



## **Post-fire behaviour of high-strength steel welded I-section columns**

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### **Abstract**

High-strength steels have been increasingly used in long-span bridges and high-rise buildings in recent years, owing to their exceptional material strengths that enable the design of smaller member sizes and reduced self-weight. For the design of steel structures, fire safety is always a critical concern. In most scenarios, fire occurs only in parts of a building, and may be extinguished before it spreads to the entire building. Therefore, it is important that the extent of damage and residual load-carrying capacities of structural components after exposure to fire can be quantified through accurate evaluations, which enable an informed decision about whether a structure should be entirely dismantled or partially repaired or can be directly reused. To this end, this study is proposed. In this paper, a numerical modelling investigation was conducted to explore the structural behaviour and residual resistances of S690 high-strength steel welded I-section columns after exposure to elevated temperatures ranging from 30 °C to 950 °C. Finite-element models were first developed and validated by existing test results, and were then used to conduct a series of parametric studies to generate additional post-fire performance data. Based on the test and numerical data, the current European code and American specification were evaluated; the results indicate that the European code yields conservative post-fire resistance predictions while the American specification leads to many unsafe predictions. Then, a new design method was proposed, which was shown to provide improved design accuracy.

### **1. Introduction**

High-strength steels, with material yield strengths greater than or equal to 460 MPa, have been increasingly used in bridge and structural engineering (Zhao et al. 2023). Compared with normal-strength steels, high-strength steels possess more exceptional mechanical strengths, enabling structural members to be designed with smaller sizes and lighter self-weights, which can facilitate transportation of the structural members and increase interior spaces of the whole structure. With regard to high-strength steel welded I-section structural members, previous relevant research is briefly reviewed herein. Stub column tests were conducted on high-strength steel welded I-sections (Li et al. 2019; Shi et al. 2014; Sun et al. 2019b) to study their local buckling behaviour and cross-section compression resistances. The in-plane bending behaviour

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and resistances of high-strength steel welded I-section beams were experimentally investigated by Lee et al. (2013) and Sun et al. (2019a). The test results obtained from both stub column and beam tests indicated that the cross-section resistances of high-strength steel welded I-sections under compression can be generally well predicted by the existing design rules, but their bending resistances are unduly underestimated by the design rules. Sun et al. (2021b) conducted combined loading tests on high-strength steel welded I-sections to investigate their local buckling behaviour and load-carrying capacities under combined compression and bending; based on the test results, the relevant codified design rules were evaluated, revealing good design accuracy. Pin-ended concentric compression tests were conducted on high-strength steel welded I-section columns (Ban et al. 2012; Shi et al. 2012; Sun et al. 2020, 2021a) to study their flexural buckling behaviour, with the inaccuracy of the codified buckling curves highlighted and improved design methods proposed. It is worth noting that all the aforementioned studies on high-strength steel welded I-section members were conducted at ambient temperature, while their structural behaviour and residual resistances after exposure to elevated temperatures have not received much attention. Fire safety is one of the main considerations for the design of a steel structure, but fires always occur in parts of structures and only a small percentage of them will cause the whole structures to collapse (Sun et al. 2023). Therefore, it is important to accurately evaluate the extent of damage and residual load-carrying capacities of the structures after exposure to fire, which then lets strengthening and retrofitting become possible.

The present study focuses on the post-fire behaviour and residual resistances of S690 high-strength steel welded I-section columns. A numerical modelling programme was performed, with finite-element (FE) models developed to simulate the existing test results and employed to conduct parametric studies to generate sufficient post-fire structural performance data. The obtained numerical data were used to evaluate the suitability of the relevant ambient temperature design rules, as given in EN 1993-1-12 (EC3) (CEN 2007) and ANSI/AISC 360-16 (AISC 2016), for S690 high-strength steel welded I-section columns after exposure to elevated temperatures. Finally, an improved design method was proposed.

