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Fire Resistance Calculation Model for Steel-Reinforced Concrete Frame Structures

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ABSTRACT: Frame structures with steel-reinforced concrete (SRC) columns and reinforced concrete (RC) beams are widely used in high-rise buildings, which are susceptible to significant fire hazards. Therefore, analyzing and designing the fire resistance of such SRC-RC frame structures (hereinafter referred to as SRC frame structures) is critical. Developing an effective calculation model for fire resistance is essential, as it provides a robust tool for assessing fire response and facilitating fire-resistant design. In this study, a fire resistance calculation model for SRC frame structures was developed using sequential thermal-mechanical coupling, with carefully selected thermal parameters and material constitutive models. The model was further refined by appropriately determining element types, boundary conditions, and numerical solution methods. Using the proposed model, the temperature distribution and deformation in fire resistance tests of SRC frame structures were numerically simulated. The calculated results agree well with experimental data, validating the accuracy and reliability of the proposed model for analyzing the fire resistance performance of SRC frame structures. Based on this validated model, a systematic analysis of fire resistance was conducted, revealing the failure mechanisms and internal force redistribution of SRC frame structures under fire conditions. The results indicate that in the early stage of a fire, thermal expansion induces axial compression in the beams, thereby reducing bending moments at both beam ends and the mid-span. In the later stage, as beam deflection increases, the axial compression induced by thermal expansion gradually diminishes and eventually disappears.

KEYWORDS: Fire resistance; calculation model; steel-reinforced concrete; frame structures; sequential thermal-mechanical coupling

1 Introduction

Steel-reinforced concrete (SRC) frames are widely used in building construction. Given that fire can cause severe structural damage, evaluating the fire resistance of SRC frames is crucial to ensure safety under extreme conditions.

Considerable efforts have been devoted to understanding the fire resistance behavior of SRC structures. Huang et al. [1], Yu et al. [2], Han et al. [3], and Du et al. [4] reported experimental and theoretical studies on SRC columns, contributing to the development of reliable finite element models for predicting their fire resistance limits. Li et al. [5] proposed a practical approach for calculating the fire resistance limit of composite columns composed of high-strength steel and high-strength concrete. Li et al. [6] numerically investigated the fire resistance of concrete-encased steel slender columns with high-strength materials and

proposed a simplified design method for evaluating their performance. Zhang et al. [7] numerically and experimentally investigated the fire behavior of thin-walled steel tube-confined reinforced concrete (RC) stub columns under axial compression, evaluating the effects of key parameters on their fire resistance. Wang et al. [8] numerically and experimentally investigated the fire resistance of steel-reinforced reactive powder concrete columns, examining the effects of key variables on their fire performance. Huang et al. [9] conducted fire tests on restrained SRC columns and established a mechanical model for their behavior at elevated temperatures. Mao et al. [10] demonstrated that incorporating shaped steel into concrete-filled steel tubular (CFST) columns significantly enhanced fire performance. Shao et al. [11] developed a design method for the fire resistance capacity of reinforced-concrete-filled steel tube columns under axial compression, explicitly distinguishing the contributions of the steel tube, concrete core, and reinforcement. Song et al. [12] presented a method for evaluating the fire resistance of SRC beam-column joints. Quan et al. [13] experimentally investigated steel-reinforced ultra-high toughness cementitious composite beam-to-column joints (SRUJs) under fire exposure and found that while no spalling occurred in SRUJs, explosive spalling was severe in conventional SRC beam-column joints.

Han et al. [14] carried out fire resistance tests on CFST frames and observed that the failure mode of the frame was directly related to the relative load-bearing capacity of beams and columns at elevated temperatures. Wang et al. [15] identified two distinct failure modes in SRC column–RC beam frames: beam-induced failure and column-induced failure. Xu and Bao [16] examined the fire-induced mechanical behavior of entire structures and concluded that overall fire performance was highly complex. Lan et al. [17] performed a numerical study on RC assemblies using a sequential thermal-mechanical coupling approach, capturing the effects of fire on failure modes, load-bearing capacity, and the evolution of load-resisting mechanisms in RC frames. Li et al. [18] evaluated the fire resistance of planar frames consisting of circular CFST columns and RC beams, finding that appropriate protective measures could shift the failure mode from column-dominated to beam-dominated. Li et al. [19] developed a refined finite element model of a prefabricated RC frame structure and analyzed its fire resistance performance. Li et al. [20] investigated the collapse resistance mechanisms and response of self-resilient composite frame substructures under fire-induced progressive collapse, identifying key factors influencing their performance in such scenarios. Lou et al. [21] proposed a thermo-mechanical framework to analyze the fire behavior of a single-span steel portal frame, observing that the fire response of a restrained steel beam could be divided into a thermal expansion stage and a catenary stage. Haouach et al. [22] carried out numerical investigations into the behavior of a three-dimensional (3D) RC frame subjected to fire following an earthquake. Their results revealed that factors such as vertical loads, story drifts, fire scenarios, and structural damage collectively influenced building collapse probabilities, underlining the critical role of holistic risk assessment in structural design. de Souza et al. [23] developed a probabilistic thermo-mechanical finite element framework for assessing the fire resistance of RC structures. Liu et al. [24] investigated the residual bond performance of RC structures after coupled corrosion-fire damage. Their results showed that fire-induced thermal stress and phase transformation accelerated bond deterioration. Based on these findings, they proposed a strength prediction formula and a constitutive model to evaluate structural fire resistance under multi-factor coupled damage. Jin et al. [25] combined experiments and numerical simulations to study RC beam-column assemblies under fire and middle-column removal, revealing that fire reshaped the load redistribution path and weakened progressive collapse resistance. Yao et al. [26] investigated the progressive collapse resistance of RC beam-column connections under fire conditions, revealing how fire exposure altered load redistribution mechanisms within the structural assembly. Qi et al. [27] conducted a collapse analysis of restrained CFST columns under fire conditions, elucidating the influence of axial restraint stiffness on fire-induced failure mechanisms.

