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A hybrid steel–shape memory alloy prestressed concrete beam for improved fire resistance

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Summary

Prestressed concrete beams are widely used in structures and infrastructures for their superior control effects on cross-sectional area, cracking, and deformation. However, the fire resistance of ordinary prestressed beams is unsatisfactory due to the serious material degradation of concrete and steel, as well as the prestress loss at elevated temperatures. To achieve an acceptable fire resistance for prestressed concrete beams, the introduction of shape memory alloy (SMA) is proposed via the hybrid usage of prestressed steel and SMA bars. When the hybrid steel–SMA prestressed beam is exposed to fire, both the strength and modulus of elasticity of prestressed SMA bars will increase, which can compensate for the material degradation of prestressed steel bars. In addition, a desirable recovery stress can be generated in SMA bars at elevated temperatures and further compensate for the prestress loss of prestressed steel bars. To study the fire resistance of the proposed beam, the fitting formulae of material properties of concrete, steel, and SMA at elevated temperatures are given firstly. Then, the analytical model and working mechanism of the hybrid beam are also presented. Further, the methodology for the prediction of fire resistance, and the formulae for the calculation of beam's flexural bearing capacity and deflection, are also presented. A parametric study is further conducted to investigate the relationships between fire resistance and the area and location of prestressed SMA bars. Moreover, an optimal analysis is also established to obtain the most economical material cost of SMA bars. As a result, the proposed prestressed beam can achieve a better fire resistance in terms of both flexural bearing capacity and deflection control.

KEYWORDS

concrete beam, fire resistance, genetic algorithm, prestressed beam, shape memory alloy

1 | INTRODUCTION

With the increasing requirement of span to depth ratio for horizontal flexural members such as beams, the prestressing technology has been widely employed in the modern structural engineering and bridge engineering.^{1–4} The prestressing technology improves the performance of concrete beams by enhancing flexural bearing capacity, delaying, and limiting concrete cracking, as well as significantly reducing the structural weight and deflection.^{5,6} Reinforced concrete (RC) beams built in existing buildings can also be externally prestressed to upgrade their

bearing capacity and reduce their deflection. In a word, the prestressed concrete beams play a vital role in RC structures and infrastructures.

Fire resistance is essential for RC structures to guarantee the safety of occupants' lives and property.⁷ Unfortunately, prestressed concrete beams are vulnerable to building fires,⁸ because the severe material degradation^{9–13} and prestress loss^{14–16} at elevated temperatures will lead to the overdeformation and even the destruction of prestressed beams. As the temperature increases, the reduction of yield strength and modulus of elasticity will be induced in steel bars,^{9,12,13} and cause insufficient flexural bearing capacity and stiffness of prestressed beams.^{6,9,16,17} Although a thick concrete cover is commonly prescribed for steel bars, it cannot guarantee an effective fire protection when fire-induced cracking and spalling occur.¹⁰ Specially, prestressed steel bars are more sensitive to the elevated temperatures.¹⁴ In addition to the considerable degradation of material properties, serious prestress loss due to transient high-temperature stress relaxation may result in more catastrophic consequences.¹⁵ The prestress loss resulting from a fire disaster will lead to an excessive deflection, thus accelerating concrete cracking, rebar exposure, and heat conduction, and eventually cause the prestressed beam to prematurely reach its fire resistance limit. Due to the inevitable material degradation and prestress loss under fire exposure, the prescribed performance of ordinary prestressed beam is unattainable,^{6,9,16,17} which will hinder the evacuation and rescue operations in a fire disaster.

With the development of intelligent materials, the potential of shape memory alloys (SMAs) have drawn increasing attention since they were discovered in 1960s.^{18,19} Not only the super-elasticity but also the shape memory effect of a high standard and quality leads to a wide application in fields of aerospace and civil engineering.^{18–35} The phase of SMA is determined by not only temperatures but also stresses, and it can transform from one phase to another under the effects of varying temperatures or stresses.³⁶ Furthermore, the deformation of SMA induced by stress can be recovered by heating SMA to a specific temperature, which is referred as the well-known shape memory effect.¹⁸ Therefore, a satisfactory recovery stress can be provided by SMA during a fire,³⁷ which can be considered as an effective compensation for the prestress loss of prestressed bars, thus improving the flexural stiffness of the beam, and delaying or limiting the cracking. The steady-state high-temperature test of Ni–Ti SMA showed that the yield strength and modulus of elasticity of SMAs can increase by a few times of that at room temperature via heating SMAs.³⁸ Thus, the severe material degradation of steel bars in a fire disaster can be compensated by the hybrid usage of SMA and steel bars, which will lead to a robust flexural bearing capacity of the beam. From this view, researchers have made some attempts to improve the performance of RC beams by employing SMA bars. Wong and Liu³⁹ proposed a hybrid steel–SMA concrete beam to resist the fire hazard, which successfully maintained the flexural bearing capacity at a desirable level, and the excessive deformation was also avoided. Rojob and El-Hacha⁴⁰ presented a SMA strengthening technique, where SMA bars were internally anchored on the tension side of beam. Thus, a tension force was generated in the SMA bars when being heated, leading to a significant enhancement in the flexural bearing capacity of the beam.

Inspired by the research of Wong and Liu,³⁹ as well as Rojob and El-Hacha,⁴⁰ an attempt is made in this study to develop an unbonded and post-tensioned steel–SMA prestressed beam, which is capable of improving the fire resistance of prestressed concrete beams under the limitation of building space. Contrast to the studies of Wong and Liu,³⁹ and Rojob and El-Hacha,⁴⁰ the hybrid usage of prestressed steel and SMA bars is employed to fully utilize the shape memory effect of SMA and then to improve the performance of the prestressed beams when exposed to fire. In the present study, the fitting formulae of material properties in the hybrid steel–SMA prestressed beam at elevated temperatures are first given. Then, the analytical model and the working mechanism of the hybrid prestressed beam are also presented. Moreover, a theoretical analysis is conducted to calculate the effective prestress, flexural bearing capacity, and deflection, thus to predict the fire resistance. A parametric study is further implemented to investigate the relationships between fire resistance and design parameters. Furthermore, an optimal analysis is conducted to obtain the optimal material cost of steel–SMA prestressed beam subjected to a given fire resistance requirement. Finally, several conclusions concerning the proposed beam are given.

2 | MATERIAL MODELS OF THE PROPOSED BEAM IN THE FIRE

2.1 | Concrete

2.1.1 | Temperature profile

The severe deterioration of concrete's strength would happen when the RC beam is exposed to fire, which depends on the experienced cross-sectional temperature profiles.⁴¹ One approach to derive the cross-sectional temperature fields is

to conduct fire tests for full-scale structural members,⁴² whereas it is too time-consuming and expensive.⁴¹ Except for fire tests, finite element or finite difference procedures can also be employed to implement the thermal analysis, which are based on the principle of energy balance and not restricted to the type of materials or the geometrical shape of structural members.⁴³ However, a detailed finite element or finite difference procedure is also time-consuming and sophisticated. In addition, design graphs recommended by Eurocode⁴⁴ and ACI code⁴⁵ can also be utilized to determine the temperature profiles, which provide series of temperature contour plots at varying fire exposure time. Nevertheless, these contour plots are given in rough time intervals (0.5 or 1.0 h) and obtained based on the fire test with respect to several specific types of concrete (siliceous aggregate concrete for Eurocode, and normal-weight and semilight-weight concrete for ACI code).⁴¹

Alternatively, Wickstrom⁴⁶ proposed a simplified expression to evaluate the sectional temperature considering the depth and fire exposure time. However, this method could not account for growth rates of the gas temperature in a fire or the type of the concrete aggregate. Further, Kodur et al.⁴¹ presented an improved expression based on Wickstrom's method, which could take concrete characteristics and fire exposure conditions into consideration. This improved method is based on the nonlinear regression analysis with respect to a database of cross-sectional temperatures in fire disasters, and is well fitted in temperature evolution within the RC members in varying real fire scenarios. From the perspective of conciseness and feasibility, Kodur's model is employed in this study to conduct the following analyses, where the temperature T (in °C) of a given point (x,y) within the concrete cross-section (as shown in Figure 1) can be expressed as⁴¹

$$\begin{cases} T = c_2 [-1.481\eta_x\eta_y + 0.985(\eta_x + \eta_y) + 0.017] T_g \\ \eta_x = 0.155 \ln \frac{t/60}{x^{1.5}} - 0.348\sqrt{x} - 0.371 \\ \eta_y = 0.155 \ln \frac{t/60}{y^{1.5}} - 0.348\sqrt{y} - 0.371 \end{cases}, \quad (1)$$

where c_2 is a material parameter and equal to 1.0 with respect to the normal strength and carbonate concrete. η_x and η_y are the heat transfer factors induced through one fire-exposed surface. x and y are the shortest distances (in m) from the given point (x,y) in the concrete cross-section to the fire exposure surfaces. t is the duration (in min) of fire exposure of the cross-section, and T_g is the gas temperature (in °C).

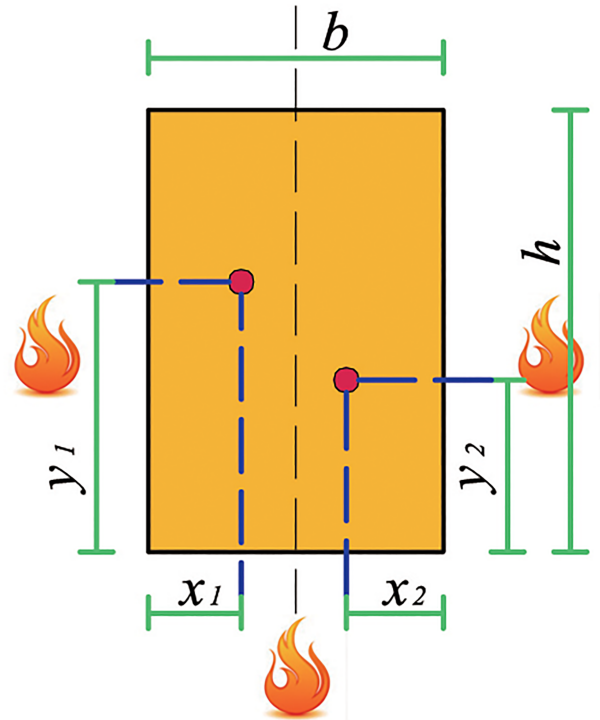


FIGURE 1 Definitions of the coordinates x and y within the concrete cross-section

As for the gas temperature T_g of an enclosed building environment in a fire scenario, it is very complex and determined by several factors, such as the size of compartments, the characteristics of fuels, and the presence of fire protection measures.⁴⁷ With time went by, the ignition of all combustible surfaces could happen because of the radiation resulting from the smoke layer. This phenomenon is referred as flashover, and a unique temperature T_g could be assumed for the whole compartment at this point. By establishing the equilibrium equations concerning the heat produced by fire, and the heat dissipated by conduction, convection, and radiation, the post-flashover parametric curve⁴⁸ could be obtained and then describe the relationship between gas temperature T_g and fire exposure time t (as shown in Figure 2). Moreover, another well-known post-flashover curve, ISO-834 standard curve,⁴⁹ is given as

$$T_g = 20 + 345 \log_{10}(8t + 1), \quad (2)$$

where T_g is the gas temperature (in °C) and t is the fire exposure time (in min). Although this ISO-834 curve is monotonically increasing and less realistic than the parametric one (as shown in Figure 2), it is independent of building properties, and it allows a definition of fire resistance and a comparison for different design schemes exposed to fire.⁴⁷ Hence, ISO-834 curve is employed by many scholars in not only standardized fire tests but also theoretical studies,^{17,42} and is also utilized in this study.

