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Fire Resistance Studies on High Strength Steel Structures

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Abstract

High strength steels have been widely applied in recent years due to high strength and good working performance. When subjected to fire conditions, the strength and elastic modulus of high strength steels deteriorate significantly and hence the load bearing capacity of structures reduces at elevated temperatures. The reduction factors of mechanical properties of high strength steels are quite different from mild steels. Therefore, the fire design methods deduced from mild steel structures are not applicable to high strength steel structures. In recent ten years, the first author of this paper has carried out a lot of fundamental research on fire behavior of high strength steels and structures. Summary of these research is presented in this paper, including mechanical properties of high strength steels at elevated temperature and after fire exposure, creep response of high strength steels at elevated temperature, residual stresses of welded high strength steel member after fire exposure, fire resistance of high strength steel columns, fire resistance of high strength steel beams, local buckling of high strength steel members, and residual strength of high strength steel columns after fire exposure. The results show that the mechanical properties of high strength steel in fire condition and the corresponding fire resistance of high strength steel structures are different from those of mild steel and structures, and the fire design methods recommended in current design codes are not applicable to high strength steel structures.

Keywords: High strength steels, Fire resistance, Mechanical properties, Creep, Residual stress, Fire exposure

1. Introduction

Steels used in construction applications are classified into different categories based on chemical composition, tensile strength properties, and fabrication process as carbon steels, low-alloy steels, heat-treated carbon steels, and high strength steels. Recent years have seen an increasing demand for high strength steels in structural applications. High strength steel is gaining more widespread usage as a load bearing construction material due to its combination of high yield strength, good working performance. In some significant structures and landmark constructions, high strength steels have been successfully used and good economic effect has been found, such as New York Freedom Tower, Beijing "Bird's Nest" Olympic Stadium and French cable-stayed road-bridge Millau Viaduct.

As we know, steels have excellent mechanical properties at ambient temperature. However, the buildings made of steel can be inevitably exposed to fire hazards. Deterioration of mechanical properties and creep deformation are considered the most important factors affecting the behavior of steel structures in fires. Much research has been carried out on fire resistance of mild steel structures and design specifications have been published (BSI, 2003, ECS, 2005, GB, 2017). There are no design guidelines in current fire design codes for assessing fire exposed high strength steel structures.

In recent ten years, the first author of this paper have carried out a lot of fundamental research on fire behavior of high strength steels and structures. The topics include mechanical properties of high strength steels at elevated temperature and after fire exposure, creep response of high strength steel at elevated temperature, residual stress of welded high strength steel member after fire exposure, fire resistance of high strength steel columns, fire resistance of high strength steel beams, local buckling of high strength steel members, and residual strength of high strength steel columns after fire exposure. In this paper, the main findings of these research were summarized and presented.

2. Mechanical Properties of High Strength Steels at Elevated Temperatures

Yield strength and elastic modulus of steel are two key mechanical properties needed in fire resistance design of steel structures. To develop high temperature strength and stiffness properties of high strength steels, a comprehensive test program was designed.

2.1. Test Program

Q460 and Q690 steels are two commonly used high strength structural steels, with nominal yield strength of 460 MPa and 690 MPa, respectively. A tensile testing machine SANS-CMT5305 with 300 kN loading capacity

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was used to tensile the specimens made of Q460 and Q690 steels at elevated temperature (Wang et al., 2013b, 2018b). The high-temperature chamber with a temperature controller, which can reach the maximum temperature of 1200ºC, was used to heat the specimen. The extensometer was placed on the displacement collection instrument to measure the strain of the specimen. The shape and dimensions of the specimens were prepared in accordance with SAC (2010). To improve the reliability of test results, at every temperature, three specimens were prepared.

In the tension tests, the specimen was heated up to a predefined target temperature with a heating rate 20ºC/min and then loaded until it failed while maintaining the same temperatures. When the target temperature is attained, the specimen is maintained at this temperature for 20 min to ensure the total specimen achieve uniform temperature. And then tensile load was applied to the specimen until failure. Strain-control was used in this investigation. When strain ε is lower than 1.0%, the tensile rate was 0.003/min, when strain ε is higher than1% but lower than 2%, the tensile rate uniformly increased to 0.02/min. When strain ε is higher than 2%, the tensile rate maintains the rate of 0.02/min.

2.2. Test Results and Comparison

The tested stress-strain curves at different temperature levels can be obtained based on aforementioned test procedures. Since the yield plateau disappears at elevated temperatures, yield strength of steel was determined according to different strain level. A commonly accepted yield strengths of steel (f_{yT}) at strain levels of 0.2% offset was recommended (Wang et al., 2013b). The 0.2% yield strength (noted as $f_{0.2}$) is the intersection point of the stress-strain curve and the proportional line offset by 0.2% strain. The elastic modulus at elevated temperature (E_T) is defined as the slope of the stress-strain curve in the linear-elastic range. The yield strength and elastic modulus reduction factor were calculated as the ratio of the yield strength and elastic modulus of steel at elevated temperatures to those at ambient temperature (f_y and E). The yield strength f_{yT} and elastic modulus E_T for Q460 and Q690 steels were plotted in Fig. 1 by comparison with those of other types of high strength steels tested by Qiang et al. (2012b, 2012c, 2016) and current provisions in ECS 2005 and GB 2017. As mentioned above, these code provisions are based on measured data for mild steel. As can be seen from Fig. 1, different steel exhibits different reduction factor of mechanical properties, and the use of code provisions for the fire resistance design of high strength steels may lead to conservative or unsafe predictions.