Existing studies, as reflected in the literature review above, have mainly concentrated on experimental investigations of individual columns or frames, while relatively little effort has been devoted to the development of calculation models. Such models are crucial for establishing practical fire safety design guidelines. To fill this gap, this study proposes a calculation model for evaluating the fire resistance of frame structures with SRC columns and RC beams (hereinafter referred to as SRC frame structures), which is validated against experimental results. Based on this model, a systematic analysis of the fire resistance behavior of such structures is conducted.

2 Introduction to the Fire Resistance Test of SRC Frame Structures

Wang et al. [28] conducted fire resistance tests on 5 frame specimens comprising SRC columns and RC beams. As detailed in Table 1, the rectangular column cross-sections in the test measured either 260 mm × 260 mm (standard size) or 240 mm × 240 mm. The columns incorporated two types of H-shaped structural steel: H120 mm × 100 mm × 12 mm × 12 mm (Section 1) and H120 mm × 100 mm × 12 mm × 20 mm (Section 2). The beams featured a rectangular cross-section of 200 mm × 260 mm, and all specimens included floor slabs. Table 1 lists the specimen parameters, while Fig. 1 illustrates the specimen dimensions, reinforcement configuration, and thermocouple arrangement. Fig. 1b shows the thermocouple locations within the column section: Thermocouple ① was positioned at the lower part of the longitudinal reinforcement; Thermocouple ② was at the end of the steel flange, and Thermocouple ③ was at the midpoint of the steel web. The thermocouple ④ arrangement for the beam section is presented in Fig. 1c, with Thermocouple ④ located at the upper part of the longitudinal reinforcement, Thermocouple ⑤ at the midpoint of both the section height and width, and Thermocouple ⑥ at the lower part of the longitudinal reinforcement.

Table 1: Details of SRC frame specimens.

Specimen No.	Column Dimension (mm × mm)	Column Steel	Concrete Grade	Column/Beam Load (kN)	Parameter Variation
SRCF01	260 × 260	Section 1	C35	1963/63	Standard specimen
SRCF02	260 × 260	Section 1	C50	1963/83	Concrete grade
SRCF03	240 × 240	Section 1	C35	1963/63	Column dimension
SRCF04	260 × 260	Section 2	C35	1963/63	Column steel ratio
SRCF05	260 × 260	Section 1	C35	1556/63	Column compression load ratio

The section steel was grade Q345, while the longitudinal reinforcement and stirrups were grades HRB400 and HPB235, respectively. The concrete grades were C35 and C50. Details regarding the measured mechanical properties of these materials are provided in Reference [28].

The test commenced with the application of concentrated loads at the column tops and beam mid-spans, with specific load values detailed in Table 1. Upon reaching and stabilizing at the designated loads, the heating system was activated. Subsequently, the furnace temperature was increased in accordance with the ISO 834 standard temperature–time curve.

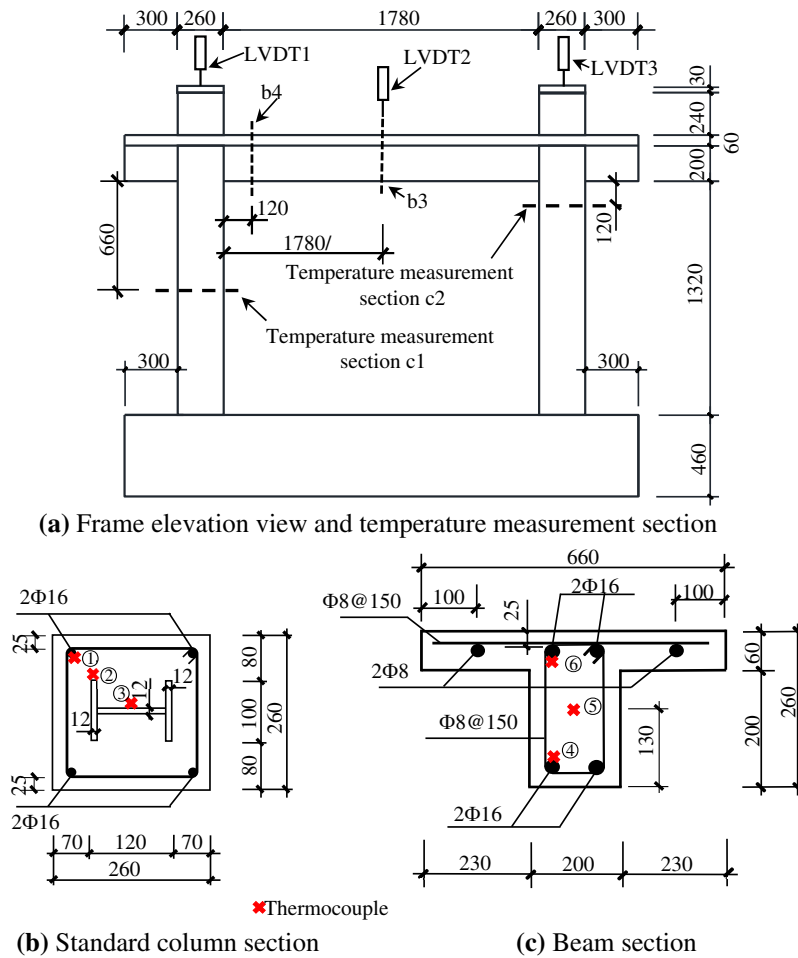


Figure 1: Details of columns and beams (unit: mm).