2.1.2 | 500°C isotherm method

The stress-strain relationship of concrete at a specific temperature is given by Eurocode,⁴⁴ whereas their usage is restricted in directly calculating the bearing capacity and stiffness of the concrete cross-section.³⁹ Moreover, the 500°C isotherm method recommended by Eurocode⁴⁴ allows the use of a reduced and equivalent cross-section for the simplified calculation, which is widely employed by many scholars in the research of structural fire safety.^{39,50} This method relies on an assumption that the thickness of the fire-damaged zone in the concrete cross-section is equivalent to the average depth d outside the 500°C isotherm contour,⁴¹ as shown in Figure. The cross-section is then reduced: The inner concrete zone maintains its stiffness and strength, whereas the peripheral fire-damaged zone is assumed to have no contribution to concrete's stiffness and strength. In such a way, the calculation efficiency can be significantly improved.

To obtain the equivalent temperature field, equations could be established with respect to the thickness d according to the 500°C isotherm method,⁴⁴ which can be expressed as

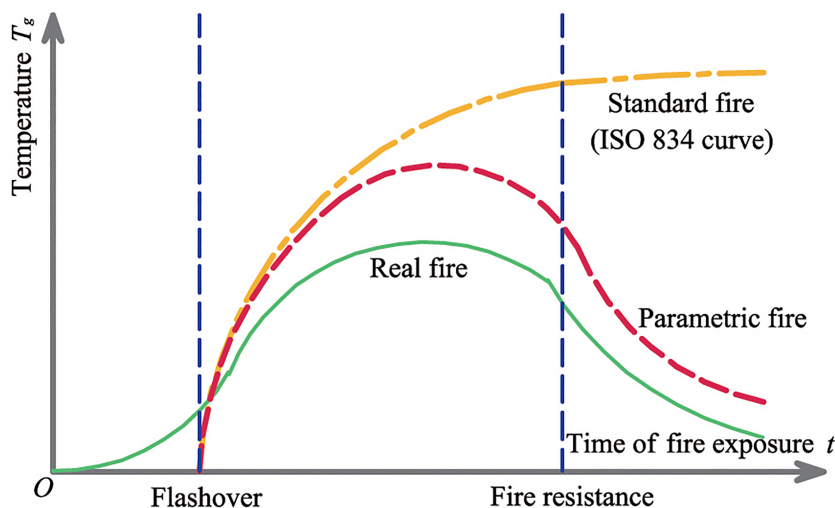
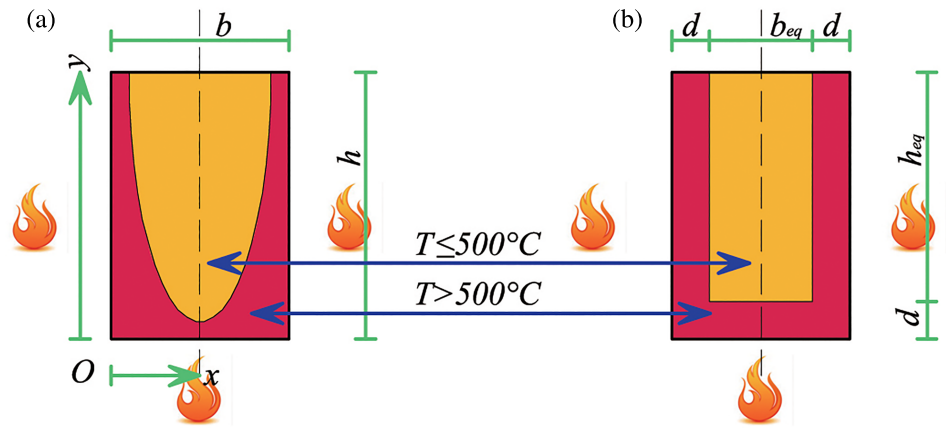


FIGURE 2 Comparison of ISO-834 standard fire, parametric fire and real fire curves⁴⁷

FIGURE 3 Diagram of 500°C isotherm method. (a) Real temperature field and (b) Equivalent temperature field



$$\begin{cases} b_{eq}h_{eq} = bh - 2 \int_0^h \int_0^{b/2} \Gamma(T - 500) dx dy \\ b_{eq} = b - 2d \\ h_{eq} = h - d \end{cases}, \quad (3)$$

where b_{eq} and h_{eq} are the equivalent width and height of the inner concrete cross-section, respectively. b and h are the width and height of the whole cross-section at room temperature (20°C), respectively. d is the equivalent thickness of the fire-damaged concrete zone in the cross-section. $\Gamma(\cdot)$ is a mathematical unit step function and defined as

$$\Gamma(T - 500) = \begin{cases} 1, & T > 500 \\ 0, & T \leq 500 \end{cases}. \quad (4)$$

Solving Equation 3, we can obtain the thickness of the fire-damaged concrete zone as

$$d = \left\{ 2h + b \pm \left[(2h + b)^2 - 16 \int_0^h \int_0^{b/2} \Gamma(T - 500) dx dy \right]^{1/2} \right\} / 4, \quad (5)$$

subjected to $d \in [0, \min\{b/2, h\}]$.

2.2 | Ordinary and prestressed steel bars

2.2.1 | Yield strength and modulus of elasticity

The yield strength and moduli of elasticity of ordinary and prestressed steel bars at varying temperatures are also given by Eurocode.⁴⁴ These data are derived from steady-state or transient-state fire tests, and could account for varying categories of steel bars (hot rolled or cold worked for ordinary steel bars, and cold worked or quenched and tempered for prestressed steel bars). Considering that these data are discrete and inconvenient to directly apply, curve fitting is conducted by using optimization software 1stOpt⁵¹ with quasi-Newton algorithm.⁵² The selection of curve-fitting algorithm is determined by the overall consideration of fitting accuracy and conciseness. Then, the yield strength and moduli of elasticity of ordinary and prestressed steel bars at elevated temperatures can be approximated, in the form of ratio of high temperature (T) to room temperature (20°C), by the following equations.

a. Ratio of yield strength of ordinary steel bars

$$\frac{f_{sy}(T)}{f_{sy,20^\circ\text{C}}} = 1.057 - 2.510 \times 10^{-3}T + 5.981 \times 10^{-10}T^3 + \frac{5.731 \times 10^{-3}T}{\ln(T)}. \quad (6)$$

b. Ratio of modulus of elasticity of ordinary steel bars

$$\frac{E_s(T)}{E_{s,20^\circ\text{C}}} = (1.027 + 6.105 \times 10^{-11} T^{3.791})^{-1}. \quad (7)$$

c. Ratio of yield strength of prestressed steel bars

$$\frac{f_{psy}(T)}{f_{psy,20^\circ\text{C}}} = \frac{f_{psy}(T)}{\beta f_{psk,20^\circ\text{C}}} = 1.015 - 1.981 \times 10^{-3} T + 9.559 \times 10^{-7} T^2 - 1.531 \times 10^{-2} e^{-T}. \quad (8)$$

d. Ratio of modulus of elasticity of prestressed steel bars

$$\frac{E_{ps}(T)}{E_{ps,20^\circ\text{C}}} = 0.972 + 1.420 \times 10^{-8} T^{2.5} - 9.866 \times 10^{-3} \frac{T}{\ln(T)} + \frac{1.844}{T}. \quad (9)$$

In these formulae, the subscript “20°C” represents design values at room temperature. T is the temperature at the center of bars. The parameter β is given as⁴⁴

$$\beta = \left(\frac{\epsilon_{ud} - f_{ps,0.1k,20^\circ\text{C}}/E_{ps,20^\circ\text{C}}}{\epsilon_{uk} - f_{ps,0.1k,20^\circ\text{C}}/E_{ps,20^\circ\text{C}}} \right) \times \frac{f_{psk,20^\circ\text{C}} - f_{ps,0.1k,20^\circ\text{C}}}{f_{psk,20^\circ\text{C}}} + \frac{f_{ps,0.1k,20^\circ\text{C}}}{f_{psk,20^\circ\text{C}}}, \quad (10)$$

where $\epsilon_{ud} = 0.02$ is a strain limit, ϵ_{uk} is the elongation at the maximum load, and $\epsilon_{uk} = \epsilon_{ud}/0.9$ is recommended by Eurocode.⁵³ $f_{psk,20^\circ\text{C}}$ is the specified value of the tensile strength of prestressed steel bars at room temperature, and $f_{ps,0.1k,20^\circ\text{C}}$ is the 0.1% proof stress (equal to the yield strength $f_{psy,20^\circ\text{C}}$ herein). In the cross-sectional design, $f_{psk,20^\circ\text{C}} \equiv f_{ps,0.1k,20^\circ\text{C}}$ and then $\beta f_{psk,20^\circ\text{C}} = f_{psy,20^\circ\text{C}}$ could be assumed temporarily herein according to Eurocode.⁵³

The comparisons between the recommended values in Eurocode⁴⁴ (points) and the formulae-based fitting values (curves) are shown in Figures 4 and 5, which reveal that the empirical formulae are sufficiently accurate.

