mu_{\mathrm{dm}}}{\sqrt{\sigma_{\mathrm{cp}}^2 + \sigma_{\mathrm{dm}}^2}}, \tag{5}$$

where μ_{cp} and σ_{cp} are the mean value and mean square deviation of the structural collapse time t_{cp}, respectively; μ_{dm} and σ_{dm} are the mean value and mean square deviation of the design demand time t_{dm}, respectively.

3 Case analysis

3.1 Model of steel frame

The structural collapse probability analysis methodology was applied to an eight-storey steel frame. The steel frame was selected from the test structure in the Cardington fire test (BS, 1999). The structural plan arrangement is shown in Fig. 2. The steel building represents a typical office building. There are five bays along the long span direction, and

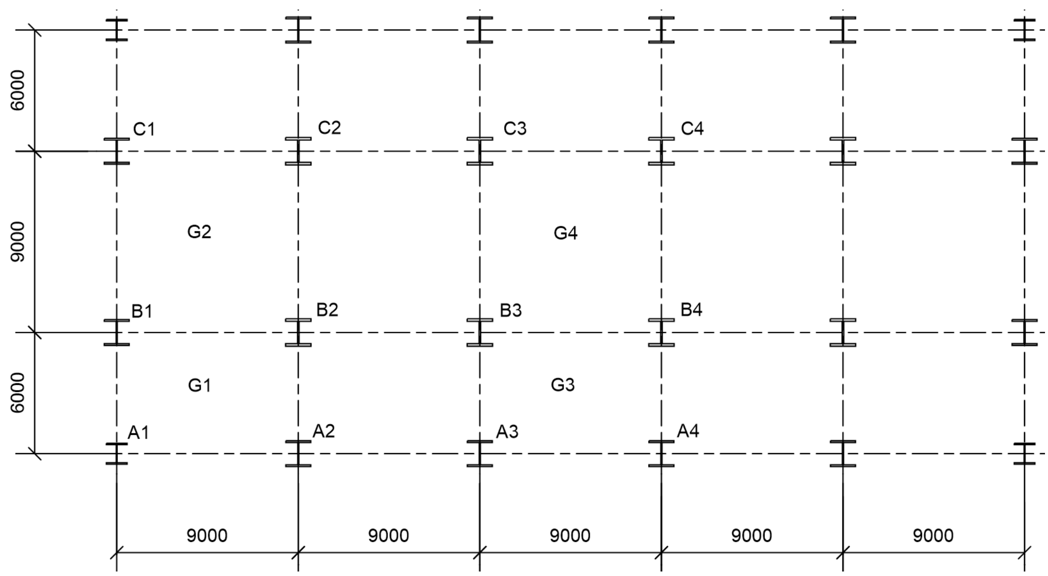

Fig. 2 Structural plan arrangement of the steel frame (unit: mm)

the length of each span is 9 m. There are three bays in the short span direction, and the length of each span is 6 m, 9 m, and 6 m, respectively. The height of each floor is 4 m. The corner columns are taken as UC254×254×89. The H-shaped sections UC305×305×137 and UC305×305×198 are used for the edge columns and middle columns, respectively. The beam is made of H-shaped steel, and the section is UB356×171×45. The connection between the beam and column is assumed to be rigid. Steel grade S275 is used for the columns and beams of the structure. The yield strength is 275 MPa, while the elasticity modulus is 210 GPa. The floor consists of a 100-mm-thick concrete slab. For the slab, the diameter of the reinforcement bars is 10 mm, and the space between bars is 150 mm. The thickness of concrete covering the steel bars is 25 mm. The compressive strength is 20.1 MPa for concrete, and the yield strength is 300 MPa for the reinforcements. In structural analysis, the dead load is 4.94 kPa, which includes mainly the self-weight of the building materials. Since this building is an office building, the live load could be considered as 2.5 kPa. According to the regulations of Eurocode1 (EC1) (CEN, 2002), the load combination (dead load+0.5 live load) was adopted in the fire time history analysis. In this study, a uniformly distributed load of 6.19 kPa was applied to the slabs. The finite element model of the prototype frame is shown in Fig. 3.