3 Calculation Model for Fire Resistance Performance of SRC Frame Structures

A sequential coupling method was employed to develop a calculation model for the fire resistance performance of SRC frame structures. The model comprises two components: a temperature-field calculation model and a high-temperature mechanical performance calculation model.

3.1 Thermal Parameters

The thermal parameters of steel and concrete, including density (ρ), thermal conductivity (λ), and specific heat capacity (c), were derived from the formulas proposed by Lie [29]. Although Eurocode 1 [30] also specified thermal parameters for steel and concrete, those parameters were designed for use with the high-temperature mechanical property parameters provided in Eurocode 2 [31]. In this study, the thermal parameters from Lie [29] were adopted to maintain compatibility with the corresponding mechanical property parameters at elevated temperatures. Both sets of parameters were validated by experimental data and demonstrated to be accurate.

3.2 High-Temperature Mechanical Properties of Materials

The uniaxial compressive stress–strain relationship of concrete and the stress–strain relationship of steel at high temperatures, as established by Lie [29], were adopted in this study.

3.3 Establishment of the Finite Element Calculation Model

3.3.1 Temperature Field Calculation Model

A finite element heat transfer model was established using solid heat transfer elements to investigate the temperature distribution within the SRC frame structure. Perfect thermal contact was assumed at all material interfaces; thus, the thermal resistance between concrete and reinforcement, as well as between concrete and steel, was neglected. A tie constraint was applied in the finite element model to enforce temperature equality at both the concrete–reinforcement and concrete–steel interfaces. As demonstrated in Section 4.1, the temperature field error induced by this tie constraint is relatively small, as confirmed by experimental results. The top surface of the slab was treated as a heat dissipation surface, where a convective heat transfer coefficient of $9 \text{ W}/(\text{m}^2 \cdot ^\circ\text{C})$ was adopted to account for combined convective and radiative heat transfer from the slab top to the ambient environment [30]. The remaining surfaces of the model, which were exposed to fire inside the furnace, were assigned convective and radiative boundary conditions. According to Eurocode 1 [30], a convective heat transfer coefficient of $25 \text{ W}/(\text{m}^2 \cdot ^\circ\text{C})$ and an effective radiative emissivity of 0.7 were used for the fire-exposed surfaces.

Concrete and shaped steel were discretized using 3D linear heat transfer hexahedral elements (DC3D8). Each DC3D8 element consists of eight nodes, with each node assigned a single temperature degree of freedom. These elements utilize linear interpolation functions and employ full integration. The reinforcement was discretized using linear heat transfer link elements (DT3D2), each with two nodes and one temperature degree of freedom per node. These elements also use linear interpolation functions and leverage full integration. The temperature field was then computed through transient heat transfer analysis.

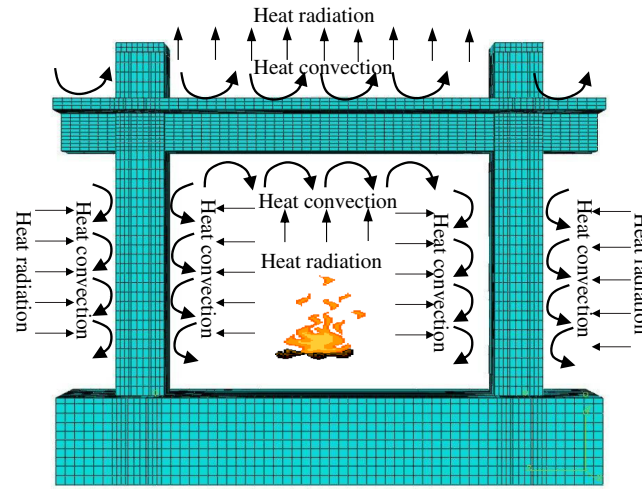
3.3.2 High-Temperature Mechanical Performance Calculation Model

The interface between the reinforcement and concrete was constrained using the tie constraint, indicating that no slip occurs between the two materials. Given that the structural response under fire conditions is quasi-static, slip between the reinforcement and concrete is negligible. The contact surface between steel and concrete was simulated using contact pairs, with a hard contact formulation in the normal direction and a Coulomb friction model in the tangential direction, assuming a friction coefficient of 0.3 [15]. During the fire resistance tests, the ground beam of the SRC frame specimen was fixed. Accordingly, in the fire resistance performance calculation model, the ground beam was assigned fixed boundary conditions. The finite element model for fire resistance performance is illustrated in Fig. 2, with the mesh division of steel and reinforcement shown in Fig. 2b.