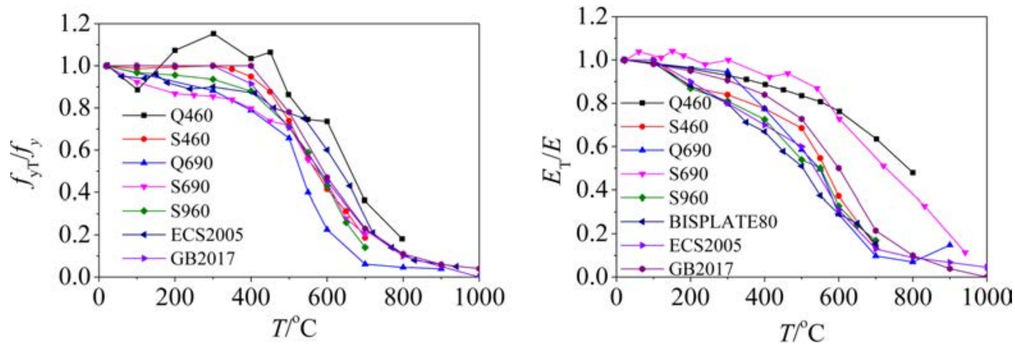
3. Mechanical Properties of High Strength Steels after Fire Exposure

To determine the post-fire mechanical properties of high strength Q460 and Q690 steels, tensile coupon tests were conducted to obtain their residual stress-strain curves and mechanical properties (yield stress f_{yT}' and elastic modulus E_T') after being exposed to pre-selected temperatures up to 900ºC (Wang et al., 2015b).

3.1. Test Methods and Specimens

The post fire mechanical properties tests of high strength Q460 and Q690 steels are carried out on typical material testing machine with the specimens being exposed to high temperatures. The heating of specimens was conducted in an electric furnace. After reaching every pre-selected elevated temperature, the temperature was held for 20 min. to ensures the total specimen achieving the uniform temperature. Two cooling methods were considered, which is natural air cooling and cooling by water. Natural cooling is defined as the specimen cools down in ambient temperature. Water cooling is defined as the specimen cools down in water.

The shapes and sizes of the specimens were in accordance with SAC (2010). A tensile load was then applied at a constant strain rate of 0.00025s^{-1} during elastic stage and yield stage. And then the constant displacement rate of 3 mm/min was applied to the specimen until failure. The extensometer was fixed on the middle part of specimen to



(a) Yield strength (b) Elastic modulus

Figure 1. Comparison of mechanical properties of test results with codes

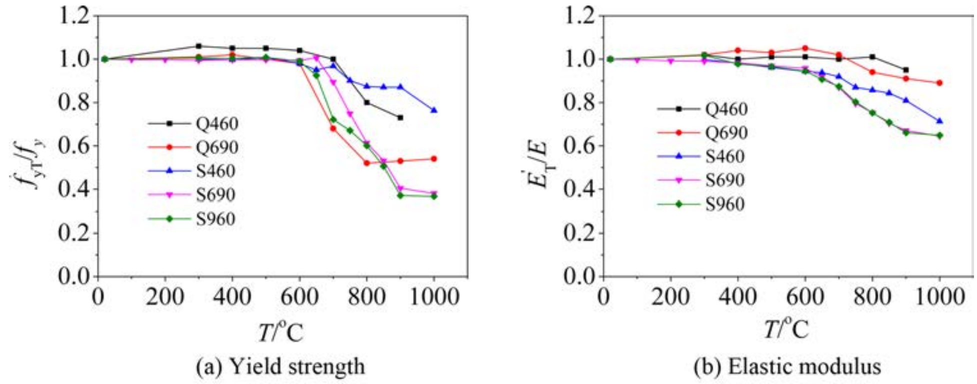


Figure 2. Comparison of mechanical properties of high strength steels after fire exposure.

obtain the extension of gauge length. The load and displacement were recorded by the data acquisition system. At every temperature, including ambient temperature, 3 specimens were tested to obtain more reliable results by averaging the test data.

3.2. Test Results and Discussions

Like the definition of yield strength at elevated temperature, the yield strength based on 0.2% proof stress method is measured for the yield strength of high strength steels after fire exposure. The elastic modulus of post-fire steel is determined from the initial slope of the stress-strain curve. The post-fire material properties of steels are represented by residual factor. The yield strength and elastic modulus of post-fire high strength Q460 and Q690 steels based on both cooling methods can be obtained by averaging the three specimens at the same temperature and with same cooling method. The residual factors of the yield strength and elastic modulus are plotted in Fig. 2 and are also compared with those of other type-high strength steels (Qiang el al., 2012a, 2013).