2.2.2 | Prestress loss of prestressed steel bars

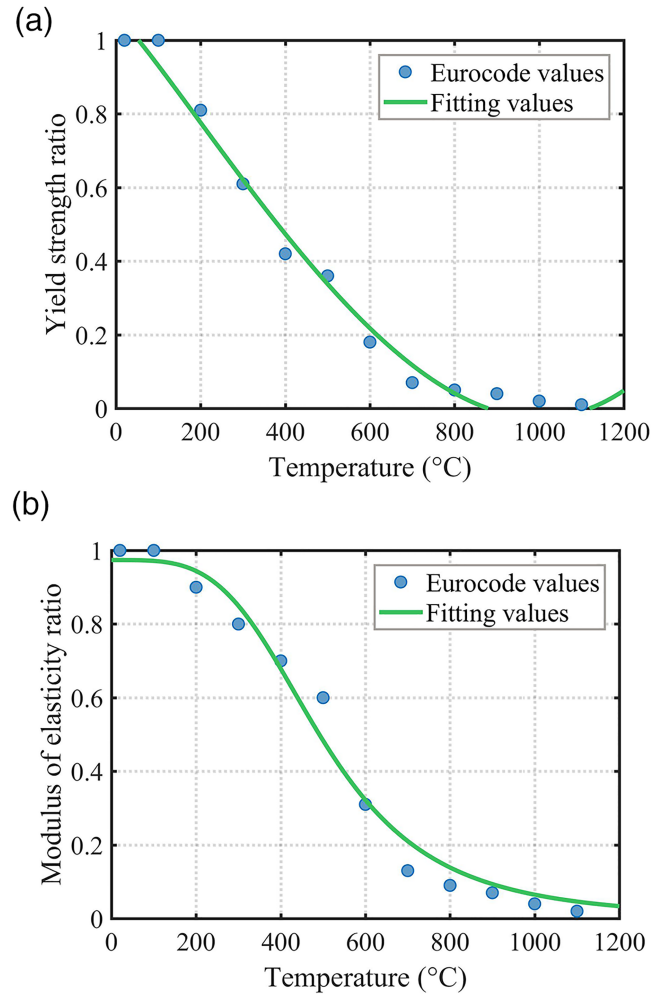
Prestress loss plays a vital role in the residual performance of prestressed RC beams. A considerable reduction of prestressing force will be induced by two effects at elevated temperatures: (a) a gradual recoverable reduction of prestressing force induced by the restrained thermal expansion and (b) an unrecoverable reduction caused by the creep and relaxation of steel. Gales et al.¹⁵ presented a computable model to predict the transient high-temperature relaxation, whereas a complex iterative program is involved. Based on the experimental data from relaxation tests, Eurocode⁵³ has defined a concise parameter, referred as the equivalent time t_{eq} , to represent the prestress loss at increasing temperatures while curing the concrete beam. The equivalent time t_{eq} can cater for the effects of heat treatment on the prestress loss due to the relaxation of the prestressed bars, which is given as⁵³

$$t_{eq} = \frac{1.14 T_{\max} - 20}{T_{\max} - 20} \sum_{i=1}^n (T_{\Delta t_i} - 20) \Delta t_i, \quad (11)$$

where $T_{\Delta t_i}$ is the temperature (in °C) of prestressed steel bars during the time interval Δt_i , and $T_{\max}$ is the maximum temperature (in °C) during the heat treatment. The equivalent time t_{eq} (in h) should be added to the time after tensioning t_t (in h). As for hot rolled and processed bars, the relaxation loss can be calculated as⁵³

$$\frac{\Delta \sigma_{ps}}{\sigma_{psi}} = 1.98 \times 10^{-5} \rho_{1000} \exp(8\mu) \times \left(\frac{t_t + t_{eq}}{1000} \right)^{0.75(1-\mu)}, \quad (12)$$

FIGURE 4 Comparisons between the recommended values (points) and the formulae-based fitting values (curves) of yield strength and modulus of elasticity for ordinary steel bars. (a) Yield strength ratios and (b) Modulus of elasticity ratios



where $\Delta\sigma_{ps}$ is the absolute value of the relaxation loss of the prestress, σ_{psi} is the absolute value of the initial prestress, $\mu = \sigma_{psi}/f_{psk,20^\circ\text{C}}$ is the stress ratio, and $\rho_{1000} = 4\%$ is the relaxation loss at 1,000 h after tensioning.

2.3 | SMA bars

2.3.1 | Yield strength and modulus of elasticity

Both temperature and stress can motivate the phase transition of SMA. By dividing the content of martensite into stress-induced martensite and temperature-induced martensite, Brinson's model³⁶ has well illustrated the relationship among the strain, stress, and temperature of SMA. However, this model is too sophisticated to calculate the properties of SMA in the hybrid beam. Alternatively, Sadiq et al.^{37,38} have given the experimental data of yield strength and modulus of elasticity of Ni-Ti SMA in the fire test. In this steady-state high-temperature tensile test, Ni 54.8 wt% Ti balance wt% wires with 0.95-mm nominal diameter are tested. SMA wires are firstly nonannealed with 40% cold drawn and then heat treated at 350°C for 0.5 h and quenched into ice water to obtain the optimal shape memory effect. Moreover, an environmental chamber is used to control SMA's temperature, and a noncontact laser extensometer is employed to measure SMA's strain. Based on these data given by Sadiq et al.,^{37,38} the yield strength and modulus of elasticity of prestressed SMA bars can be approximated, in the form of ratio of high temperature (T) to room temperature (20°C), by the following fitting formulae.

a. Ratio of yield strength

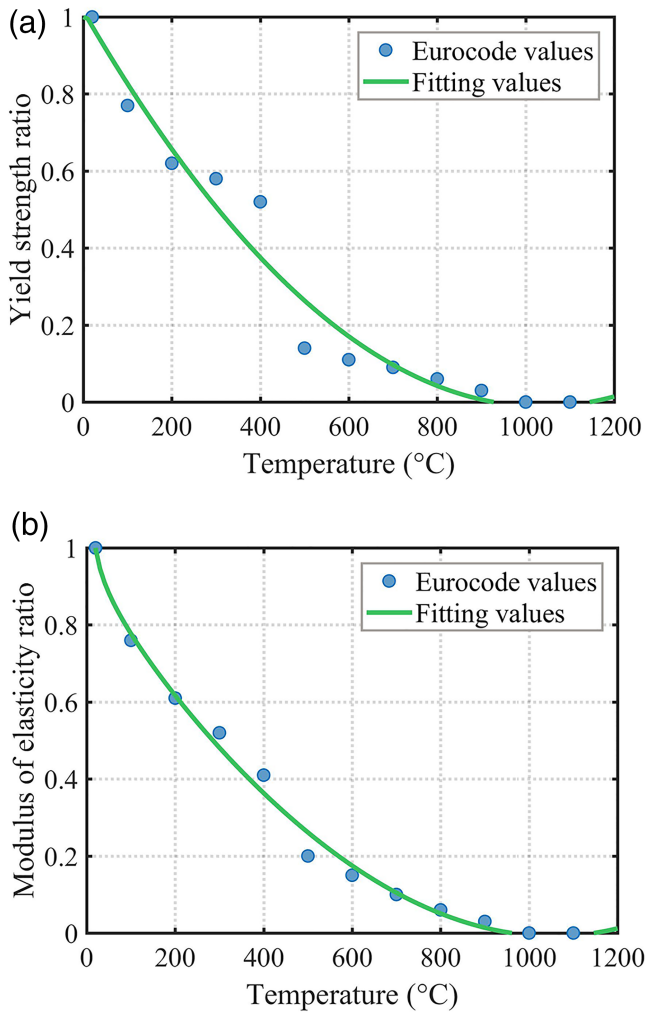


FIGURE 5 Comparisons between the recommended values (points) and the formulae-based fitting values (curves) of yield strength and modulus of elasticity for prestressed steel bars. (a) Yield strength ratios and (b) Modulus of elasticity ratios

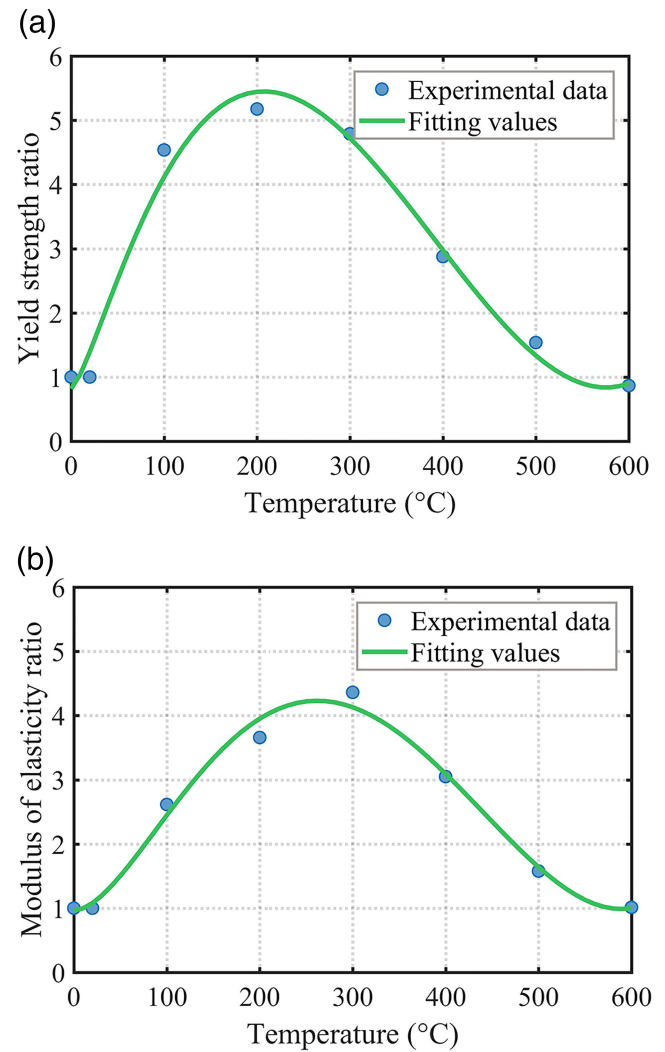
$$\frac{f_{pay}(T)}{f_{pay,20^{\circ}\text{C}}} = 0.817 + 9.696 \times 10^{-3} T^{1.5} - 8.081 \times 10^{-4} T^2 + 1.684 \times 10^{-5} T^{2.5}. \quad (13)$$

b. Ratio of modulus of elasticity

$$\frac{E_{pa}(T)}{E_{pa,20^{\circ}\text{C}}} = 0.965 + 4.289 \times 10^{-4} T^2 - 3.535 \times 10^{-5} T^{2.5} + 7.285 \times 10^{-7} T^3. \quad (14)$$

The comparisons between the experimental data³⁸ (points) and the formulae-based fitting values (curves) are shown in Figure 6, which reveal that the presented empirical formulae are sufficiently accurate. It should be noted that, the analyses in the next sections are based on an assumption that all the SMAs with varying diameters have the same material properties. DesRoches et al.,^{54,55} Soul et al.,⁵⁶ and other scholars proposed that there is a material difference between thick bars and thin bars because of the different metallurgical preparation (thin bars are usually hot rolled and thick bars are cold worked). Considering that the difference between SMA material properties (in terms of the initial modulus of elasticity, and loading and unloading plateau stresses) of thick and thin bars is less than about 10%,⁵⁵ it is reasonable to make the above-mentioned assumption. As for the practical engineering, it is convenient to use only SMA's diameters near to what has been tested.