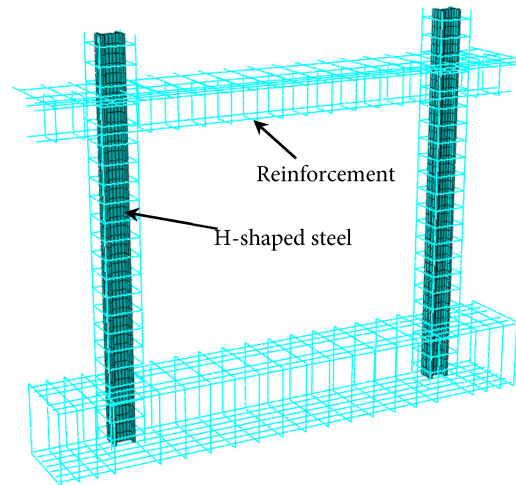
Both concrete and shaped steel were discretized using C3D8R elements. These elements are 3D hexahedral elements with reduced integration, featuring linear interpolation functions and eight nodes, with each node having three translational degrees of freedom (U1, U2, U3). The reinforcement was discretized using T3D2 truss elements, which also adopt linear interpolation functions and employ full integration.

A sequential thermo-mechanical coupling method was adopted to evaluate the mechanical performance of the SRC frame under fire conditions. In this approach, the temperature field obtained from the preceding heat transfer analysis was imported as a predefined field into the mechanical model. Subsequently, the structural response of the frame at elevated temperatures was computed using static analysis procedures. The

fire resistance calculation model exhibits significant material and geometric nonlinearities. An incremental-iterative method employing the Full Newton scheme was used to compute the mechanical performance of the SRC frame at elevated temperatures. An automatic incremental strategy was adopted: when convergence was readily achieved, the program increased the increment size to accelerate the analysis; conversely, when convergence became difficult, the increment size was reduced to facilitate convergence. A force convergence criterion with a tolerance of 5% was applied. Using this nonlinear computation strategy, the fire resistance performance of the SRC frame was evaluated.



(a) Temperature calculation model



(b) Mesh division of steel and reinforcement

Figure 2: Fire performance calculation model.

4 Comparison of Calculated and Experimental Results

4.1 Temperature Field

The temperature field calculation model was used to determine the thermal responses at all measurement locations. Fig. 3 compares the calculated and experimental temperature (T)–time (t) curves at selected measuring points. The predicted temperatures align closely with the experimental values, demonstrating the accuracy of the model. Similar agreement is observed for the other test specimens.

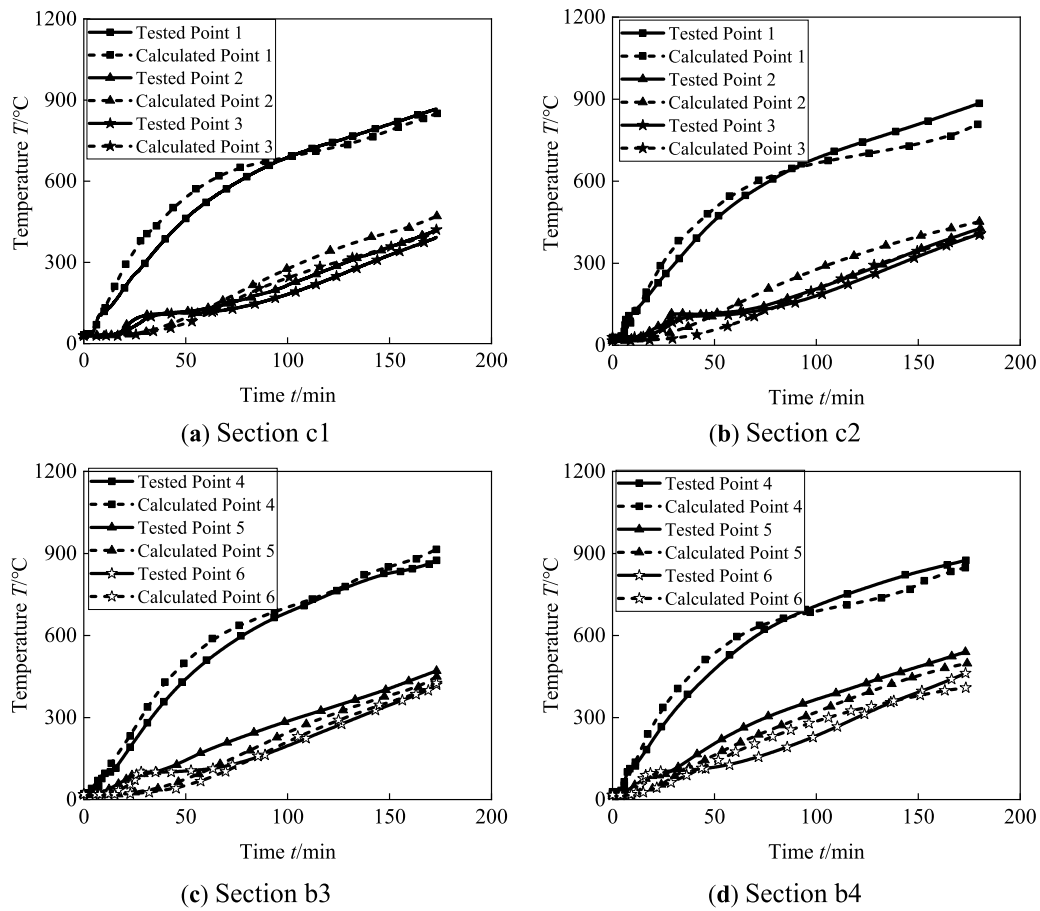


Figure 3: Comparison between the experimental and calculated temperature–time curves of specimen SRCF01.