## 2. Numerical Modelling

In this section, a numerical modelling programme is conducted to numerically investigate the structural behaviour and residual load-carrying capacity of high-strength steel columns after exposure to fire, using the ABAQUS finite-element (FE) package. FE models on S690 high-strength steel welded I-section columns were first developed and validated by the existing test results. The validated FE models were then adopted to perform systematic parametric studies to derive post-fire performance data over a range of elevated temperatures, cross-section dimensions and member slendernesses.

### 2.1 Development of FE Models

FE models were developed based on the tested S690 high-strength steel welded I-section column specimens reported in Sun et al. (2020), where ten pin-ended column tests were carried out at ambient temperature (i.e., 30 °C) and the ten column specimens were observed to fail by minor-axis flexural buckling. The geometric properties of the developed FE models are in line with that of the tested column specimens, as presented in Table 1, where  $L$  is the member length,  $L_e = L + 110$  mm is the member effective length,  $h$  is the outer section height,  $b$  is the flange width,  $t$  is

the plate thickness and  $\omega_g$  is the initial global geometric imperfection amplitude. The shell element S4R has been widely used in previous numerical modelling of welded I-section steel members (Ran et al. 2023a, 2023b) and was also employed herein. A mesh sensitivity study considering a series of element sizes varying between  $0.5t$  and  $4t$  was performed to select a suitable shell element size; the results generally show that an element size equal to the plate thickness  $t$  provides satisfactory computational accuracy and efficiency. With regard to the material modelling of grade S690 high-strength steel, the engineering stress–strain curve, measured from the material coupon tests (Sun et al. 2020), was first converted into the true stress–strain response and then incorporated into ABAQUS. The pin-ended boundary conditions adopted in the column tests were accurately represented in the FE models. Specifically, each end section of the numerical models was coupled to one reference point. Each reference point was located longitudinally at a distance of 55 mm away from the corresponding end section. The top reference point can translate longitudinally and rotate about the minor axis, while the bottom reference point only has the rotation about the minor axis. Initial global geometric imperfections were also incorporated into each column FE model, with the distribution profile given as the lowest global buckling mode obtained from an elastic eigenvalue analysis and factored by the corresponding measured imperfection amplitude.

Table 1. Measured geometric properties of tested column specimens. (Sun et al. 2020)

Cross-section	Specimen ID	$L$ (mm)	$L_e$ (mm)	$h$ (mm)	$b$ (mm)	$t$ (mm)	$\omega_g$ (mm)
I-50×50×5	A1	239	349	49.38	49.48	4.96	0.11
	A2	340	450	49.69	49.44	5.00	0.20
	A3	450	560	47.33	49.46	4.95	0.20
	A4	578	688	49.36	49.55	4.92	0.29
	A5	696	806	49.54	49.45	4.96	0.31
I-80×60×5	B1	299	409	80.09	59.39	4.97	0.12
	B2	469	579	79.54	58.96	4.99	0.17
	B3	596	706	79.43	59.12	4.93	0.20
	B4	650	760	79.99	59.27	4.97	0.25
	B5	720	830	78.06	58.99	4.95	0.37

## 2.2 Validation of FE Models

Once the FE models were developed, the materially and geometrically nonlinear ‘Static, Riks’ analysis was used to derive the numerical results, including numerical failure modes and loads as well as load–mid-height lateral deflection curves. The accuracy of the numerical results from the FE models was evaluated through comparison with the test results. Graphical comparisons between the test and numerical load–mid-height deflection curves for the tested column specimens are displayed in Fig. 1, where the test curves are found to be well captured by their FE counterparts. The mean test-to-FE ultimate load ratio is equal to 1.01, demonstrating that the developed FE models can provide good predictions of the failure loads. In addition to the good agreements between the test and FE load–deflection curves and failure loads, the experimentally obtained failure modes were also found to be accurately simulated by the FE models, as depicted in Fig. 2 for a typical specimen. Overall, the developed FE models were shown to simulate well the performance of S690 high-strength steel welded I-section columns failing by flexural buckling and were therefore demonstrated to be validated.