3.2 Fire protection and compartment fire scenarios

Fire protection is usually required for the steel of a building. Three levels of fire protection (high, medium, and low) were considered in this study, based on the research of Jiang and Li (2017a). For high fire protection, the fire rating of the column is 3 h and the fire rating of the beam is 2 h; for medium fire protection, the fire rating of the column is 2 h and the fire rating of the beam is 1.5 h; for low fire protection, the fire rating of the column is 1 h and the fire rating of the beam is 1 h. The insulation material is CAFCO 300 with a specified thermal conductivity of 0.078 W/mK and density of 240 kg/m^3 at room temperature (Dwaikat et al., 2011). According to current fire design specifications, the thicknesses of fire protection coatings of the components were designed using the ISO 834-1 standard fire curve (ISO, 1999). Based on heat conduction analysis, the thicknesses of fire protection coating of beams and columns under different fire protection levels are listed in Table 1.

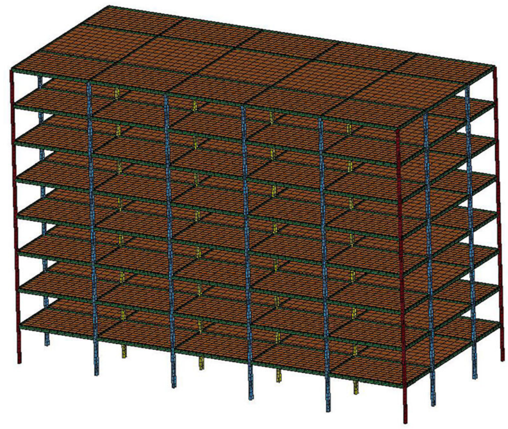

Fig. 3 Finite element model of the prototype frame

Table 1 Thicknesses of fire protection coating of beams and columns

Item	Fire protection level	Nominal value of thickness (cm)
Column coating	Low	1.3
	Medium	2.2
	High	3.2
Beam coating	Low	2.3
	Medium	3.7
	High	4.9

A compartment fire scenario was mainly considered in this study. The ISO 834-1 standard temperature curve (ISO, 1999) was applied to compartment fire scenarios. The main reason for choosing the ISO 834-1 standard fire curve was the fire duration. For high fire protection structures, the fire duration should be more than 3 h. According to our previous study (Ren, 2020), the fire duration of the parametric fire curve is generally no more than 2 h. Therefore, the ISO 834-1 standard fire curve was selected in this study. As the load ratio of the component on the ground floor is the largest, the positions of the fire compartment were the corner compartment (G1), short span compartment (G2), long span compartment (G3), and middle compartment (G4) on the ground floor.

3.3 Random variables

The effects of the uncertainty of random variables on the collapse performance of structures in a fire were the main focus of this study. The random variables studied in this section include the steel yield strength (fy), steel elastic modulus (Es), concrete compressive strength (fc), yield strength of reinforcement (fyr), column fire protection (frc), and beam fire protection (frb). The reasons for selecting these random variables were as follows. As the degradation of yield strength and stiffness of steel at high temperature is the main factor influencing the fire resistance of steel structure (Kodur et al., 2010), the yield strength and elastic modulus of steel were selected as random variables. According to a relevant study (Huang et al., 2003), the membrane action of the concrete slab also has an important influence on the collapse resistance of the structure. The membrane action of the slab is related to the concrete compressive strength and the yield strength of reinforcement, so these two random variables should be selected. In addition, fire protection affects the temperature distribution of the structure under fire. Therefore, fire protection was also selected as a random variable in this study. The distribution law and coefficient of variation (COV) of each random variable can be found in previous studies (Wiśniewski et al., 2012; Ding et al., 2017). The basic statistical quantities of random variables used to establish the fragility curve are listed in Table 2.