As is shown in Fig. 2, the deterioration of mechanical properties of high strength steels is not obvious for the temperature exposed lower than 600°C, and the deterioration exhibits significant decline for the exposure temperature exceeding 600°C. The deterioration of yield strength is more obvious than that of elastic modulus at the similar temperature.

4. Creep Strain of High Strength Steels at Elevated Temperatures

When exposed to elevated temperatures, steel structures undergo permanent deformations even when the applied stress level is below that of yield stress and this time-dependent deformation is referred to as creep. Thus, creep can be defined as an increase in strain in a solid material under constant stress over a period. A set of creep tests were carried out on Q460 and Q690 steel coupons at various stress levels in 300~900ºC range. Using this high temperature creep data, creep models based on the creep power law, were proposed for high strength steels.

4.1. Test Program

For undertaking high temperature creep tests, high temperature creep test equipment was specially designed and fabricated. The test set-up comprises of a tensile testing equipment, an electric furnace, a strain measurement device and a data acquisition system. The tensile testing equipment is MTS loading system with a load capacity of 50 kN. In this equipment, the axial deformation in the test coupon at elevated temperature can be measured through two displacement transducers (±10 mm LVDTs), with 0.001 mm sensitivity, that are mounted outside the furnace through two connecting rods placed inside the furnace. The electric furnace comprises of cylindrical chamber with a maximum heating length of 150 mm. The furnace and LVDTs are connected to a data acquisition system and a computer wherein temperature, tension load and displacements during the creep test are recorded. More details of test set-up are available in Wang et al. (2016, 2017, 2018b).

4.2. Test Results

Data generated from above creep tests can be utilized to develop creep response of Q460 and Q690 steels as a function of temperature and stress level. The measured creep strain at various stress levels and at different temperatures, from 300ºC to 900ºC, are plotted in Fig. 3. In Fig. 3, symbol "⊕" represents the initiation (boundary point) of secondary creep and tertiary creep, and symbol "⊗" represents the occurrence of fracture at the termination of tertiary creep. Generally, at a given temperature, the creep strain at a higher stress level is larger than that at a lower stress level. For the creep deformation at high temperature, the value is large enough and most specimen did not experience fracture.

4.3. Creep Model

Some empirical models to predict creep strain of carbon

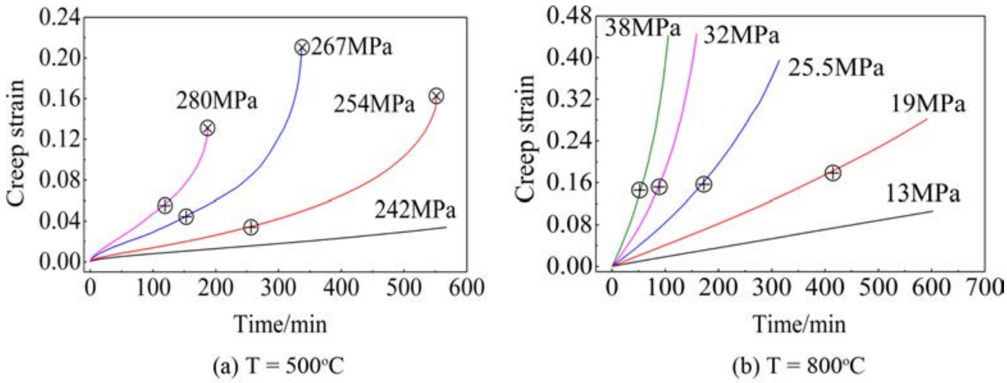


Figure 3. Typical creep strain-time curves of Q460 steel at various stress.

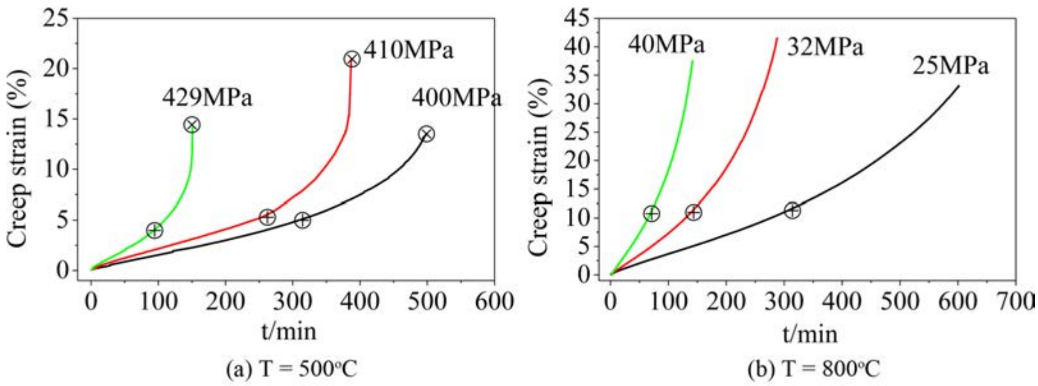


Figure 4. Typical creep strain-time curves of Q690 steel at various stress.

steel at elevated temperature are available in the literatures. Fields & Fields (1989) creep model is one of the widely used creep models in structural fire engineering applications. This model incorporates a power law and represents general expression for creep strain, ε_{cr}, in the form of a Norton-Bailey equation, which is given as

$$\varepsilon_{cr} = at^b\sigma^c \tag{1}$$

where ε_{cr} is the creep strain in steel; t is time (in min) and σ is stress (in ksi); a, b and c are temperature-dependent material properties.