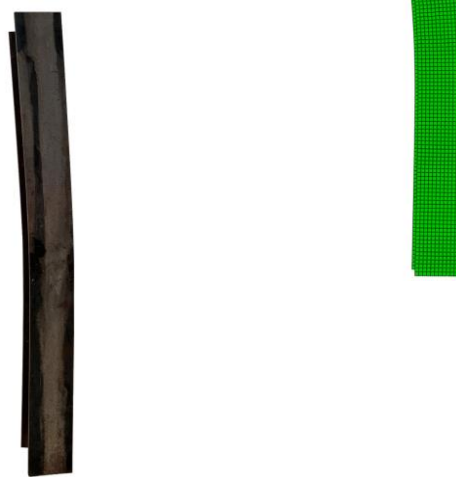
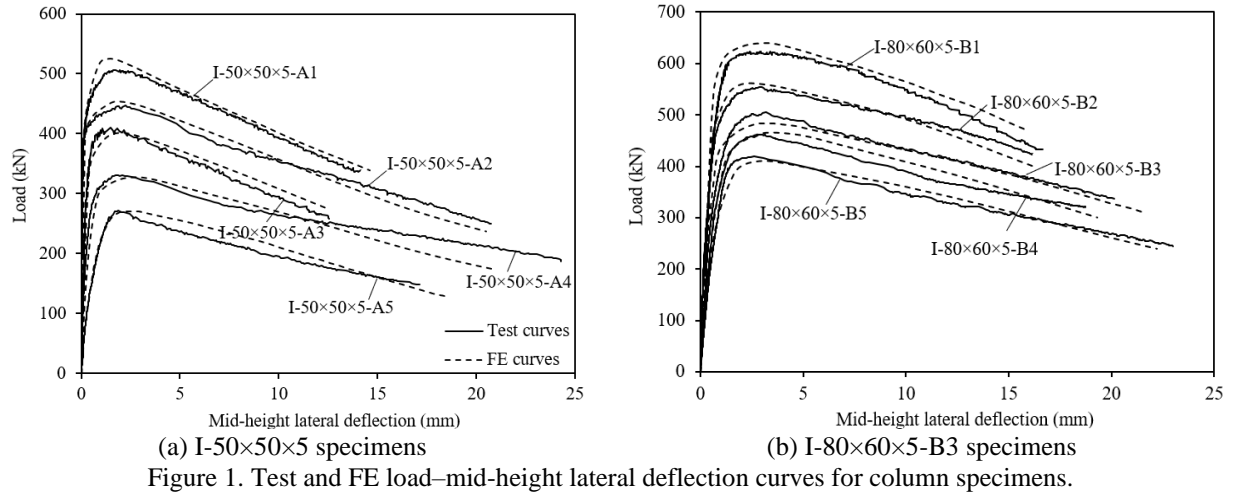


Figure 2. Test and FE flexural buckling failure modes of typical column specimen I-80×60×5-B3.

### 2.3 Parametric Studies

Upon validation of the FE models, systematic parametric studies were conducted to expand the data bank over a broader range of elevated temperatures, cross-section dimensions and member slendernesses. For the modelled I-sections, their geometric dimensions were selected carefully to ensure that they are categorised as non-slender I-sections according to both the EC3 and AISC cross-section classification frameworks. Specifically, the outer section heights were fixed at 100 mm, while the flange widths ranged from 50 mm to 100 mm and the thicknesses from 4 mm to 6 mm, enabling a wide range of cross-section dimensions and aspect ratios to be examined. The effective model length fell within the range of 450 mm to 3300 mm, resulting in a variety of member slendernesses. Seven elevated temperature levels were considered, including 30 °C, 300 °C, 600 °C, 700 °C, 800 °C, 900 °C and 950 °C, and the corresponding post-fire material stress–strain curves are shown in Fig. 3 (Su et al. 2020), with key post-fire material properties summarised in Table 2. All modelled columns were developed using the aforementioned modelling techniques, procedures and assumptions, but with the initial global geometric imperfection amplitudes taken as  $L_e/1000$  (Sun et al. 2020). In total, numerical data for 218 post-fire high-strength steel welded I-section columns were derived through parametric studies.