To quantitatively assess the model's accuracy, the root mean square error (RMSE) between the calculated and experimental temperatures was computed over the entire test duration for each measurement point shown in Fig. 3. As shown in Fig. 3a, the RMSE values for Points 1, 2, and 3 in Section c1 are 50.3, 47.5, and 40.9°C, respectively. As shown in Fig. 3b, the RMSE values for Points 1, 2, and 3 in Section c2 are 49.2, 50.4, and 32.1°C, respectively. As shown in Fig. 3c, the RMSE values for Points 4, 5, and 6 in Section b3 are 38.5, 47.0, and 31.1°C, respectively. As shown in Fig. 3d, the RMSE values for Points 4, 5, and 6 in Section b4 are 42.6, 39.5, and 37.8°C, respectively. The maximum RMSE across all points is 50.4°C. Overall, these quantitative results demonstrate that the proposed temperature field calculation model reliably predicts the thermal response of the SRC frame structure under fire exposure.

Minor discrepancies exist between the measured and calculated temperatures at each measuring point. These deviations are primarily attributed to factors such as differences between the material thermal parameters proposed by Lie [29] and those of the actual materials used in the tests, crack development in the concrete specimens during testing, non-uniform distribution of concrete materials, and thermal resistance at the concrete–steel interfaces (including both reinforcing bars and shaped steel). Nevertheless, the calculated temperatures agree well with the measured values, with only minor deviations. Thus, the proposed temperature field calculation model is considered both reliable and accurate.

4.2 Failure Modes

During testing, the frame exhibited two distinct failure modes: column failure and beam-column failure. Column failure refers to axial compression failure outside the plane of the frame; therefore, the calculation model incorporated initial geometric imperfections in the out-of-plane direction. In accordance with the Chinese Code for Design of Concrete Structures (GB50010-2010), the out-of-plane eccentricity of the concentrated load at the column top was taken as the greater value between 20 mm and $h/30$, where h represents the SRC column width. Here, $h = 260$ mm, resulting in an eccentricity of 20 mm.

Fig. 4 presents the failure modes of specimens SRCF01 and SRCF02, comparing the experimentally observed failure patterns with the deformed shapes at failure predicted by numerical simulations using the proposed fire resistance calculation model. In Fig. 4, PEEQ denotes the effective plastic strain of the concrete, reflecting the strain magnitude and thus indicating the failure mode of the SRC frame specimen. Additionally, PE22 represents the vertical plastic strain of the SRC frame, capturing the tensile deformation of the concrete on the specimen surface and indirectly revealing the overall failure mode. The simulated failure modes and strain distributions agree well with the test results. Notably, the simulated failure modes of the remaining specimens also align closely with the experimental observations.

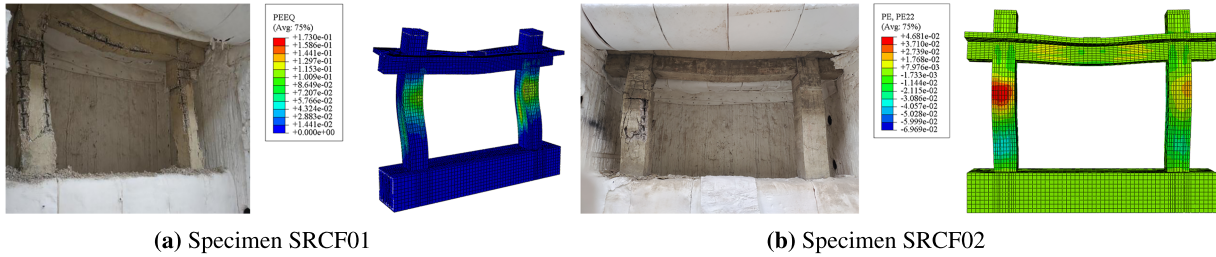
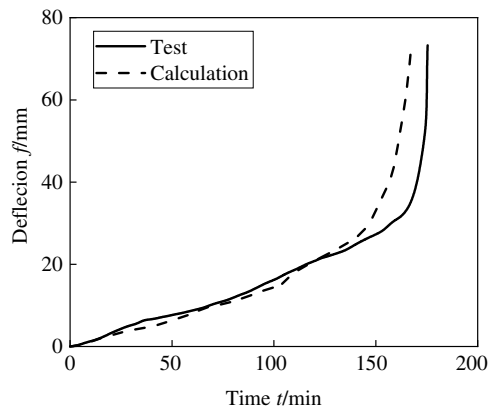


Figure 4: Comparison of calculated and experimental failure modes.

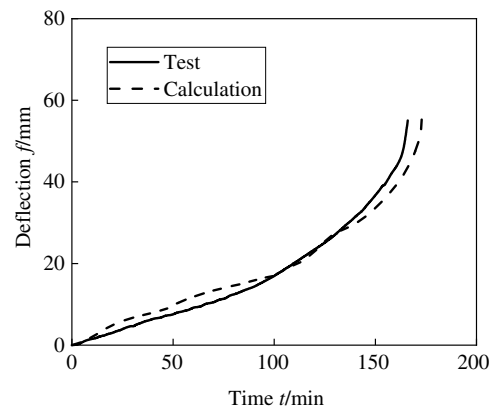
4.3 Deformation

Fig. 5 compares the experimental and calculated deflection (f)–time (t) curves for the mid-spans of the test specimens. The corresponding vertical displacements of the column tops are presented in Fig. 6, with upward displacement defined as positive. Overall, the calculated results agree well with the experimental data for both beam deflections and column vertical displacements. Specifically, for the duration up to the lesser of the calculated and experimental fire resistance limits, the RMSEs for the deflections in Fig. 5a–c are 1.42, 2.08, and 3.5 mm, respectively. For the displacements in Fig. 6a–d, the RMSEs over the same duration are 1.57, 1.87, 0.91, and 1.15 mm, respectively. As illustrated in Fig. 5, once the time exceeds the lesser of the two fire resistance limits, specimen failure occurs, characterized by a rapid increase in deflection without further time progression. Consequently, the RMSE between the experimental and calculated deflections cannot be evaluated beyond this point. Therefore, the RMSE was calculated only within the duration up to the lesser of the two fire resistance limits; the same approach was applied to Fig. 6.