Due to the facts that creep response in steel is highly material dependent, current models are not applicable to Q460 and Q690 steels. Test data generated from creep test can be utilized to develop creep model as a function of temperature and stress levels for these two high strength steels.

To simulate high temperature creep response in Q460 steel using the form of Fields and Fields creep model, a nonlinear regression fitting tool was employed and the values of coefficients a, b and c were evaluated at different temperatures, using the creep test data in Q460 steel. These values of a, b and c are tabulated in Table 1.

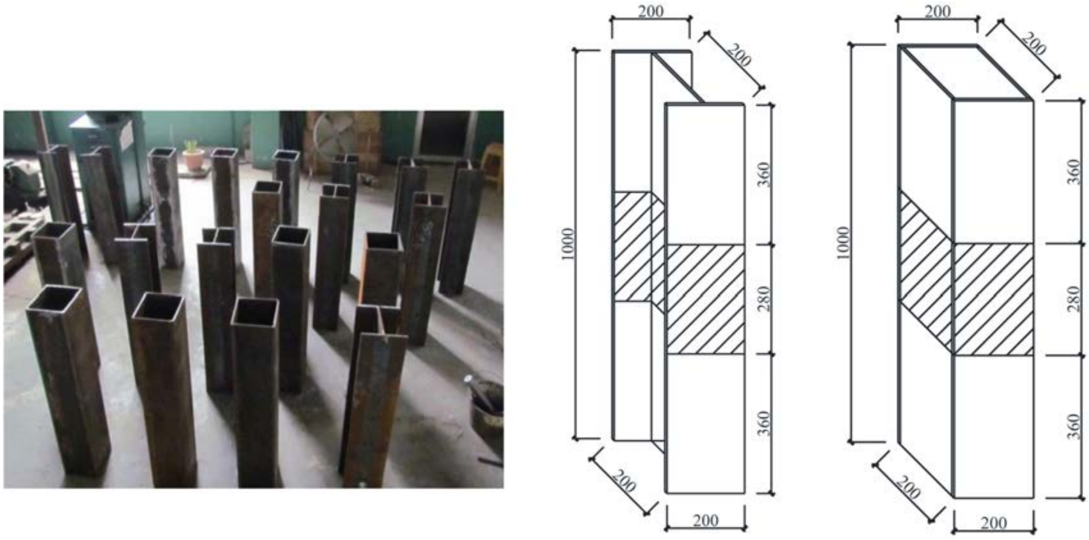
To predict high temperature creep response in Q690 steel using Field and Field creep model, a nonlinear regression fitting tool was employed. Due to a wider range of temperatures in the creep tests, the creep model was added a parameter "d". The new expression is shown in Eq. (2)

$$\varepsilon_{cr} = a_1 t^{b_1}\left(\frac{\sigma}{d}\right)^{c_1} \tag{2}$$

The values of coefficients a_1, b_1 and c_1 were fitted as a function of temperature. The results of fitting are the applicable scope of these coefficients for temperature is 450~900°C. These fitting expressions of a_1, b_1, c_1, d were shown in Eqs. (3a)~(3d).

Table 1. Coefficients of a, b and c in Fields and Fields creep model for Q460 steel

Temperature	a	b	c
400°C	6.48E-22	0.49	11.25
450°C	1.00E-25	0.62	13.94
500°C	1.55E-21	0.80	12.37
550°C	9.38E-12	0.89	6.75
600°C	4.20E-09	0.95	5.53
700°C	2.52E-05	1.05	3.50
800°C	5.00E-03	0.98	2.25
900°C	4.00E-02	0.66	1.98



(a) Photo of specimens (b) Dimensions of H-shaped and box section

Figure 5. Specimens and dimensions.

$$a_1 = \begin{cases} 10^{-1.4868+0.00439T} & 450°\text{C} \leq T \leq 500°\text{C} \\ 10^{1.347+0.000378T} & 550°\text{C} \leq T \leq 900°\text{C} \end{cases} \quad (3a)$$

$$b_1 = 0.8937 + 9.68 \times 10^{-5} \quad (3b)$$

$$c_1 = 84.76 - 0.2238T + 0.0001524T^2 \quad (3c)$$

$$d = 1399.6 - 1.47119T \quad (3d)$$

5. Residual Stress of Welded High Strength Steel Sections

Presence of residual stress exhibits a pronounced influence on the stiffness and fatigue life of steel structures. The fire exposure has great influence on the residual stress distribution due to plastic deformation and creep behavior in steel at elevated temperatures (Wang et al., 2015a). To provide benchmark data for the theoretical models and post fire design recommendations, this section presents the experimental investigation on post fire residual stress in welded H-shaped and box sections by means of sectioning method for high strength Q460 steels (as shown in Fig. 5). The target fire temperatures, 200ºC, 400ºC, 600ºC and 800ºC were selected and the residual stress were measured after the temperature of specimens reached the target temperature and cooled down to room temperature (Wang and Qin, 2016, Wang et al., 2018a).