4 Results

4.1 Collapse sensitivity analysis

This section investigates the effect of randomness of material properties and fire protection on the collapse time of steel frame in a fire. Fig. 4 presents the effect of random variables on the collapse time of structures in corner fire scenarios under different fire protection conditions. In the tornado diagram, μ_v is the mean value of the random variable, and σ_v is the mean square deviation of the random variable. As shown in Fig. 4, the steel yield strength, column fire protection coating thickness, and beam fire protection coating thickness have a great influence on the structural collapse time. In addition, at the high fire protection level, the thickness of the column fire protection coating has a greater influence on the structural collapse time. However, in the medium and low fire protection levels, the yield strength of steel has a greater influence on the structural collapse time. The reason for this difference may be that the random range of fire coating thickness of the column is larger at the high fire protection level.

The influence of random variables on structural collapse also differs depending on the location of compartment fire. Fig. 5 presents the effect of random variables on the collapse time of structures in different compartment fire scenarios at the high fire protection level. The steel yield strength, column fire protection coating thickness, and beam fire protection coating thickness still have a great influence on the collapse time of the structure. The effect of steel yield strength is greater than that of the thickness of fire protection coating in the middle compartment fire scenario. In the other three compartment fire scenarios, the influence of the thickness of the fire protection coating on the column is greater. This demonstrates that the steel structure could bear the redistribution of internal force after the failure of the steel column in the middle

Table 2 Basic statistical quantities of random variables

Random variable	Nominal value	COV, σ/μ (%)	Distribution
Yield strength of steel (MPa)	275	11	Lognormal
Elastic modulus of steel (MPa)	2.10×10^5	3	Lognormal
Concrete compressive strength (MPa)	20.1	10	Lognormal
Yield strength of reinforcement (MPa)	300	5	Lognormal
Thickness of column fire resistant coating (low) (cm)	1.3	5	Lognormal
Thickness of column fire resistant coating (medium) (cm)	2.2	5	Lognormal
Thickness of column fire resistant coating (high) (cm)	3.2	5	Lognormal
Thickness of beam fire resistant coating (low) (cm)	2.3	5	Lognormal
Thickness of beam fire resistant coating (medium) (cm)	3.7	5	Lognormal
Thickness of beam fire resistant coating (high) (cm)	4.9	5	Lognormal

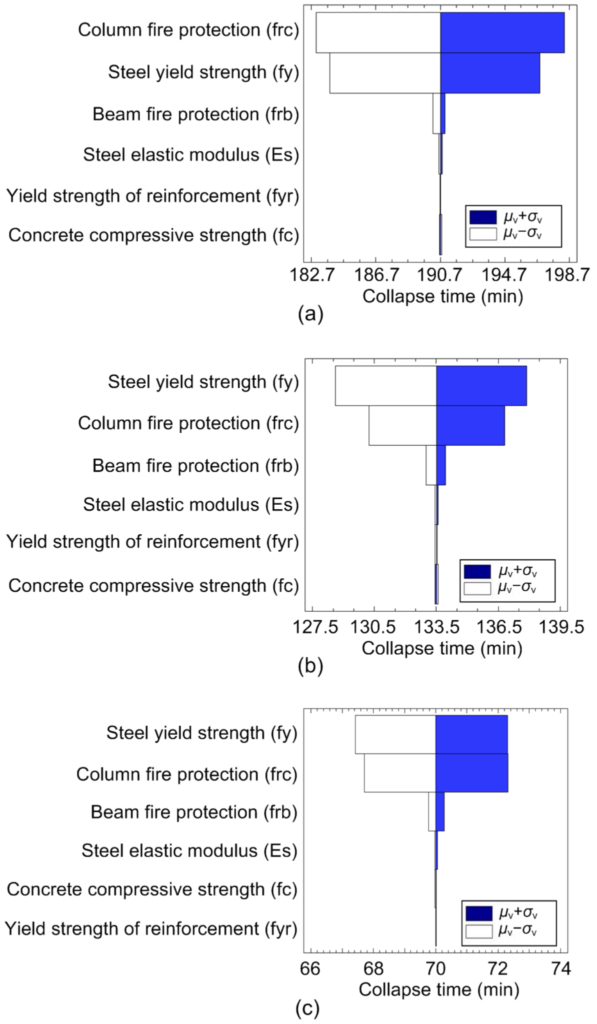

Fig. 4 Tornado diagrams of a corner compartment fire at different fire protection levels
(a) High fire protection level; (b) Medium fire protection level; (c) Low fire protection level