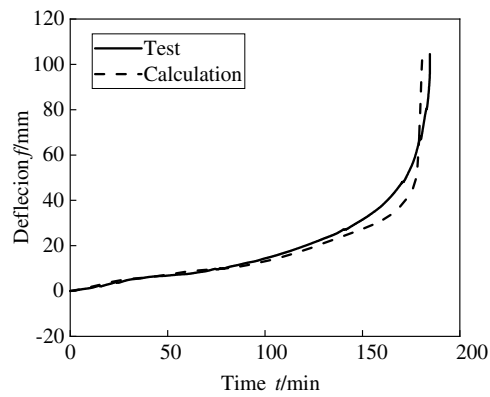
Minor discrepancies exist between the calculated and measured deformations of the SRC frame specimens. These deviations are attributed to differences between the assumed and actual material properties, as well as discrepancies in the temperature fields. Furthermore, concrete spalling under elevated temperatures influences deformation behavior. As the current fire resistance model for SRC frames does not accurately account for concrete spalling, this limitation further contributes to the observed deviations between calculated and measured deformations.



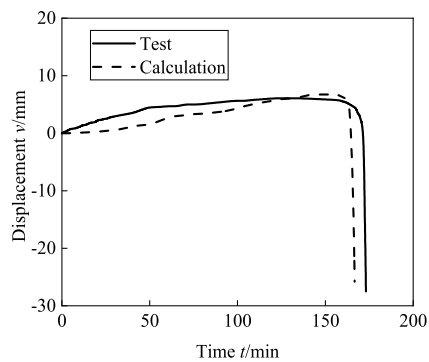
(a) Specimen SRCF01



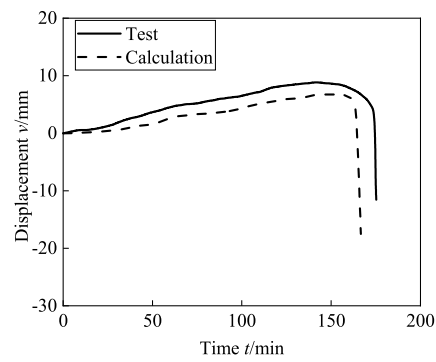
(b) Specimen SRCF02



(c) Specimen SRCF04

Figure 5: Comparison of calculated and experimental mid-span deflection (f) vs. time (t) curves.

(a) Left column of specimen SRCF01



(b) Right column of specimen SRCF01

Figure 6: (Continued)

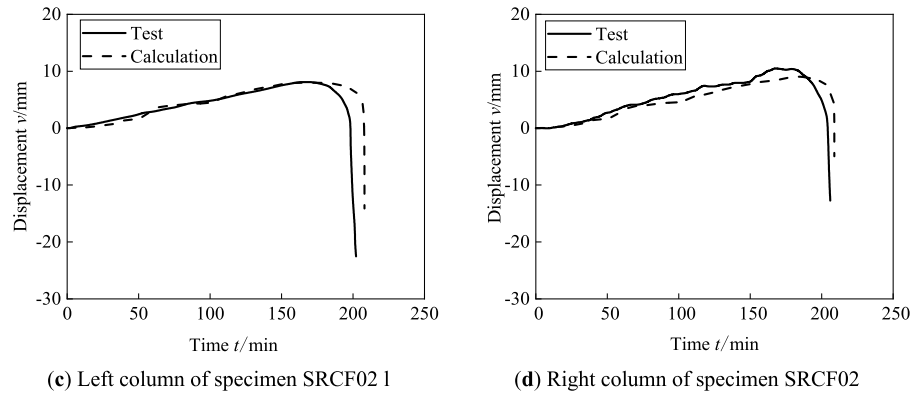


Figure 6: Comparison of calculated and experimental vertical column displacement (v) vs. time (t) curves.

5 Fire Resistance Analysis of Frame Structures

5.1 Parametric Design

Wang et al.'s [28] fire resistance tests on SRC frame structures identified column failure and beam-column failure as the primary failure modes. To further investigate the failure modes and mechanisms of such frames under elevated temperatures, a parametric study was conducted here using specimen SRCF01, varying the beam load ratio m (F_b/N_b) and the column axial compression ratio n (F_c/N_c). Here, F_b represents the beam mid-span load, and N_b denotes the load corresponding to the ultimate bending capacity of the frame beam at room temperature, calculated using the plastic limit load method. F_c is the concentrated load at the column top, while N_c is the sum of the bearing capacities of the concrete and steel components of the column section at room temperature. The ratio n was set to 0.4 and 0.6, and m to 0.26, 0.52, and 0.78 to systematically evaluate their influence on the failure modes and mechanisms.