5.1. Test Program

A comprehensive test program was designed to undertake residual stress measurement after fire exposure on welded H-shape section and box section fabricated with Q460 steel. The test program of this study is consisted of two stages, namely, heating and cooling of stub columns in an electric furnace and the measurement of residual stress in specimens after fire exposure as well as at ambient condition by means of sectioning method.

5.2. Test Results of Residual Stress after Fire Exposure

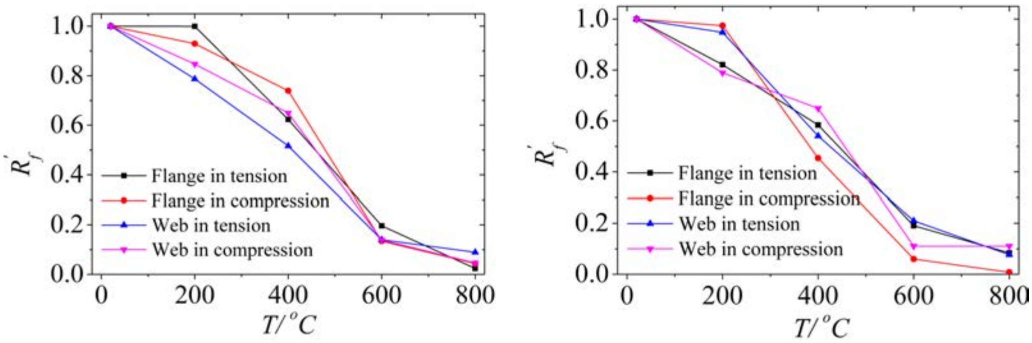
To investigate the effect of fire exposure on the magnitude of residual stress in welded steel sections, the reduction factor of residual stress R_f' were plotted in Fig. 6. The reduction factors are defined as the ratio of maximum residual stress (corresponding to tensile and compressive residual stress in flange and web, respectively) after fire exposure to that at room temperature. Fig. 6 illustrates that the reduction factors of the residual stress decreases when the exposed temperature increases. The residual stresses reduce to 20% when the exposed temperature exceeds 600ºC.

6. Fire Response of High Strength Steel Columns

6.1. Axial Compression Columns

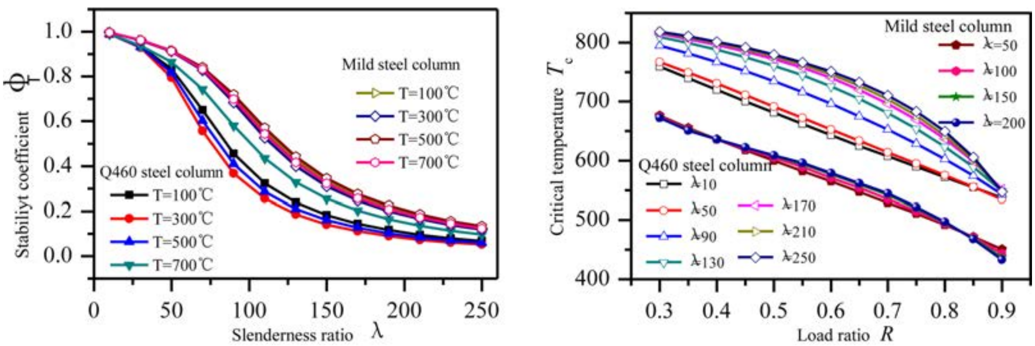
To develop a methodology for evaluating fire resistance of high strength Q460 steel columns, the load bearing capacity of high strength Q460 steel columns is investigated by extending the current approach of evaluating load bearing capacity of mild steel columns at room temperature with due consideration to high temperature properties of high strength Q460 steel.

The comparison of stability coefficient of high strength Q460 steel columns with mild steel columns are shown in Fig. 7(a). As can be seen from the figure, the stability coefficient of high strength Q460 steel columns is lower than



(a) H-shaped section (b) Box section

Figure 6. Reduction factor of residual stress after fire exposure.



(a) Stability coefficient (b) Critical temperature

Figure 7. Comparison of stability coefficient and critical temperature.

that of mild steel columns at the similar temperature and slenderness ratio. The critical temperature of high strength Q460 steel column obtained and compared with mild steel columns is shown in Fig. 7(b). This figure indicates that the critical temperature of high strength steel columns is much higher than that of mild steel at the similar temperature and slenderness ratio.

Parametric studies were also carried out by employing the proposed approach to study the effect of residual stress and geometrical imperfections. Results from parametric studies show that, only for a long column (slenderness higher than 75), the magnitude and distribution mode of residual stress have little influence on ultimate load bearing capacity of high strength Q460 steel columns, but the geometrical imperfections have significant influence on any columns. At a certain slenderness ratio, the stability factor first decreases and then increases with temperature rise (Wang et al., 2013a, 2014).