FIGURE 6 Comparisons between the experimental data³⁸ (points) and the formulae-based fitting values (curves) of yield strength and modulus of elasticity for prestressed SMA bars. (a) Yield strength ratios and (b) Modulus of elasticity ratios



2.3.2 | Recovery stress

Owing to the existence of shape memory effect, a considerable recovery stress can be generated in SMA bars at elevated temperatures if both ends of SMA are constrained.¹⁸ In the research conducted by Sadiq et al.,³⁸ the recovery stress is generated from a constrained SMA with an initial residual strain equating to 4%. With formula fitting, the recovery stress of SMA $\Delta\sigma_{pa}$ (in MPa) can be calculated by employing Equation 15, and the satisfactory accuracy of the fitted formula is shown in Figure 7. To compare the changes of stresses of prestressed SMA and steel bars, the prestress loss of steel bars $\Delta\sigma_{ps}$ is also shown in Figure 7, which is based on Equation 12 and the cross-sectional properties given in Section 5.1. It can be observed that, when the temperature of bars is less than 350°C, the recovery stress provided by SMA bars $\Delta\sigma_{pa}$ could compensate the prestress relaxation of steel bars $\Delta\sigma_{ps}$.

$$\Delta\sigma_{pa} = 588.4 \exp \left[- \left(\frac{T - 306.4}{89.21} \right)^2 \right] + 567.9 \exp \left[- \left(\frac{T - 158.9}{88.51} \right)^2 \right]. \quad (15)$$

Comprehensively, the material properties of concrete at elevated temperatures could be obtained by Kodur's model⁴¹ and 500°C isotherm method.⁴⁴ And the properties of steel and SMA bars can be derived by employing the above fitting formulae after calculating the temperature of the center of bars via Kodur's model.⁴¹ Then, the residual properties of concrete cross-section, the prestress loss of prestressed steel bars, and the recovery stress of prestressed SMA bars can be obtained conveniently.

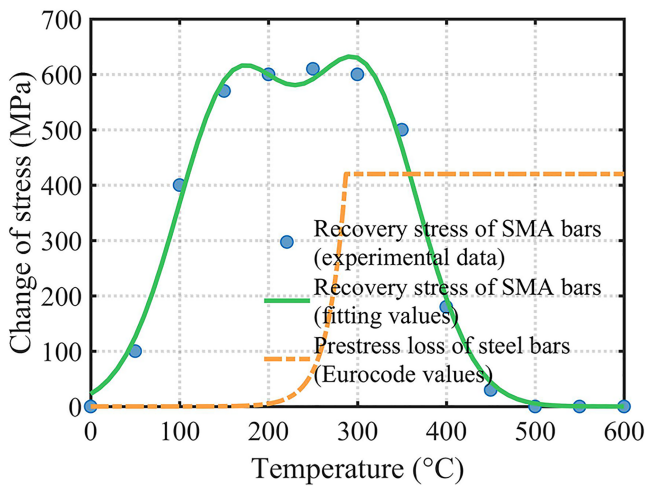


FIGURE 7 Changes of stresses of prestressed SMA bars (points are the experimental data³⁸ and the solid curve represents the formula-based fitting values) and steel bars (dashed curve)

3 | OUTLINE OF HYBRID STEEL-SMA PRESTRESSED BEAM

3.1 | Analytical model

In order to conduct the theoretical analysis of the proposed hybrid steel-SMA prestressed beam, an idealized analytical model is employed herein, as shown in Figure 8. The geometrical size of beam's cross-section is $b \times h = 300 \text{ mm} \times 500 \text{ mm}$, and the length of clear span is $l = 9,000 \text{ mm}$. A'_s and A'_{ps} represent areas of ordinary and prestressed steel bars in the upper zone, respectively. A_s , A_{ps} , and A_{pa} are the areas of ordinary steel bars, prestressed steel bars, and prestressed SMA bars in the lower zone, respectively. The number of each type of bars is $n = 2$, and $d'_s = d'_{ps} = 22 \text{ mm}$ and $d_s = d_{ps} = d_{pa} = 28 \text{ mm}$ are default values of diameters of bars. The default concrete covers (herein defined as the vertical distances from the centers of bars to the top or bottom edge of cross-section) of the ordinary and prestressed bars are $a_s = a'_s = 50 \text{ mm}$ and $a_{ps} = a'_{ps} = a_{pa} = 100 \text{ mm}$, respectively. The sustained load is assumed to be a uniform force of $q \equiv 40 \text{ N/mm}$, and three surfaces of the beam are exposed to the fire scenario described by Equation 2.

3.2 | Working mechanism

As for an ordinary prestressed beam at room temperature (shown in the first line of Figure 9), its deformation caused by the external load can be effectively reduced by the inverted arch effect beneficial from the prestress of the prestressed steel bars, thus presenting an acceptable limited deformation. However, when the ordinary prestressed beam is exposed to fire (the second line of Figure 9), the material properties of concrete and steel bars degrade seriously, thus inducing an obvious increase of deformation and a serious reduction of flexural bearing capacity. What makes it worse is that a significant prestress loss of prestressed steel bars will be induced by elevated temperatures and cause a considerable reduction of the inverted arch effect, which further increases the deformation of the beam.

If prestressed SMA bars are introduced to partially replace the prestressed steel bars (the third line of Figure 9), not only could they perform as an alternative of the prestressed steel bars at room temperature, but also improve the performance of the beam when exposed to fire. As temperature increases, both the yield strength and modulus of elasticity of SMA bars increase by a few times of that at room temperature, thus increasing the flexural bearing capacity and

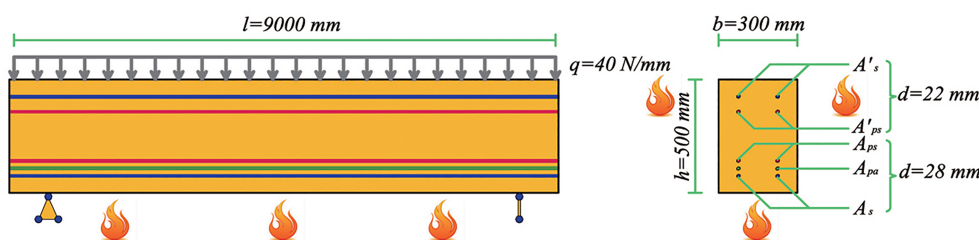


FIGURE 8 Analytical model of steel-SMA prestressed beam exposed to fire under the uniform load

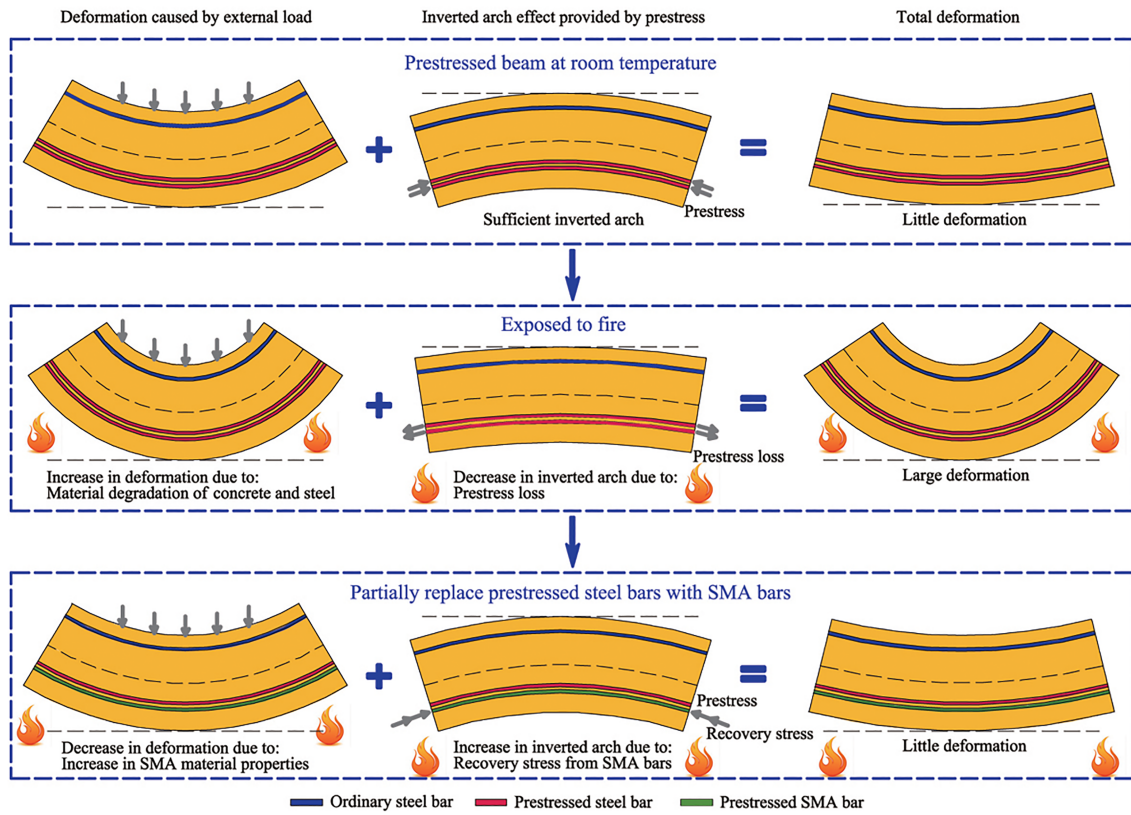


FIGURE 9 Working mechanism of steel-SMA prestressed beam exposed to fire

decreasing the beam's deformation. Furthermore, a desirable recovery stress can be generated in SMA bars and further enhance the inverted arch effect. Hence, the fire-induced excessive deflection could be suppressed by prestressed SMA bars. Regarding the powerful flexural bearing capacity compensation and deflection suppression effects of prestressed SMA bars, the improvement of fire resistance can be realized by the proposed prestressed beam. Herein, it should be noted that no maintenance of flexural bearing capacity or stiffness should be expected when the temperature of SMA bars exceeds 400°C , because both enhancement effects of yield strength and modulus of elasticity of SMA bars, and the recovery stress are negligible at this point (as shown in Figures 6 and 7).