5.2 Failure Modes

The parametric analysis reveals that variations in the beam load ratio (m) and column axial compression ratio (n) lead to two typical failure modes in the SRC plane frame: beam bending failure and column failure, as illustrated in Figs. 7 and 8, respectively. In these figures, PEEQ denotes the effective plastic strain of the concrete, reflecting the magnitude of plastic strain as previously described.

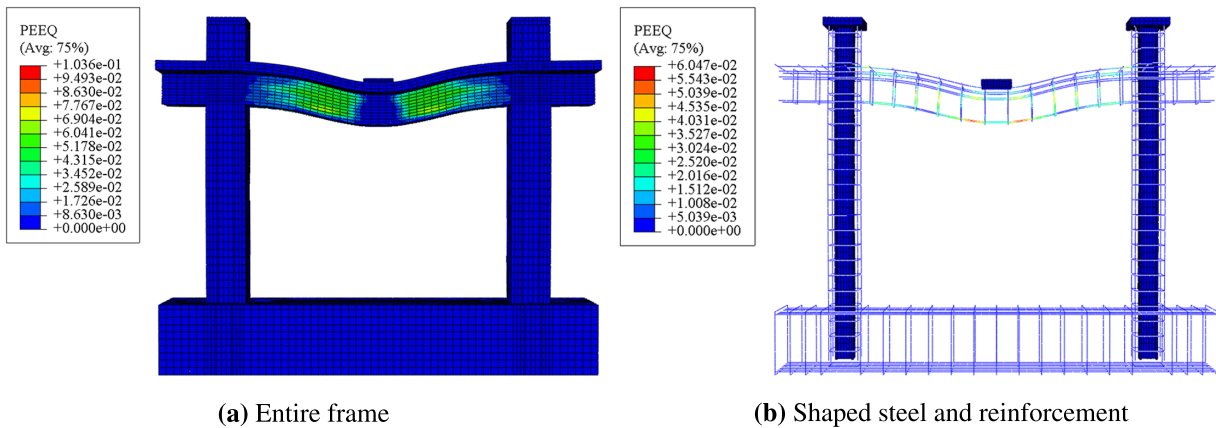


Figure 7: Beam failure modes for $n = 0.6$ and $m = 0.78$.

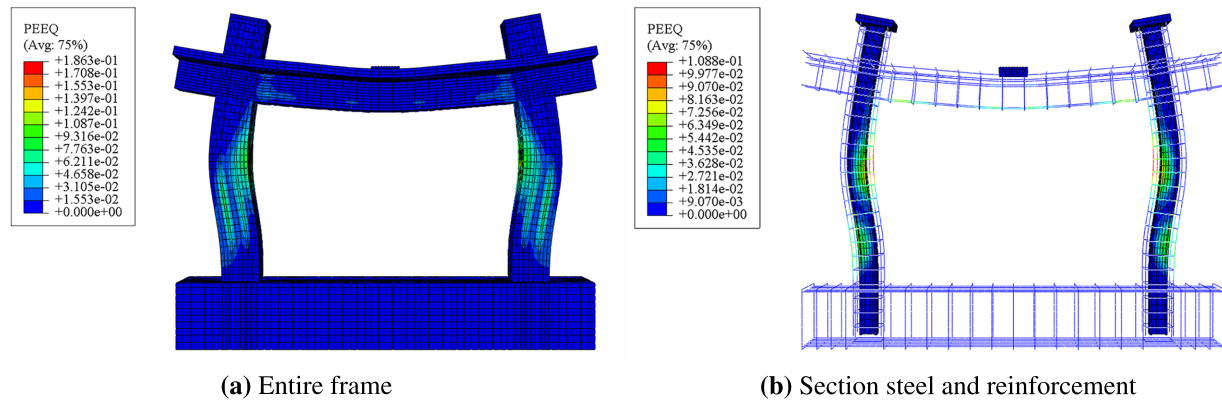


Figure 8: Column failure modes for $n = 0.6$ and $m = 0.26$.

The analysis indicates that beam failure occurs in the SRC frame structure at high temperatures when $n = 0.6$ and $m \geq 0.52$. Conversely, column failure occurs when $m = 0.26$ and $n \geq 0.6$.

5.3 Internal Force Redistribution

Using the typical beam failure mode ($n = 0.4$, $m = 0.26$) as an example, the variations in axial force N and bending moment M were analyzed for three typical sections: the beam mid-span, the beam end, and the column mid-height. Here, axial force N is defined as positive for tension and negative for compression, while bending moment M is positive when the lower side of the beam and the outer side of the column are in tension.

Fig. 9 presents the axial force (N)–time (t) relationship at the beam mid-span section. In the early stage, the magnitude of the axial compressive force increases in magnitude (absolute value) due to thermal expansion of the beam, which is restrained by the columns. Subsequently, as beam deflection develops, the axial compression gradually decreases and eventually transitions to tension.

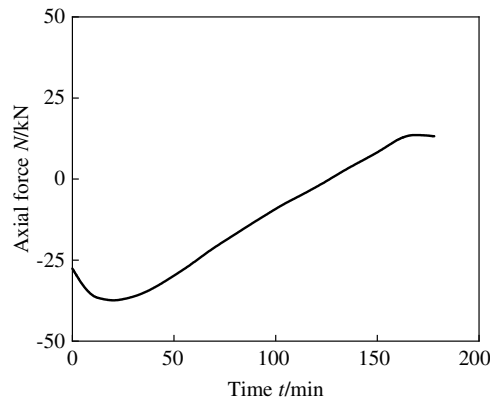


Figure 9: Axial force (N)–time (t) curve at the mid-span section of the beam.