compartment due to the existence of more redundant constraints. The structure does not collapse until the steel cannot withstand the load. Therefore, the influence of steel yield strength is greater than that of the thickness of the fire protection coating. In the other three compartment fire scenarios, the redundancy constraint after the failure of the fire column is lower. Thus, the failure of the fire column has a greater impact on the collapse of the structure. As a result, the column fire protection has a greater impact.

4.2 Collapse time fragility curve

To satisfy the accuracy of the Monte Carlo method, the number of samples was set to 500 for

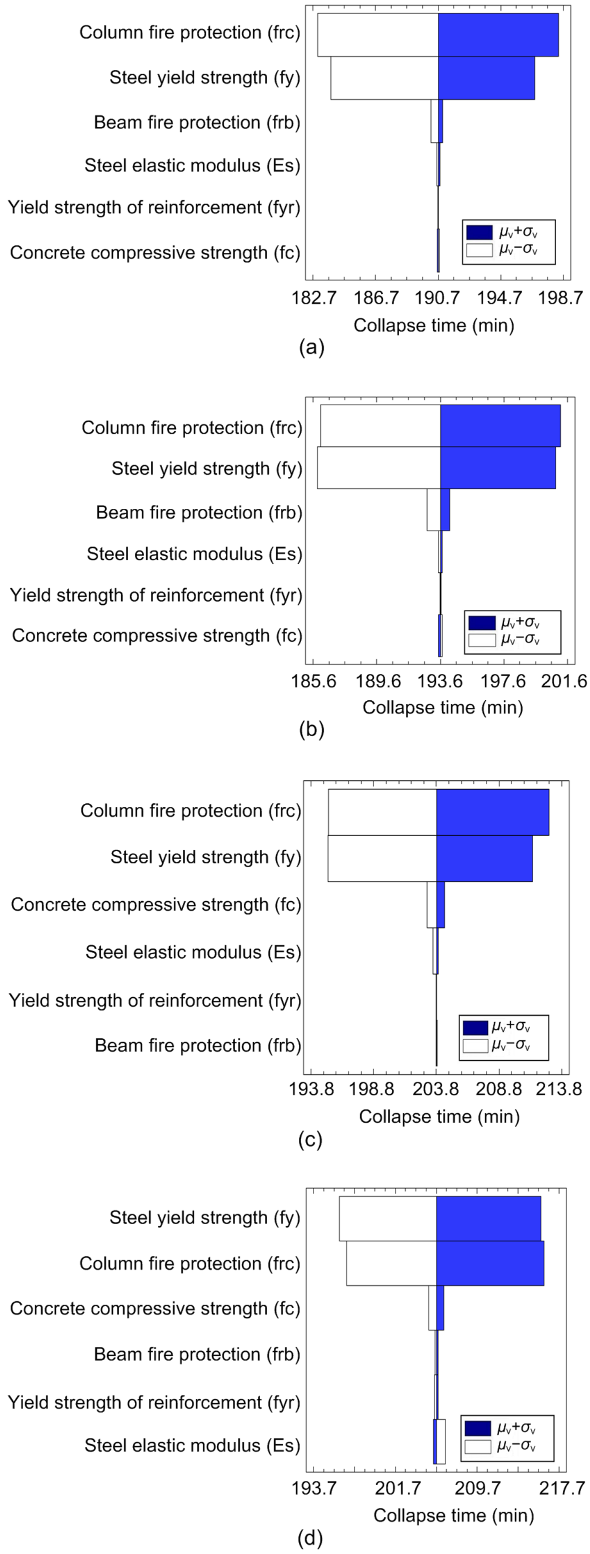

Fig. 5 Tornado diagrams of different compartment fires at the high fire protection level
(a) Corner compartment fire; (b) Short span compartment fire; (c) Long span compartment fire; (d) Middle compartment fire