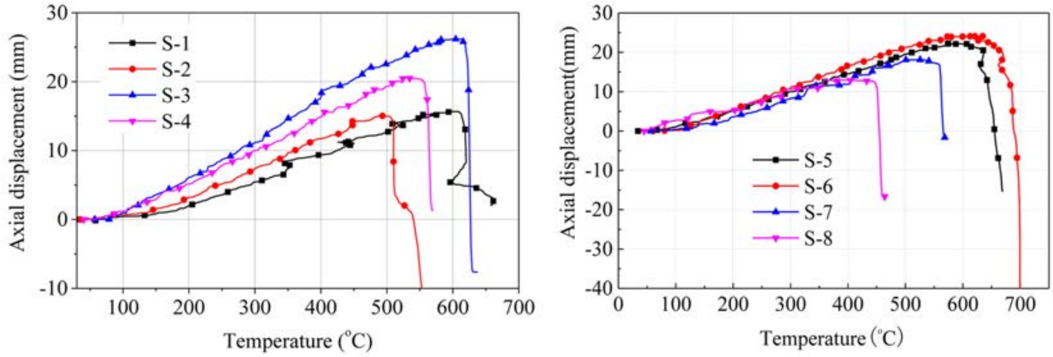
6.2. Restrained Columns

The behavior of restrained high strength Q460 steel columns at elevated temperature were obtained from full-scale fire tests (Wang et al., 2018c). In the fire tests, applied load and restraint stiffness (β_a and β_r are axial restraint ratio and rotational restraint ratio, respectively) are two key factors to be examined. Eight column specimens were made of Q460 steel plate welded to a H-shape section of H200×195×8×8, in which four specimens were designed with axial end restraints with the length of 4.3m, whereas the others are designed with both axial and rotational end restraints with the length of 4.48 m. The information of the test specimens is tabulated in Table 2.

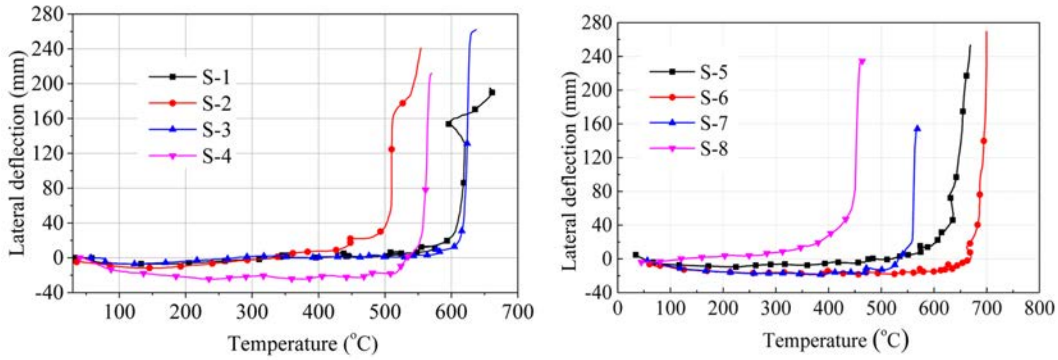
Column responses such as the axial displacement, deflection at column middle height and axial force induced by thermal expansion associated with temperature evolution were reported in Fig. 8. Column buckling and failure temperatures were determined based on the criteria of the axial displacement and lateral deflection of the specimens

Table 2. Parameters of the specimens

Specimen No.	End restraint	Load (ratio)	β_a	β_r
S-1	Axial	0.25	0.45	0
S-2		0.40	0.45	0
S-3		0.25	0.17	0
S-4		0.40	0.17	0
S-5	Axial and rotational	0.20	0.45	36
S-6		0.20	0.17	14
S-7		0.36	0.45	36
S-8		0.38	0.17	14



(a) Relationship between axial displacement and specimen's temperature



(b) Relationship between the lateral deflection and specimen's temperature

Figure 8. Axial displacements of test specimens.

(a) Specimen S-1

(b) Specimen S-2

(c) Specimen S-3

(d) Specimen S-4

(e) Specimen S-5

(f) Specimen S-6

(g) Specimen S-7

(h) Specimen S-8

Figure 9. Failure modes in test specimens.

at elevated temperatures. The test results show that both the applied load and restraint stiffness have considerable influences on fire resistances of high strength steel columns. It was observed that the columns with only axial

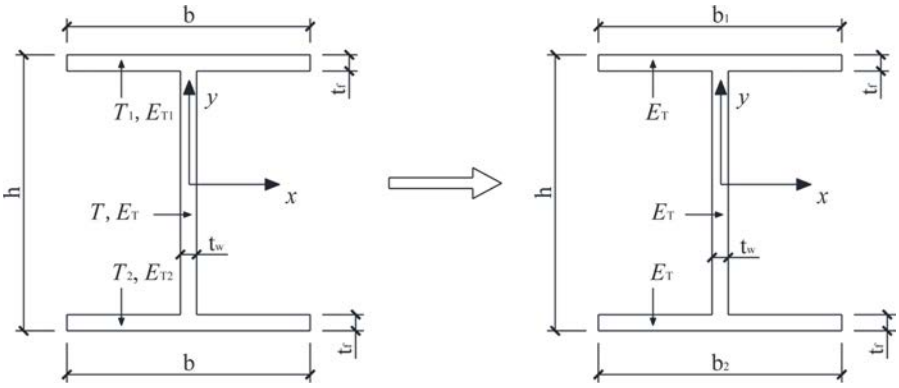


Figure 10. Dimensions of original and equivalent cross sections of the steel beam.