4 | THEORETICAL ANALYSIS FOR FIRE PERFORMANCE

4.1 | Methodology of the prediction for fire resistance

A flowchart is presented in Figure 10 to illustrate the methodology for predicting the fire resistance of the hybrid steel-SMA prestressed beam. To determine the fire resistance (i.e., the time when the beam fails to resist the sustained load), the fire exposure process is divided into a series of steps. In any given step, the axial force N_p (the resultant force of steel and SMA bars after the prestressing construction) and its eccentric distance e can be determined with material properties at elevated temperatures, which are given in Section 2. Then, the flexural bearing capacity $[M] = M_u$ and the deflection f of the hybrid prestressed beam can also be calculated.

For the hybrid prestressed beam exposed to fire, there are two possible failure modes: insufficient bearing capacity or excessive deflection. When either of the two conditions is reached, the beam yields its fire resistance:

$$t_{res} = t_{i-1}, \text{ subjected to } M = [M] \text{ or } f = [f], \quad (16)$$

where M is the moment of cross-section caused by the external sustained load q and $[f]$ is the deflection limit of beam defined by the design code.⁵⁷

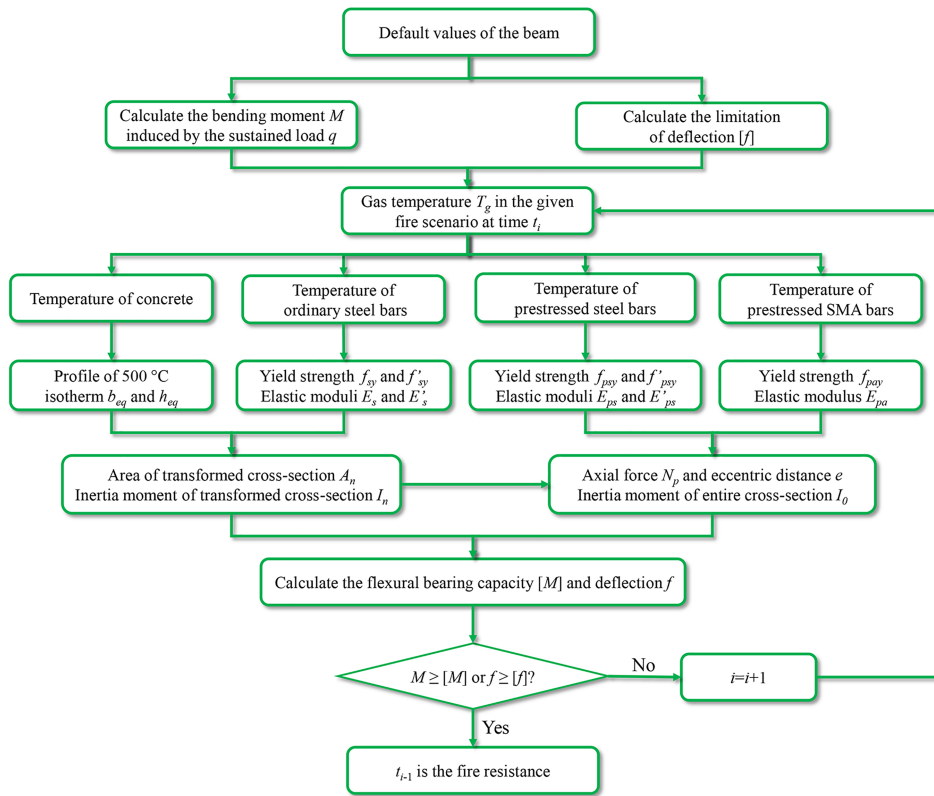


FIGURE 10 Flowchart to predict the fire resistance of the hybrid steel-SMA prestressed beam

4.2 | Effective prestress of the concrete

The effective prestress of concrete cross-section in the hybrid steel-SMA prestressed beam, represents the cross-sectional stress after the prestressing construction and before loading.⁵⁷ The effective prestress directly determines the flexural bearing capacity $[M]$ and the deflection f , so it should be computed in the first step. As shown in Figure 11, the internal forces of reaction exerted on the cross-section are balanced by the axial resultant force N_p with an eccentric distance e .

The effective prestresses of the concrete at the lower and upper edges of the cross-section (σ_{pc} and σ'_{pc} , respectively) can be calculated as

$$\begin{cases} \sigma_{pc} = \frac{N_p}{A_n} + \frac{N_p e h}{I_n} \\ \sigma'_{pc} = \frac{N_p}{A_n} - \frac{N_p e h}{I_n} \\ N_p = \sigma_{ps} A_{ps} + \sigma'_{ps} A'_{ps} + \sigma_{pa} A_{pa} - \sigma_s A_s - \sigma'_s A'_s \\ N_p e = \sigma_{ps} A_{ps} y_{ps} - \sigma'_{ps} A'_{ps} y'_{ps} + \sigma_{pa} A_{pa} y_{pa} - \sigma_s A_s y_s + \sigma'_s A'_s y'_s \end{cases}, \quad (17)$$

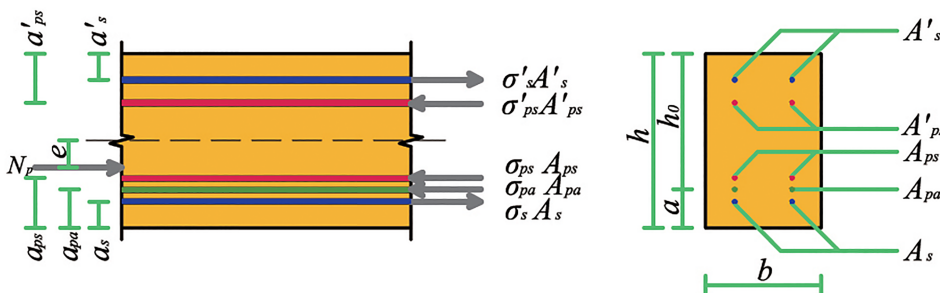


FIGURE 11 Diagram of the concrete cross-section of the hybrid prestressed beam

where A_n and I_n are the area and moment of inertia of the transformed cross-section (only considering concrete and ordinary steel bars), respectively, which are given as⁵⁷

$$\begin{cases} A_n = A_c + n\pi(\alpha_s d_s^2 + \alpha'_s d_s'^2)/4 \\ I_n = I_c + n\pi[(\alpha_s^2 d_s^4 + \alpha_s'^2 d_s'^4)/64 + (\alpha_s d_s^2 y_s^2 + \alpha'_s d_s'^2 y_s'^2)/4] \\ A_c = b_{eq} h_{eq} \\ I_c = b_{eq} h_{eq}^3/12 + A_c(h - h_{eq})^2/4 \end{cases}, \quad (18)$$

where A_c and I_c are, respectively, the area and moment of inertia of the net cross-section (only considering concrete). $\alpha_s = E_s/E_c$ and $\alpha'_s = E'_s/E_c$ are the ratios of elastic moduli of ordinary steel bars to concrete. The distances from the centers of bars to the neutral axis of cross-section (assumed to be at the height of $h/2$) are expressed as

$$\begin{cases} y_s = h/2 - a_s, & y'_s = h/2 - a'_s \\ y_{ps} = h/2 - a_{ps}, & y'_{ps} = h/2 - a'_{ps} \\ y_{pa} = h/2 - a_{as} \end{cases}. \quad (19)$$

In addition, the effective prestresses of bars can be calculated as

$$\begin{cases} \sigma_s = \alpha_s \sigma_{pc} + \sigma_{lc}, & \sigma'_s = \alpha'_s \sigma'_{pc} + \sigma'_{lc} \\ \sigma_{ps} = \sigma_{con,s} - \sigma_{ls}, & \sigma'_{ps} = \sigma'_{con,s} - \sigma'_{ls} \\ \sigma_{pa} = \sigma_{con,a} - \sigma_{la} \end{cases}, \quad (20)$$

where $\sigma_{con,s}$, $\sigma'_{con,s}$, and $\sigma_{con,a}$ are, respectively, initial tension stresses in the prestressing construction. σ_{ls} , σ'_{ls} , and σ_{la} are the total prestress losses at room temperature, respectively. The respective prestress losses due to shrinkage and creep of concrete are given as σ_{lc} and σ'_{lc} . These prestress losses can be calculated precisely according to Eurocode⁵³ or Chinese code,⁵⁷ whereas they are not the main research objects in this paper. And a set of reasonable values, $\sigma_{con,s} = \sigma_{con,a} = 500$ MPa, $\sigma'_{con,s} = 300$ MPa, $\sigma_{ls} = \sigma'_{ls} = \sigma_{la} = 80$ MPa, and $\sigma_{lc} = \sigma'_{lc} = 20$ MPa, are chosen herein for the following analyses.

By solving Equations 17 to 20, we can obtain N_p and e as

$$\begin{cases} N_p = \frac{A_n \{ [2I_n + h(\alpha_s A_s y_s + \alpha'_s A'_s y'_s)] N - h(\alpha_s A_s - \alpha'_s A'_s) S \}}{(A_n + \alpha_s A_s + \alpha'_s A'_s) [2I_n + h(\alpha_s A_s y_s + \alpha'_s A'_s y'_s)] - h(\alpha_s A_s - \alpha'_s A'_s) (\alpha_s A_s y_s - \alpha'_s A'_s y'_s)}, \\ e = \frac{2I_n [(A_n + \alpha_s A_s + \alpha'_s A'_s) S - (\alpha_s A_s y_s - \alpha'_s A'_s y'_s) N]}{N_p (A_n + \alpha_s A_s + \alpha'_s A'_s) [2I_n + h(\alpha_s A_s y_s + \alpha'_s A'_s y'_s)] - h(\alpha_s A_s - \alpha'_s A'_s) (\alpha_s A_s y_s - \alpha'_s A'_s y'_s)} \end{cases}, \quad (21)$$

where N and S are, respectively, the load and moment parameters and expressed as

$$\begin{cases} N = \sigma_{ps} A_{ps} + \sigma'_{ps} A'_{ps} + \sigma_{pa} A_{pa} - \sigma_{lc} A_s - \sigma'_{lc} A'_s \\ S = \sigma_{ps} A_{ps} y_{ps} - \sigma'_{ps} A'_{ps} y'_{ps} + \sigma_{pa} A_{pa} y_{pa} - \sigma_{lc} A_s y_s + \sigma'_{lc} A'_s y'_s \end{cases}. \quad (22)$$

Finally, the prestresses of concrete (σ_{pc} and σ'_{pc}) can be obtained by substituting the axial resultant force N_p and its eccentric distance e in Equation 21 into 17.