each condition calculated in this study. In a corner compartment fire, the structural collapse time fragility curves at different fire protection levels are shown in Fig. 6. We assumed that the design demand time t_{dm} for the structure at the three fire protection levels was 1 h, 2 h, and 3 h, respectively. The statistical parameters of the collapse time and reliability indexes of the steel frame at the three fire protection levels are listed in Table 3.

According to the data in Table 3, the probability that the collapse time is greater than the fire demand time of 3 h is less than 95% at the high fire protection level. At other fire protection levels, the probability could reach 95%. The reliability index of structural collapse is also the lowest at the high fire protection level. The reason may be that the coating thickness changes more at the high fire protection level when the randomness of the coating thickness is taken into account. As shown in Fig. 4a, the fire protection coating thickness has the greatest influence on the collapse time of the structure at the high fire protection level. As a result, the variation in the structural collapse time is more obvious. It also means that the uncertainty of random variables has a greater impact on the collapse time of the structure at the high fire protection level. Therefore, stricter measures should be taken for structures with a high fire protection level in the design of structural fire resistance.

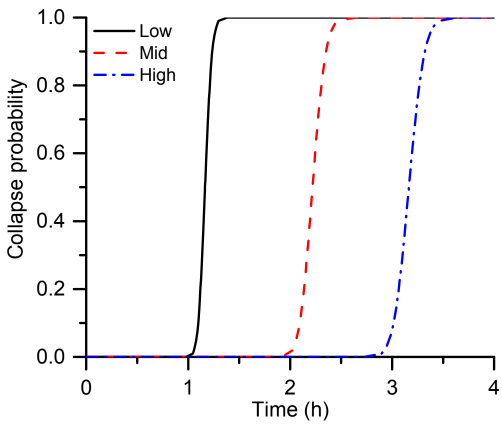

Fig. 6 Collapse time fragility curves at different fire protection levels in a corner compartment fire scenario

At the high fire protection level, the structural collapse time fragility curves in different compartment fire scenarios are shown in Fig. 7. The statistical parameters of the collapse time and reliability indexes of the structure in different compartment fire scenarios are listed in Table 4. The mean collapse time is larger than the demand time of 3 h in each compartment fire scenario. However, considering a 95% guarantee rate, the collapse time is less than 3 h in both the corner and short span compartment fire scenarios. The greater the COV of the collapse time, the greater the influence of randomness on the collapse time. In the middle compartment fire scenario, the COV of structural collapse time is the largest. This indicates that the structural collapse time is more significantly influenced by random factors in the middle compartment fire scenario. More attention should be paid to this problem in the design of structural collapse resistance under fire. Finally, according to the reliability indexes listed in Table 4, reliability is the lowest in the short span compartment fire scenario, and the highest in the long span compartment fire scenario. We conclude that the short span compartment fire is the most dangerous among the four scenarios, while the long span compartment fire is the safest.

5 Discussion

In previous sections, the Monte Carlo method was used to establish the collapse time fragility curves of steel frame structures, and a method to evaluate the

Table 3 Statistical parameters of the collapse time and reliability indexes of the structure in a corner compartment fire scenario

Fire protection level	Collapse time, t_{cp} Mean value (h)	Collapse time, t_{cp} COV (%)	95% reliability collapse time, $t_{cp,95}$ (h)	Mean square deviation (h)	Reliability index, β
High	3.17	3.80	2.97	0.12	1.41
Medium	2.22	4.50	2.06	0.10	2.20
Low	1.17	4.60	1.08	0.05	3.16