(a) Meshing of beam (b) Boundary condition and stiffeners (c) Buckling mode

Figure 11. Finite element model of steel beam.

restraints failed by flexure buckling about the weak axis whereas the columns with both axial and rotational restraints and subjected to large magnitude of the applied load failed by flexural torsional buckling (as shown in Fig. 9).

7. Fire Response of High Strength Steel Beams

Based on the critical bending moment associated with overall flexural stability and results obtained from the previous experimental investigation on the mechanical properties of Q460 steel at elevated temperature, an equivalent stiffness method (ESM) is established to evaluate the fire resistance of the beam with the consideration of the influence of temperature gradient across the section of the beam (Wang et al., 2018d). Using the equivalent stiffness principle, if the flexural stiffness of each flange with respect to their individual axis of symmetry is identical as shown in Fig. 10, the equivalent flexural stiffness of top and bottom flanges can be determined. Lateral torsional buckling resistance, critical temperature and overall stability coefficient are obtained for flexure of high strength Q460 steel beams at elevated temperature. A 3-D nonlinear finite element model (Fig. 11), which is capable of accounting for temperature gradient and predicting critical bending moment of Q460 steel beam at elevated temperature, is developed.

Results from the finite element simulations are compared with the results determined by the proposed equivalent stiffness method and there is a good agreement between the results of the two methods with the maximum difference of 6% (Table 3). Using the equivalent stiffness method, parametric studies were carried out to investigate effects of steel grade and temperature distribution pattern on fire resistance of high strength Q460 steel beams. Accordingly, a simplified design approach was proposed to predict the critical temperature and overall stability coefficient of Q460 steel beams subjected to non-uniform temperature distribution. The simplified approach is applicable to high strength Q460 steel beams with cross section dimensions ranging from 175 mm to 350 mm and 250 mm to 500 mm for flange width and section height, respectively.

Table 3. Comparison of critical temperature of high strength steel beams

Span	Load ratio R=0.7			Load ratio R=0.9		
	ESM	ANSYS	Errors	ESM	ANSYS	Errors
4m	750°C	744°C	0.80%	601°C	575°C	4.33%
6m	747°C	732°C	2.01%	589°C	557°C	5.43%
8m	742°C	728°C	1.89%	575°C	542°C	5.74%

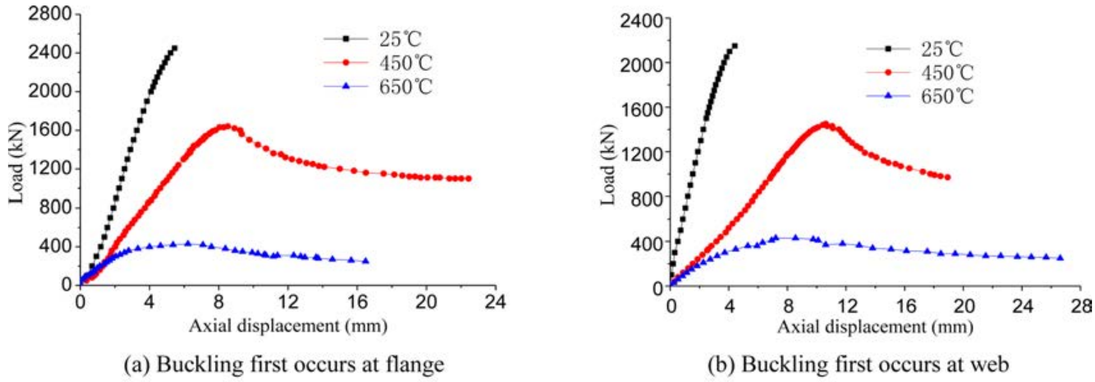


Figure 12. Axial displacement-load curves of Q460 steel columns.

8. Local Buckling of High Strength Steel Members

The existence of buckling at room and elevated temperature as well as the application of a large variety of high strength steel structural elements that contain exposed steel has been a motivation for this research, and to assess what degree of fire protection, if any, is needed for economic and safe structural design. Buckling of steel members, and local buckling, has a strong influence on the behavior of steel sections, both at ambient and elevated temperatures. Under fire conditions, steel members heat up quickly, primarily because of their usually high section factors and due to the good thermal conductivity of steel.

To examine the local stability of steel stub columns at elevated temperatures, experimental investigation on a total of 12 stub column specimens, including 6 mild Q235 steel columns and 6 high strength Q460 steel columns, was conducted under axial load in fire condition (Wang et al., 2014). The main purpose of these studies is to evaluate the variations of the buckling resistance of steel columns under different specified elevated temperature; in addition, to investigate the failure mode of stub columns at elevated temperature and find the difference between mild steel and high strength steel; and finally, to validate the EC3 treatment and the finite element analysis on local buckling of stub steel columns at elevated temperature. The axial displacement of stub columns, buckling deflection and local buckling failure mode of flange and web were recorded at room temperature and temperature of 450ºC, 650ºC.