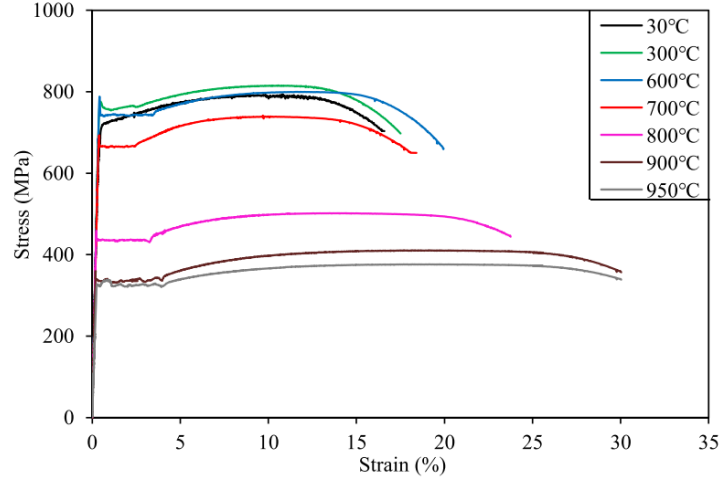


Figure 3. Post-fire material stress–strain curves of S690 high-strength steel. (Su et al. 2020)

Table 2. Key post-fire material properties of S690 high-strength steel.

Temperature $T$ (°C)	Young's modulus $E$ (GPa)	Yield strength $f_y$ (MPa)	Ultimate strength $f_u$ (MPa)
30	205.4	716	794
300	204.0	765	816
600	202.0	743	800
700	195.2	666	741
800	203.9	436	502
900	190.7	336	411
950	188.3	325	376

### 3. Design Analysis

In this section, the suitability of the existing design rules for post-fire design of S690 high-strength steel welded I-section columns, as set out in EN 1993-1-12 (CEN 2007) and ANSI/AISC 360-16 (AISC 2016), is evaluated by comparing FE failure loads  $N_u$  with the unfactored design resistances  $N_{u,pred}$ . An improved design method was then proposed for S690 high-strength steel welded I-section columns after exposure to fire, underpinned by the obtained post-fire structural performance data.

#### 3.1 Eurocode EN 1993-1-12

The current Eurocode EN 1993-1-12 for the room-temperature design of high-strength steel structures with material grades greater than S460 up to S700 was generally developed through mirroring the corresponding design rules given in EN 1993-1-1 (CEN 2014) for normal-strength steel structures. The buckling curves, as developed in accordance with the Perry-Robertson buckling formula, are used for the design of high-strength steel columns susceptible to global instability. Regarding S690 high-strength steel welded I-section columns failing by minor-axis flexural buckling, the EC3 resistance prediction  $N_{u,EC3}$  can be determined from Eq. 1.

$$N_{u,EC3} = \chi A f_y \quad (1)$$

where  $\chi$  is the reduction factor and can be determined from the EC3 design buckling curve, as given by Eq. 2,

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \leq 1 \quad (2)$$

where  $\phi$  is a buckling parameter and can be calculated from Eq. 3, in which  $\alpha$  is the imperfection factor considering the effects of geometric imperfections and residual stresses on the buckling resistances of columns. For S690 high-strength steel welded I-section columns failing by minor-axis flexural buckling, the buckling curve ‘c’ is adopted and  $\alpha = 0.49$ .

$$\phi = 0.5 \left[ 1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right] \quad (3)$$

The suitability of the EC3 buckling curve ‘c’ for the post-fire design of S690 high-strength steel welded I-section columns was evaluated, based on the post-fire structural performance data obtained through the parametric studies. Fig. 4 depicts the normalised numerical failure loads (by the cross-section yield loads  $A_f y$ ) plotted against the member slendernesses  $\bar{\lambda}$ , together with the EC3 curve ‘c’; it was generally found that the EC3 curve ‘c’ lies well below the numerical data points for S690 high-strength steel welded I-section columns after exposure to fire, revealing design conservatism. The  $N_u/N_{u,EC3}$  ratios are plotted against the member slendernesses in Fig. 5, while the mean  $N_u/N_{u,EC3}$  ratio and the corresponding COV are listed in Table 3. Both the graphical and quantitative evaluation results indicated that the current EN 1993-1-12 leads to safe but rather conservative resistance predictions for S690 high-strength steel welded I-section columns after exposure to fire.