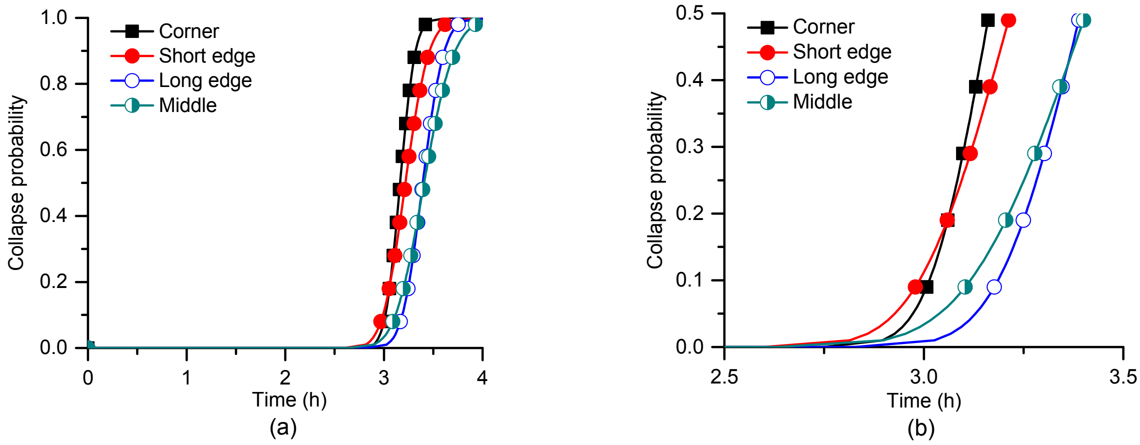

Fig. 7 Collapse time fragility curves at the high fire protection level in different compartment fire scenarios
(a) Collapse time fragility curves; (b) Detail of collapse time fragility curves (from 2.5 to 3.5 h)

Table 4 Statistical parameters of the collapse time and reliability indexes of the structure at high fire protection level

Compartment fire scenario	Collapse time, t_{cp} Mean value (h)	Collapse time, t_{cp} COV (%)	95% reliability collapse time, $t_{cp,95}$ (h)	Mean square deviation (h)	Reliability index, β
Corner	3.17	3.8	2.97	0.12	1.41
Short span	3.22	5.7	2.93	0.19	1.16
Long span	3.39	4.9	3.13	0.17	2.29
Middle	3.41	6.9	3.04	0.24	1.74

collapse resistance of steel frame structures under fire was proposed based on the collapse time fragility curves. Then, the collapse resistance of structures at different fire protection levels and in different compartment fire scenarios was studied using this evaluation method.

Firstly, the collapse time sensitivity analysis was carried out to illustrate the effect of random variables on the collapse time of structures in a fire. Through the analysis, the steel yield strength and column fire protection coating thickness were shown to have a great influence on the structural collapse time. The results suggest that the column is important for structural collapse resistance in a fire. The results obtained using the model are consistent with those of previous studies (Agarwal and Varma, 2014; Jiang and Li, 2017b).

Then, the collapse fragility curves of the structures at different fire protection levels were fitted. Considering the uncertainty of the random variables, the collapse time of the structure was likely to be less than the design demand time. The collapse fragility curves of the structures in different compartment fire scenarios were also fitted. The results show that a short span compartment fire and a corner compartment fire are relatively dangerous for structural collapse. We recommend that the fire protection of components in these compartments should be enhanced to prevent the structure from collapsing under fire.

Furthermore, according to the statistical parameters of structure collapse time, the reliability index of structural collapse was also calculated. The collapse resistance of structures in a fire could be quantitatively evaluated by the index. The reliability indexes of collapse time ranged from 1.16 to 3.16 (Tables 3 and 4). However, the design reliability index of the common structure was 3.7. To satisfy the design reliability index, the calculated structural collapse time should be divided by a partial safety factor γ_{cp} greater than 1.