Fig. 10 illustrates the bending moment (M)–time (t) relationships for both the mid-span and end sections of the beam. In the early stage of fire exposure, thermal expansion generates axial compression, which reduces the absolute values of the positive bending moment at mid-span and the negative bending moment at the beam ends. This reduction is attributed to the arch effect induced by thermal expansion, which creates a compression zone within the beam. As the temperature continues to rise, increased

deflection diminishes this arch effect, allowing the moments to gradually recover. At the fire resistance limit, the mid-span positive moment and the beam-end negative moment reach their bending capacities at high temperatures.

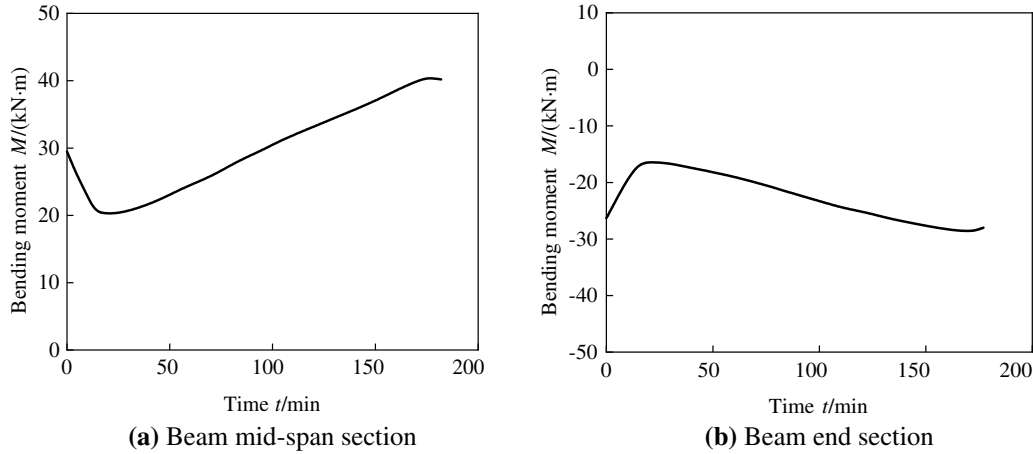


Figure 10: Bending moment (M)–time (t) curves in the beam.

Fig. 11a presents the axial force (N)–time (t) relationship at the column's mid-height section, indicating that the axial force remains nearly constant throughout fire exposure. In contrast, Fig. 11b reveals that the absolute bending moment (M) at the same location increases progressively over time. This increase occurs because beam deflection induces joint rotation via the negative bending moment in the beam, causing the upper column to deflect outward and thereby increasing the bending moment in the column's mid-section.

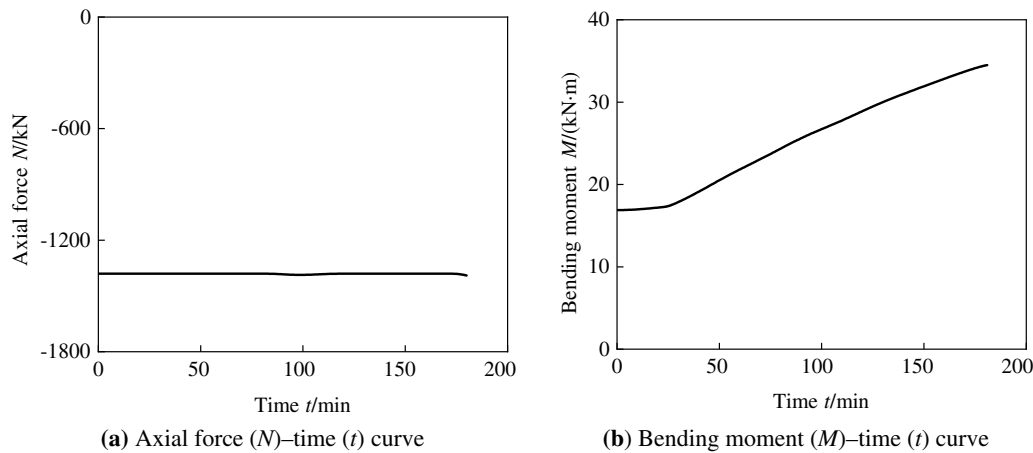


Figure 11: Internal force–time relationships in the mid-section of the column.

6 Conclusions

A calculation model for evaluating the fire resistance of SRC frame structures was developed by adopting appropriate thermal material parameters and constitutive relationships. The calculated results agreed well with the experimental data. This model was subsequently employed to analyze the mechanical behavior of SRC frame structures under elevated temperatures. Based on the test and analysis results, the following conclusions are drawn:

- (1) Under fire conditions, SRC frame structures exhibit two typical failure modes: beam failure and column failure.
- (2) In the early stage of a fire, thermal expansion induces axial compression in the beam, creating an arch effect that reduces the absolute bending moments at both the mid-span and beam ends. As the temperature rises further, the increasing beam deflection gradually weakens this arch effect, allowing the bending moments to recover progressively.

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Availability of Data and Materials: The data supporting the findings of this study are available from the Corresponding Author upon reasonable request.

Ethics Approval: This study did not involve human participants, animals, or sensitive data; therefore, ethics approval was not required.

Conflicts of Interest: The authors declare no conflicts of interest.

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