4.3 | Flexural bearing capacity

When the stress of concrete at the lower edge of cross-section is equal to zero, the stresses of the prestressed bars can be expressed as

$$\sigma_{p0s} = \sigma_{ps} + \alpha_{ps}\sigma_{pc}, \quad \sigma'_{p0s} = \sigma'_{ps} + \alpha'_{ps}\sigma_{pc}, \quad \sigma_{p0a} = \sigma_{pa} + \alpha_{pa}\sigma_{pc}, \quad (23)$$

where α_{ps} , α'_{ps} , and α_{pa} are the modulus of elasticity ratios of prestressed bars to concrete and given as

$$\alpha_{ps} = \frac{E_{ps}}{E_c}, \quad \alpha'_{ps} = \frac{E'_{ps}}{E_c}, \quad \alpha_{pa} = \frac{E_{pa}}{E_c}. \quad (24)$$

Moreover, a and a' are, respectively, the distances from the centers of gravity of bars in the lower and upper zones to the corresponding edges of the cross-section, which can be expressed as

$$a = \frac{a_s A_s + a_{ps} A_{ps} + a_{pa} A_{pa}}{A_s + A_{ps} + A_{pa}}, \quad a' = \frac{a'_s A'_s + a'_{ps} A'_{ps}}{A'_s + A'_{ps}}. \quad (25)$$

Then, the effective depth of compression zone can be obtained as $h_0 = h - a$.

As shown in Figure 12, the relative depth of the compression zone ξ_b can be calculated as⁵⁷

$$\xi_b = \min \left\{ \beta_1 \left(1 + \frac{f_{psy} - \sigma_{p0s}}{E_{ps} \varepsilon_{cu}} \right)^{-1}, \beta_1 \left(1 + \frac{f_{pay} - \sigma_{p0a}}{E_{pa} \varepsilon_{cu}} \right)^{-1} \right\}, \quad (26)$$

where $\beta_1 = 0.8$ is the ratio of heights of cross-sectional equivalent rectangular stress distribution of concrete to the real stress distribution.

Next, as shown in Figure 13a, the calculation formulae of flexural bearing capacity are given as⁵⁷

$$\begin{cases} M_u = \alpha_1 f_c b_{eq} x \left(h_0 - \frac{x}{2} \right) + f'_{sy} A'_s (h_0 - a'_s) - (\sigma'_{p0s} - f'_{psy}) A'_{ps} (h_0 - a'_{ps}) \\ \alpha_1 f_c b_{eq} x = f_{sy} A_s - f'_{sy} A'_s + f_{psy} A_{ps} + f_{pay} A_{pa} + (\sigma'_{p0s} - f'_{psy}) A'_{ps} \end{cases}, \quad (27)$$

subjected to $\begin{cases} 2a' \leq x \leq \xi_b h_0 \\ \sigma'_{p0s} \leq f'_{psy} \end{cases}$

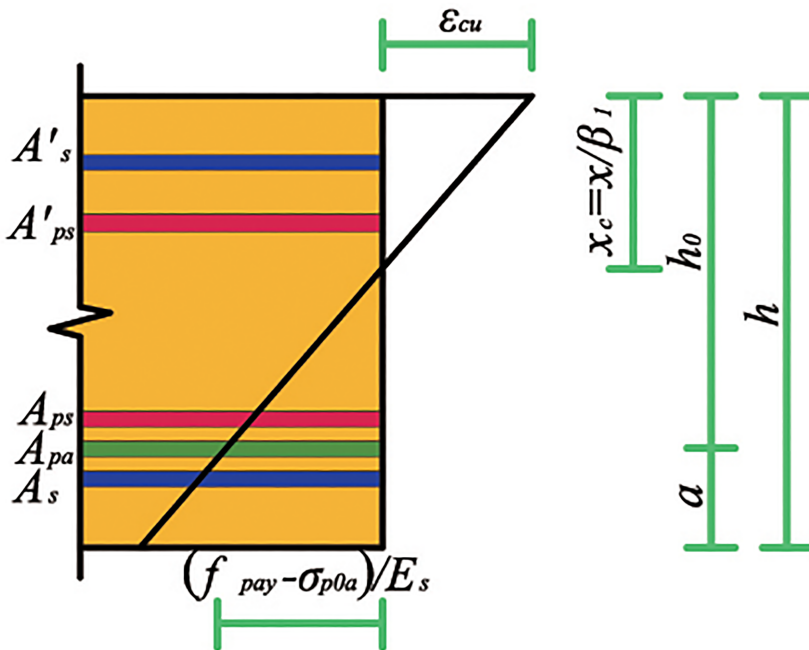


FIGURE 12 Effective depth of compression zone of concrete cross-section

selected as the study object, the other parameters remain unchanged. The default values of material parameters^{38,57} are shown in Table 1. It is worth to mention that design values are the same as the characteristic values in the structural fire design.⁴⁴

5.1.1 | Area of prestressed SMA bars

The flexural bearing capacity–time (i.e., $[M] - t$, based on Equation 27 or 28) and deflection–time (i.e., $f - t$, based on Equation 32) curves for varying diameters (in other words, areas) of the SMA bars, namely, $d = 10, 20$, and 30 mm, are shown in Figures 14 and 15, respectively. The horizontal line in Figure 14 represents the moment of cross-section M caused by the external sustained load, while that in Figure 15 is the deflection limit of beam $[f]$ defined by the design code.⁵⁷ Thus, the intersection points between the curves and the horizontal lines indicate the fire resistance.

It can be observed that with the increasing diameter of prestressed SMA bars, the flexural bearing capacity of beam increases and the deflection decreases evidently, indicating satisfactory flexural stiffness and prestress compensation effects can be realized by SMA bars, which is in accordance with the previous analysis. The results also reveal that the fire resistance of prestressed beam can be enhanced by employing steel and SMA bars simultaneously.

5.1.2 | Location of prestressed SMA bars

The performance of the hybrid steel–SMA prestressed beam is also affected by the location of prestressed SMA bars. Six concrete covers (in other words, the vertical distance from the centers of bars to the bottom surface of the beam), namely, $a = 50, 75, 100, 125, 150$, and 175 mm, are taken into account. These parameters are selected by considering the convenience of construction, and it has to be assured that the SMA bars remain in the relatively low temperature zone they need, as the SMA can no longer bear considerable loads over 300°C to 400°C . Regarding prestressed SMA bars, on one hand, the larger concrete cover represents insufficient moment lever-arm and flexural bearing capacity (as shown in Figure 16). On the other hand, the larger concrete cover makes temperature of SMA bars rise more slowly; thus, the

TABLE 1 Default values of material parameters of the hybrid steel–SMA prestressed beam

Categories	Properties	Values or types
Concrete ⁵⁷	Material type	Normal strength and carbonate concrete
	Compressive strength	$f_c = 19.1$ MPa
	Tensile strength	$f_t = 1.71$ MPa
	Modulus of elasticity	$E_c = 3.25 \times 10^4$ MPa
	Ultimate compressive strain	$\epsilon_{cu} = 0.0035$
Ordinary steel bars ⁵⁷	Material type	Hot rolled steel bars
	Yield strength	$f_{sy,20^{\circ}\text{C}} = 400$ MPa
	Tensile strength	$f_{sk,20^{\circ}\text{C}} = 540$ MPa
	Modulus of elasticity	$E_{s,20^{\circ}\text{C}} = 206 \times 10^3$ MPa
Prestressed steel bars ⁵⁷	Material type	Quenched and tempered steel bars
	Yield strength	$f_{psy,20^{\circ}\text{C}} = 785$ MPa
	Tensile strength	$f_{psk,20^{\circ}\text{C}} = 980$ MPa
	Modulus of elasticity	$E_{ps,20^{\circ}\text{C}} = 206 \times 10^3$ MPa
Prestressed SMA bars ³⁸	Material type	Ni–Ti SMA bars
	Yield strength	$f_{pay,20^{\circ}\text{C}} = 171.9$ MPa
	Tensile strength	$f_{pak,20^{\circ}\text{C}} = 1451.2$ MPa
	Modulus of elasticity	$E_{pa,20^{\circ}\text{C}} = 19.33 \times 10^3$ MPa
	Transformation temperatures	$M_f = -36.2^{\circ}\text{C}$ $M_s = 22.8^{\circ}\text{C}$
		$A_s = 52.7^{\circ}\text{C}$ $A_f = 78.5^{\circ}\text{C}$

FIGURE 14 Flexural bearing capacity–time curves for varying diameters of SMA bars