According to the reliability indexes in Table 4, the collapse resistance of structures in short span and corner compartment fire scenarios should be improved. The COV and target reliability index β_{tg} obtained by the analysis model proposed in this study could be used to adjust the fire resistance time of components in corner and short span compartments, so as to improve the collapse resistance performance of structures under these two fire scenarios. To meet the target reliability index β_{tg}, the fire resistance time of the components should be increased to γt_{dm}. The method for calculating γ is as follows:

$$\gamma = \frac{1}{1-\beta_{tg} \cdot \mathrm{COV}}. \tag{6}$$

Using the adjusted fire resistance time (γt_{dm}) to design the fire protection of components could improve the collapse resistance of the whole structure in a compartment fire and reduce the probability of collapse in the design demand time t_{dm}. Taking the collapse performance of the structure in a corner and a short span compartment fires as examples, the fire resistance time of components in a corner compartment and a short span compartment should be increased to 3.28 h and 3.45 h, respectively, to behave similarly to a long span compartment. As the collapse resistance of the structure in a short span compartment fire is lower, a greater increase in the fire resistance time of the components is needed. In future research, this problem will be studied more systematically.

Note that this study was a preliminary investigation which had some limitations. To ensure the duration of the fire, the highly idealized ISO 834-1 standard fire curve was selected. The uncertainty of fire, which also has an important effect on structural collapse time, was not considered. Therefore, the influence of compartment fire variation is underestimated in this study. In further research, a better compartment fire model, such as the parametric fire model or the zone model, will be selected to study the effect of the uncertainty of the fire model on the structural collapse time. Meanwhile, more examples should be calculated to determine the partial coefficient of collapse time.

6 Conclusions

This paper presents a probabilistic analysis of the collapse time of a steel frame structure in different fire scenarios considering the uncertainty of the material properties and fire protection coatings. Furthermore, according to the structural collapse time fragility curve and reliability index, a method to evaluate the collapse resistance of steel frame structures under fire is proposed. Finally, the collapse resistance of structures at different fire protection levels and in different compartment fire scenarios was investigated using this evaluation method. The major conclusions can be summarized as follows:

1. The uncertainty of material properties and fire protection coatings will cause uncertainty in the structural collapse time under fire. A structural collapse time fragility curve was established using the Latin hypercube sampling method, and the statistical parameters and reliability indexes of structural collapse time were obtained according to the fragility curve. The collapse performance of structures could be quantitatively evaluated by using the reliability index of collapse time.

2. According to the sensitivity analysis results, the steel yield strength and column fire protection coating thickness have a greater influence on the collapse time of a steel frame in a fire. We suggest that the column is important for structural collapse resistance in compartment fire scenarios.

3. Considering the uncertainty of the random variables, the structural collapse time is likely to be less than the fire demand time. The uncertainty of random variables has a greater impact on collapse time of structures at a high fire protection level. Therefore, stricter measures should be taken for structures with a high fire protection level in the design of structural fire resistance. The risk of structural collapse also differs when fire occurs in different compartments. The short span compartment and corner compartment fire scenarios were relatively dangerous for structural collapse. We recommend that the fire protection of components in these compartments should be enhanced to prevent the structure from collapsing in a fire.

4. Under the conditions of different fire scenarios and fire protection levels, the reliability indexes of structural collapse time ranged from 1.16 to 3.16. These reliability indexes are all smaller than the designed reliability index of 3.7. In addition, note that the influence of compartment fire variation was

underestimated in this study. In an actual fire, the dispersion of structural collapse time may be greater. To satisfy the designed reliability index, the calculated collapse time of the structure should be divided by a partial safety factor γ_{cp} greater than 1.

Contributors

Jin-cheng ZHAO designed the research. Wen REN analyzed the corresponding data and wrote the first draft of the manuscript. Wen REN and Jin-cheng ZHAO revised and edited the final version.

Conflict of interest

Wen REN and Jin-cheng ZHAO declare that they have no conflict of interest.

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