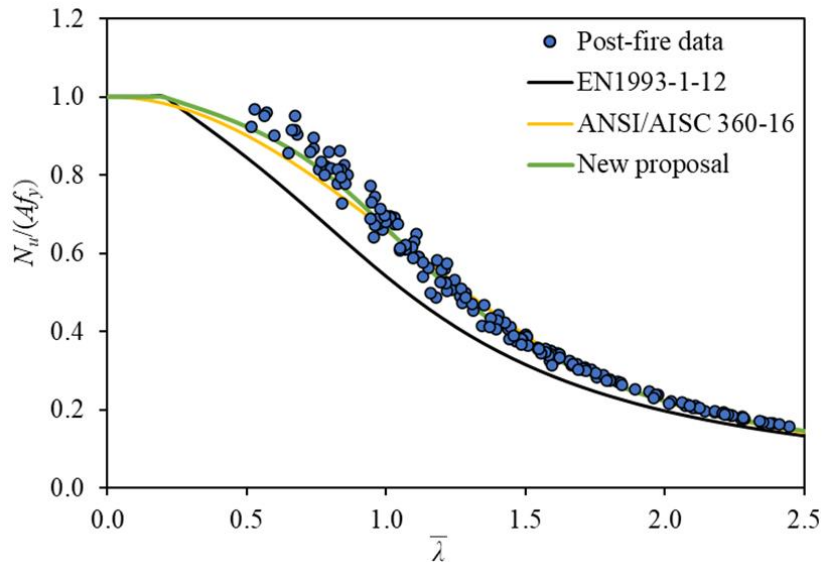


Figure 4. Comparisons of post-fire data with considered design curves.

Table 3. Comparison of numerical failure loads with design failure loads.

	$N_u/N_{u,EC3}$	$N_u/N_{u,AISC}$	$N_u/N_{u,pr}$
Mean	1.20	1.03	1.00
COV	0.010	0.023	0.019

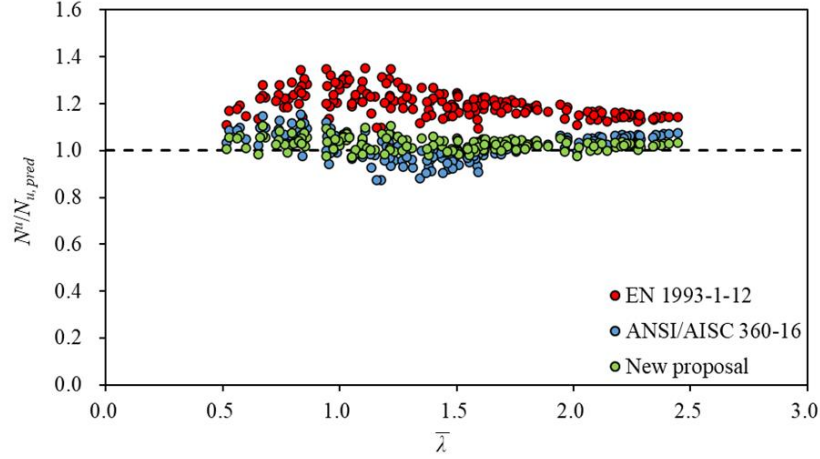


Figure 5. Evaluation of design post-fire residual resistances.