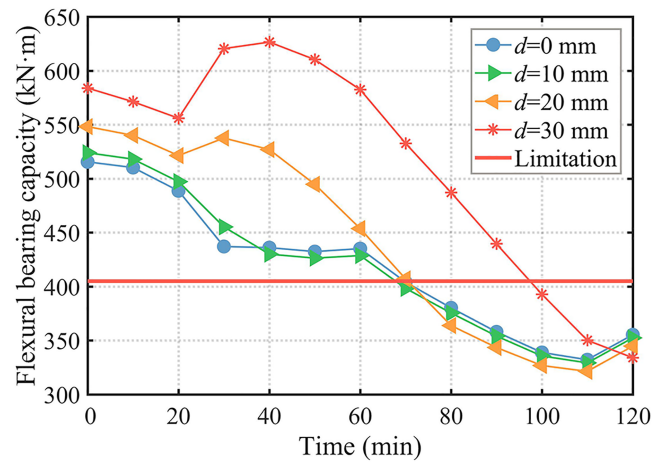


FIGURE 15 Deflection–time curves for varying diameters of SMA bars

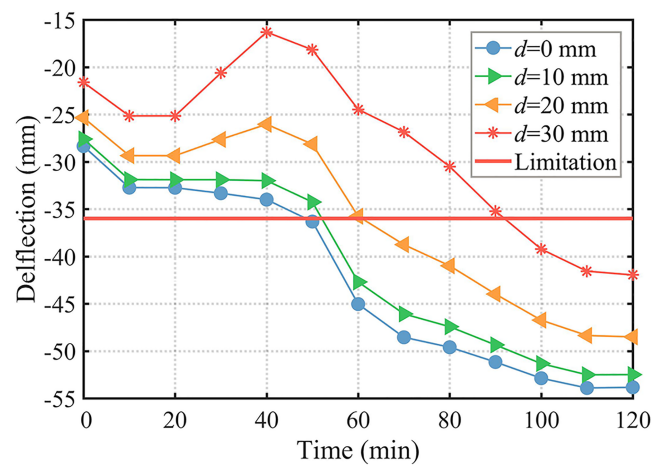
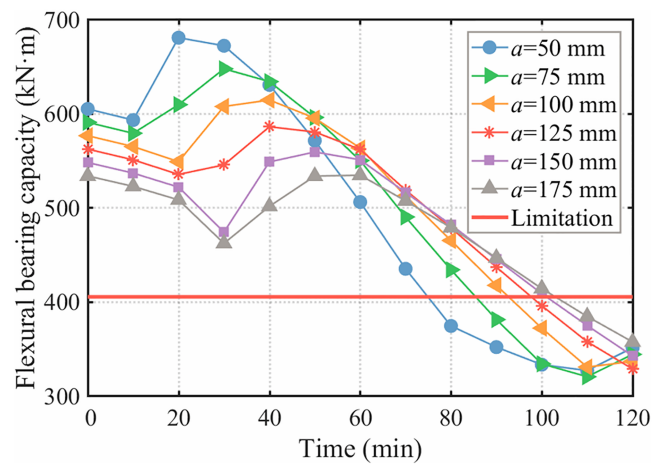


FIGURE 16 Flexural bearing capacity–time curves for varying locations of SMA bars



shape memory effect works later. Specially, when the concrete cover exceeds 150 mm, the contribution to flexural stiffness is negligible (as shown in Figure 17). From the view of fire resistance, the proposed beam fails due to insufficient flexural bearing capacity with the less concrete cover (Figure 16), and the excessive deflection is the main failure reason for the larger concrete cover (Figure 17). Thus, an optimal location of SMA bars exists.

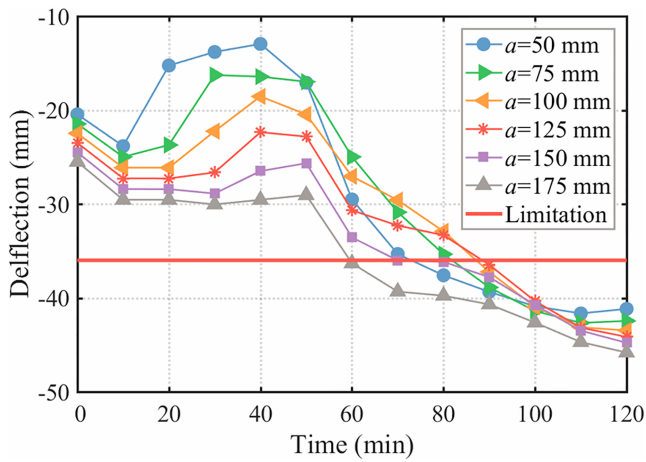


FIGURE 17 Deflection–time curves for varying locations of SMA bars

5.2 | Optimal analysis

5.2.1 | Optimal algorithm

As presented in the previous section, the fire resistance of the hybrid prestressed beam is controlled by the diameter d_{pa} (or area A_{pa}) and the location a_{pa} of prestressed SMA bars. For the sake of expensive material cost of SMA bars, it is desirable to maintain a minimum usage of SMA bars within a certain fire resistance (such as $[t_{res}] = 90 \text{ min}$ ⁵⁸). The locations of prestressed steel bars (a_{ps} and a'_{ps}) are also taken into consideration in the optimal analysis, and the range of a_{pa} , a_{ps} , and a'_{ps} are the same as a_{pa} in the parametric study (i.e., $[50, 175] \text{ mm}$). Considering the convenience of construction, the maximum diameter is selected as $d_{\max} = 35 \text{ mm}$, and spacing limitation of bars $|a_{pa} - a_{ps}| \geq 2d_{\max}$ should also be satisfied.⁵⁷ Then, the optimal problem can be summarized as

$$\min \{d_{pa}\}, \text{ subjected to } \begin{cases} t_{res}(d_{pa}, a_{pa}, a_{ps}, a'_{ps}) \geq [t_{res}] \text{ min} \\ d_{pa} \leq d_{\max} \text{ mm} \\ |a_{pa} - a_{ps}| \geq 2d_{\max} \text{ mm} \\ a_{pa}, a_{ps}, a'_{ps} \in [50, 175] \text{ mm} \end{cases} \quad (33)$$

To solve this nonlinear and multivariate optimal problem, a well-known global optimal algorithm, genetic algorithm,⁵⁹ is employed herein. This stochastic global algorithm mimics the metaphor of natural biological evolution, and the principle of survival of the fittest is applied to produce successively better approximations to a solution. The algorithm parameter settings are shown in Table 2. Among these parameters, *StallGenLimit* represents the maximum times when the average relative change of objective value is less than or equal to a given error. *PopulationSize* is the size of the population, and the calculation accuracy increases and the calculation efficiency decreases with the increasing population size. *CrossoverFraction* is the fraction of the population at the next generation, and *MigrationFraction* specifies the fraction of individuals in each subpopulation that migrates to a different subpopulation. *SelectionFcn* represents the method that selects parents of crossover and mutation children, and *Roulette* method assumes that the probability of selection for each individual is in direct proportion to its fitness.⁵⁹

5.2.2 | Optimal performance

The optimal design values for the hybrid steel–SMA prestressed beam derived from the genetic algorithm are shown in Table 3. The cross-sectional reinforcement ratio 3.46% is still in a reasonable range. It should also be noted that,

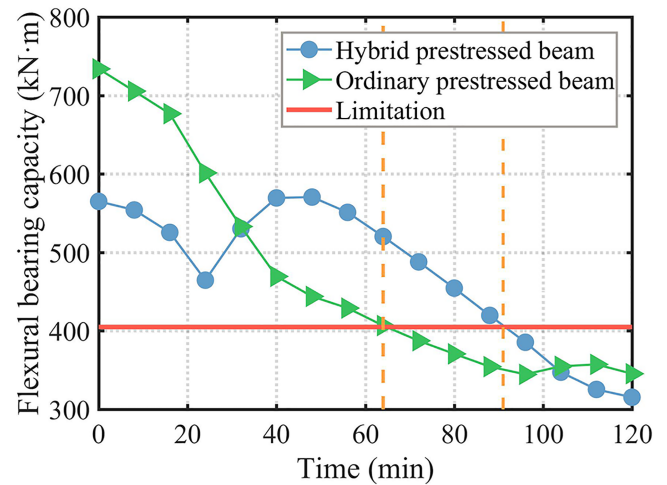
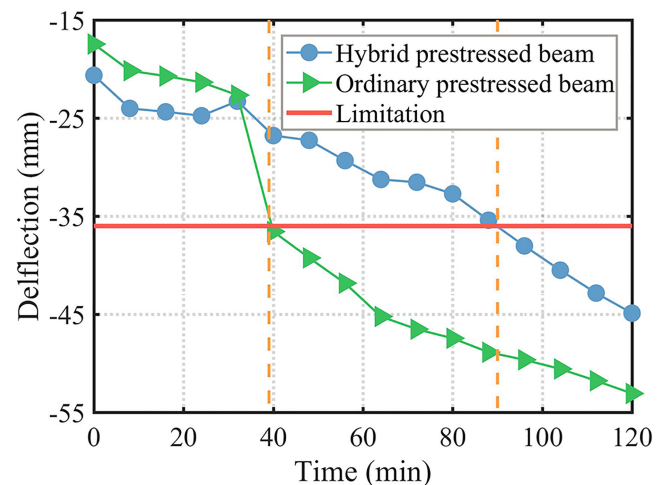
TABLE 2 Parameter settings for the genetic algorithm

<i>StallGenLimit</i>	<i>PopulationSize</i>	<i>CrossoverFraction</i>	<i>MigrationFraction</i>	<i>SelectionFcn</i>
15	100	0.8	0.2	Roulette

TABLE 3 Optimal design values for the hybrid steel–SMA prestressed beam

d_{pa} (mm)	a_{ps} (mm)	a_{pa} (mm)	a'_{ps} (mm)
27.7	51.3	121.9	175.0

although the optimization process arrives at a 27.7-mm diameter bar, the equivalent cross-section needed for the SMA part could be obtained by some smaller diameter bars in parallel. The performance of the hybrid steel–SMA prestressed beam with optimal parameters when exposed to fire is shown in Figure 18 and 19. In order to demonstrate the superior performance of the hybrid prestressed beam, the ordinary prestressed beam is also utilized as a comparison (replacing all prestressed SMA bars by extra prestressed steel bars with the same area). It is evident from the figures that (a) although the flexural bearing capacity and stiffness of the hybrid prestressed steel–SMA beam are lower than the ordinary prestressed beam at room temperature, the hybrid prestressed beam still maintains an acceptable performance. (b) With the increasing temperature, the performance of the ordinary prestressed beam degrades seriously after 30 min, whereas the hybrid prestressed beam still has a robust capacity to resist the sustained load. (c) Owing to the shape memory effect of SMA prestressed bars, an increase of fire resistance from 39 to 90 min is realized by the hybrid steel–SMA prestressed beam. Therefore, fire resistance enhancement can be realized by the hybrid usage of prestressed steel and SMA bars. However, it should be noted that the flexural bearing capacity and stiffness of the proposed beam might be reduced when the fire is extinguished and the gas temperature decreases after some time. Because in this case, SMA bars are cooling with the decreasing gas temperature, and its shape could be changed easily at that time, which results

**FIGURE 18** Flexural bearing capacity–time curves of hybrid and ordinary prestressed beams**FIGURE 19** Deflection–time curves of hybrid and ordinary prestressed beams

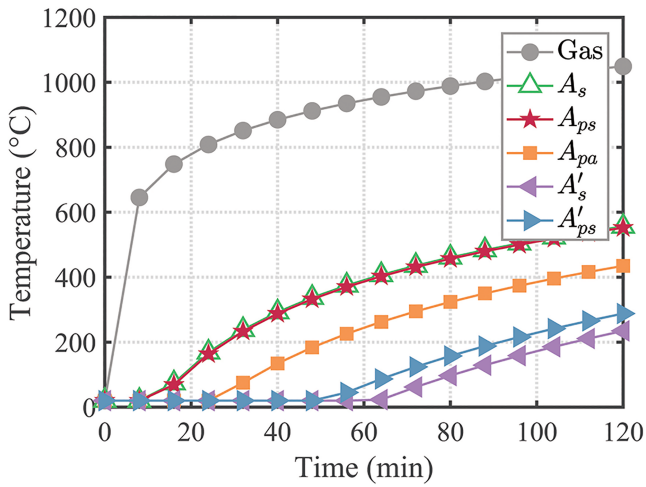


FIGURE 20 Temperature histories of bars

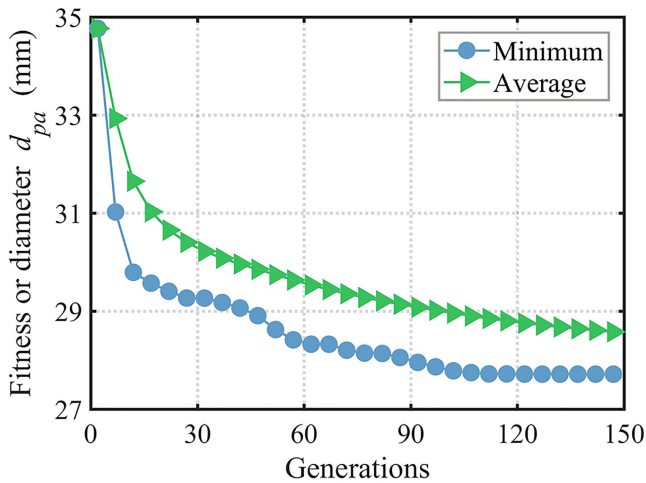


FIGURE 21 Fitness curves of the genetic algorithm

in the reduction of SMA's recovery stress. In addition, steel bars have already been damaged, and their yield strength and moduli of elasticity might be insufficient.

The temperature histories of bars are shown in Figure 20, which are seen to be quite different from each other for the bars located in different positions. Specially, the temperature of the prestressed SMA bars A_{pa} begins to rise after 25 min, and the flexural bearing capacity increases accordingly (as shown in Figure 18), indicating the satisfactory flexural bearing capacity compensation effect of SMA bars. In addition, the temperature of prestressed SMA bars A_{pa} reaches about 400°C when the time of fire exposure is 90 min. As said previously, no maintenance of flexural bearing capacity or stiffness should be expected at that time. Thus, it is reasonable to observe that the fire resistance of hybrid prestressed beam is 90 min in Figures 18 and 19. Moreover, the fitness curves (representing the current and applicable minimum diameter of SMA bars) of genetic algorithm are shown in Figure 21. After 120 generations, the fitness curves tend to be steady, revealing that the global optimal solution is found and the optimal process is reliable.

6 | CONCLUSIONS

This paper investigates the possibility of using prestressed steel and SMA bars to achieve compensations of flexural bearing capacity and flexural stiffness of prestressed concrete beams. The following conclusions can be drawn from this work.

1. Although the flexural bearing capacity and stiffness of the hybrid prestressed steel–SMA beam are lower than the ordinary prestressed beam at room temperature, the hybrid prestressed beam still maintains an acceptable performance.
2. By the action of compensation effect of SMA bars, the hybrid steel–SMA prestressed beam enhances the flexural bearing capacity to a sufficient level at elevated temperatures, indicating the validity for prestressed beams to meet the more restricted fire resistance requirements.
3. A satisfactory reduction of deflection can be realized by the hybrid steel–SMA prestressed beam owing to the compensation effect of flexural stiffness and the prestress provided by SMA bars. Thus, excessive structural retrofits for the beams after fire might be avoided.
4. The fire resistance can be improved significantly by the hybrid steel–SMA prestressed beam over the ordinary one. If the SMA bars were substituted with steel bars, the bearing capacity would be higher at room temperature, but the beam would fail in fire earlier compared to the hybrid beam. Thus, the hybrid beam has a considerable significance in evacuation and rescue operations in a building fire.
5. The hybrid steel–SMA prestressed beam is a promising structural element and can also be considered as a promising technology for structures and infrastructures, such as buildings and bridges.

DATA AVAILABILITY STATEMENT

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request (including mathematic models and Matlab codes for the calculation of the proposed beam's performance).

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CONFLICT OF INTEREST

The author(s) declare that there is no conflict of interest regarding the publication of this paper. There were no sources of funding that could have influenced the outcome of this work.

NOMENCLATURE

a, a'	the distances from the centers of gravity of bars in the lower and upper zones to the corresponding edges of the cross-section, respectively
a_s, a_{ps}, a_{pa}	the concrete covers of ordinary steel bars, prestressed steel bars, and prestressed SMA bars in the lower zone, respectively
a'_s, a'_{ps}	the concrete covers of ordinary and prestressed steel bars in the upper zone, respectively
A_c	the area of net cross-section
A_n	the area of transformed cross-section
A_s, A_{ps}, A_{pa}	the areas of ordinary steel bars, prestressed steel bars, and prestressed SMA bars in the lower zone, respectively
A'_s, A'_{ps}	the areas of ordinary and prestressed steel bars in the upper zone, respectively
b, h	the width and height of the whole concrete cross-section at 20°C, respectively
b_{eq}, h_{eq}	the equivalent width and height of the inner concrete cross-section, respectively
c_2	a material parameter and equal to 1.0 with respect to the normal strength and carbonate concrete
d	the equivalent thickness of the fire-damaged concrete zone in the cross-section
$d_{\max}$	the maximum diameter of the bars
d_s, d_{ps}, d_{pa}	the diameters of ordinary steel bars, prestressed steel bars, and prestressed SMA bars in the lower zone, respectively
d'_s, d'_{ps}	the diameters of ordinary and prestressed steel bars in the upper zone, respectively
e	the eccentric distance of the resultant force N_p
$E_{pa}(T)/E_{pa,20^\circ\text{C}}$	the ratio of modulus of elasticity of SMA bars at temperature T to 20°C
$E_{ps}(T)/E_{ps,20^\circ\text{C}}$	the ratio of modulus of elasticity of prestressed steel bars at temperature T to 20°C

$E_s(T)/E_{s,20^\circ\text{C}}$	the ratio of modulus of elasticity of ordinary steel bars at temperature T to 20°C
f	the mid-span deflection of the hybrid prestressed beam
$f_{pay}(T)/$	the ratio of yield strength of SMA bars at temperature T to 20°C
$f_{pay,20^\circ\text{C}}$	
$f_{ps,0.1k,20^\circ\text{C}}$	the 0.1% proof stress of prestressed steel bars at 20°C
$f_{psk,20^\circ\text{C}}$	the specified value of the tensile strength of prestressed steel bars at 20°C
$f_{psy}(T)/$	the ratio of yield strength of prestressed steel bars at temperature T to 20°C
$f_{psy,20^\circ\text{C}}$	
$f_{sy}(T)/f_{sy,20^\circ\text{C}}$	the ratio of yield strength of ordinary steel bars at temperature T to 20°C
f_{1l}	the mid-span deflection under a given uniformly distributed load
f_{2l}	the value of inverted arch of the simply supported prestressed beam resulting from the prestress
h_0	the effective depth of cross-section
I_c	the moment of inertia of net cross-section
I_n	the moment of inertia of transformed cross-section
I_0	the moment of inertia of the entire cross-section
l	the clear span of the concrete beam
M	the moment of cross-section induced by the external sustained load
n	the number of each type of bars
N_p	the resultant force of steel and SMA bars after the prestressing construction
q	an external and uniformly distributed load
t	the duration of fire exposure of the cross-section
t_{eq}	the equivalent time that could cater for the effects of heat treatment on the prestress loss
t_{res}	the fire resistance of the concrete beam
t_t	the time after tensioning
T	the temperature of a given point within the concrete cross-section
T_g	the gas temperature in a fire
$T_{\max}$	the maximum temperature during the heat treatment
$T_{\Delta t_i}$	the temperature during the time interval
x, y	shortest distances from a given point in the concrete cross-section to the fire exposure surface
y	the distance from the center of bars to the neutral axis of cross-section
$[f]$	the deflection limit of concrete beam defined by the design code
$[M] = M_u$	the flexural bearing capacity of the hybrid prestressed beam
$[t_{res}]$	a given fire resistance of the concrete beam
$\alpha_{ps}, \alpha'_{ps}, \alpha_{pa}$	the ratios of elastic moduli of prestressed bars to concrete, respectively
α_s, α'_s	the ratios of elastic moduli of ordinary steel bars to concrete, respectively
α_1	the ratio of maximum compressive stresses of cross-sectional equivalent rectangular stress distribution of concrete to the real stress distribution
β_1	the ratio of heights of cross-sectional equivalent rectangular stress distribution of concrete to the real stress distribution
$\Gamma(\quad)$	a mathematical unit step function
Δt_i	the length of the time interval
$\Delta\sigma_{pa}$	the recovery stress of SMA bars
$\Delta\sigma_{ps}$	the absolute value of the relaxation loss of prestress of prestressed steel bars
ε_{ud}	a strain limit of prestressed steel bars
ε_{uk}	the elongation of prestressed steel bars at the maximum load
η_x, η_y	the heat transfer factors induced through one fire-exposed surface
μ	the stress ratio of σ_{psi} to $f_{psk,20^\circ\text{C}}$
ξ_b	the relative depth of the compression zone
ρ_{1000}	the rate of relaxation loss at 1,000 h after tensioning
σ, σ'	the effective prestresses of bars in the lower and upper zones, respectively
$\sigma_{con}, \sigma'_{con}$	the initial tension stresses in the prestressing construction in the lower and upper zones, respectively
$\sigma_{lc}, \sigma'_{lc}$	the prestress losses due to the shrinkage and creep of concrete material in the lower and upper zones at room temperature, respectively

$\sigma_{ls}, \sigma'_{ls}, \sigma_{la}$	the total prestress losses of prestressed bars at room temperature, respectively
σ_{pc}	the effective prestress of the concrete at the lower edge of the cross-section
σ_{psi}	the absolute value of the initial prestress of prestressed steel bars
σ'_{pc}	the effective prestress of the concrete at the upper edge of the cross-section
$\sigma_{p0}, \sigma'_{p0}$	the stresses of prestressed bars when the stress of concrete at the lower edge is equal to zero, respectively

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