### 3.2 American Specification ANSI/AISC 360-16

The American specification ANSI/AISC 360-16 is applicable to carbon steels with the nominal material yield stresses less than or equal to 690 MPa. The flexural buckling resistance of a non-slender I-section column  $N_{u,AISC}$  is determined from Eq. 4, where  $F_{cr}$  is the critical stress and calculated from Eq. 5.

$$N_{u,AISC} = F_{cr} A \quad (4)$$

$$F_{cr} = \begin{cases} 0.658^{\bar{\lambda}^2} f_y & \text{for } \bar{\lambda}^2 \leq 1.5 \\ \frac{0.877}{\bar{\lambda}^2} f_y & \text{for } \bar{\lambda}^2 > 1.5 \end{cases} \quad (5)$$

The AISC design resistances for S690 high-strength steel welded I-section columns were evaluated through comparisons against the numerically obtained failure loads. Fig. 5 and Table 3 present the graphical and quantitative assessment results, respectively, revealing that the current ANSI/AISC 360-16 yields accurate design post-fire residual resistances on average, but results in many unsafe post-fire resistance predictions for S690 high-strength steel welded I-section columns. This is also evident in Fig. 4, where the numerical failure loads, normalised by the corresponding cross-section yield loads, are compared against the AISC design curve defined by Eq. 5.

### 3.3 Improved Design Method

Summing up of the graphical and quantitative evaluation results, it was concluded that EN 1993-1-12 generally results in conservative post-fire residual resistance predictions for S690 high-strength steel welded I-section columns, while ANSI/AISC 360-16 leads to overall accurate design flexural buckling resistances, but with some unsafe post-fire residual resistance predictions. This thus prompted the development of safe and accurate design methods for S690 high-strength steel welded I-section columns after exposure to fire. An improved design method was proposed herein through using the original EC3 design buckling curve, as defined by Eqs 1–3, but with a reduced imperfection factor  $\alpha=0.21$ , i.e. the adoption of the EC3 design buckling curve ‘a’. The recommended buckling curve for the post-fire design of S690 high-strength steel

welded I-section columns is shown in Fig. 4, exhibiting a good representation of the FE data point over the whole member non-dimensional slenderness range. The recommended design buckling curve was shown to result in a substantial improvement in the design accuracy and consistency over the original EC3 design buckling curve, while in comparison with the AISC design buckling curve, the recommended design buckling curve yields even more accurate post-fire resistance predictions for those relatively short S690 high-strength steel welded I-section columns (with  $\bar{\lambda} \leq 1.0$ ) and safe-sided post-fire resistance predictions for their intermediate and long counterparts (with  $\bar{\lambda} > 1.0$ ). This is also evident in Fig. 5, where the graphical comparison of the post-fire residual resistance predictions from the proposed design method and two considered design standards are shown. Quantitative evaluation of the accuracy of the proposed design method was conducted through comparing the numerical failure loads  $N_u$  against the predicted post-fire resistances  $N_{u,pr}$ . The mean  $N_u/N_{u,pr}$  ratio, as presented in Table 3, is equal to 1.00, indicating that proposed design method yields more accurate and safer residual resistance predictions for S690 high-strength steel welded I-section columns after exposure to fire.

#### 4. Conclusions

A thorough numerical modelling programme has been performed to investigate the structural behaviour and residual resistances of S690 high-strength steel welded I-section columns after exposure to fire. Seven elevated temperature levels ranging from room temperature to 950 °C were taken into consideration in this study. The column FE models were first developed to simulate the column tests on S690 high-strength steel welded I-sections. The developed FE models were then validated through comparisons of the experimental and numerical failure modes, failure loads and load–deformation curves. Upon validation, the column FE models were adopted to conduct systematic parametric studies to generate sufficient post-fire structural performance data over a wide range of elevated temperatures, cross-section dimensions and member slendernesses. The obtained numerical results were adopted to evaluate the suitability of the relevant column buckling design rules for post-fire design, as given in EN 1993-1-12 and ANSI/AISC 360-16. The evaluation results generally revealed that EN 1993-1-12 results in conservative resistance predictions for S690 high-strength steel welded I-section columns after exposure to fire, while ANSI/AISC 360-16 leads to an overall acceptable level of design accuracy, but with many over-predicted flexural buckling resistances. Finally, an improved design method with a new imperfection factor was developed; the new design method was shown to provide a higher level of design accuracy and consistency over the European and American design standards.

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