8.1. Test Program

To conduct the tests, the tested specimen was placed into a big furnace. The furnace can generate temperatures up to 1200ºC, with an accuracy of ±0.5ºC through a temperature control system. In addition to the same 4 LVDT to measure the displacement of axial compression as in the ambient temperatures tests, only 3 LVDT placed outside of the furnace were used to measure the horizontal buckling displacement of web or flange through three holes on the wall of the furnace. A total of 6 thermocouples were placed within each specimen to obtain temperature distribution at three different locations.

For each test, the specimen is placed in the furnace,

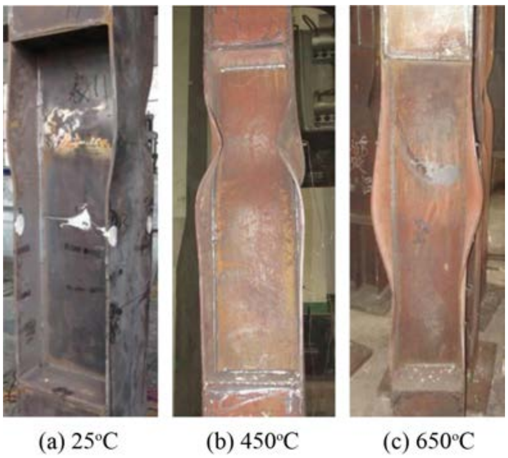


Figure 13. Buckling first occurs at flange.

Figure 14. Buckling first occurs at web.

which is turned on to heat up to a predefined target temperature. When the target temperature is attained, the specimen is maintained at this temperature for 10 min. to reach a steady-state condition throughout the portion of specimen inside the furnace. The axial load in jack was applied slowly using increments of 2.5% of load capacity before the load reaching 70% of calculated load capacity. Then, the load was controlled by the axial displacement at the top end of the columns. The displacements were controlled at 0.5 mm until the load reduced to 80% of calculated load capacity.

8.2. Test Results

Based on test result of axial displacement (shown in Fig. 12), the buckling resistances of flange and web were determined at room and elevated temperatures. It is found that with the same width-to-thickness ratio or height-to-thickness ratio, the buckling load of column specimens decreases while temperature increases. In general, the degradation of strength is 50% with the temperature of 450ºC. And it decreases more than 80% as the temperature reaches 650ºC. Both the strength and stiffness of the specimens drop rapidly between ambient temperature and 650ºC. This is like the inherent properties of the steel; the yield stress decreases significantly when the temperature reaches 650ºC.

It is observed that the failure mode of specimens at elevated temperature look like that at ambient temperature as shown in Figs. 13~14. For the web buckling specimen, the failure extent at elevated temperature is less than that at ambient temperature to protect the furnace from collision of damaged specimen.

9. Behavior of High Strength Steel Columns after Fire Exposure

If the structures experienced fires are dismantled and afterwards new alternates are built, it is wasteful and time-consuming. Upon cooling from elevated temperatures, the structural engineer then must decide if the residual strength of the load carrying members is still adequate for future use. To evaluate the residual strength of high strength Q460 steel columns after fire exposure, experimental study and finite element modeling on the structural behavior of high strength Q460 steel columns after being exposed to ISO-834 standard fire is carried out (Wang and Liu, 2016).

9.1. Test Program

To investigate the post-fire behavior of high-strength Q460 steel, a comprehensive test program was designed. The test program consisted of two stages; the first stage is fire exposure of the test specimens to ISO-834 standard fire, and the second one is compressive test on the specimen after fire exposure. In the tests, two section shapes were considered, namely welded H-shaped section and welded box section. A total of 4 columns fabricated with Q460 steel were tested, two of them are box section (B1 and B2) and the other one is H-shaped section (H1). The design length was 2540 mm for all specimens. The initial bending along the length of the column and residual stresses along cross-section were all determined before the compressive test.

The fire exposure experiments were carried out in a furnace. The temperature in the furnace was controlled automatically and was set according to the ISO-834 Standard fire, and air cooling. The specimens were put horizontally (as shown in Fig. 15) in the furnace in an unstressed condition to expose to a relatively uniform temperature zone. The temperatures of the furnace and steel columns were measured during both the heating and cooling phases as shown in Fig. 16. After fire exposure, the tests were paused while all the specimens cooled to approximately room temperature. All the specimens were tested by applying a concentric loading with a 5000 kN hydraulic compression machine. Both the top and bottom supports of the specimen were set to be fixed around strong axis and pin-supported around weak axis to assure the buckling occurs around weak axis.

9.2. Test Results

The load-deflection curves, load-axial displacement curves, load-rotation curves, load-strain curves and failure

Figure 15. Photos of specimens.

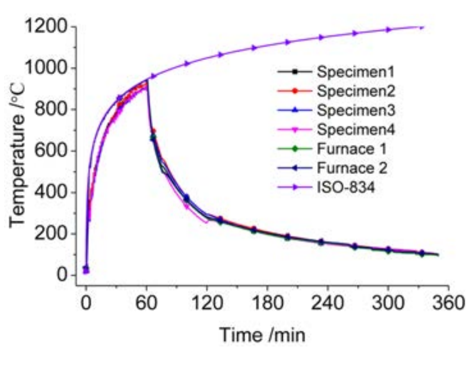


Figure 16. Temperature history of the specimens.

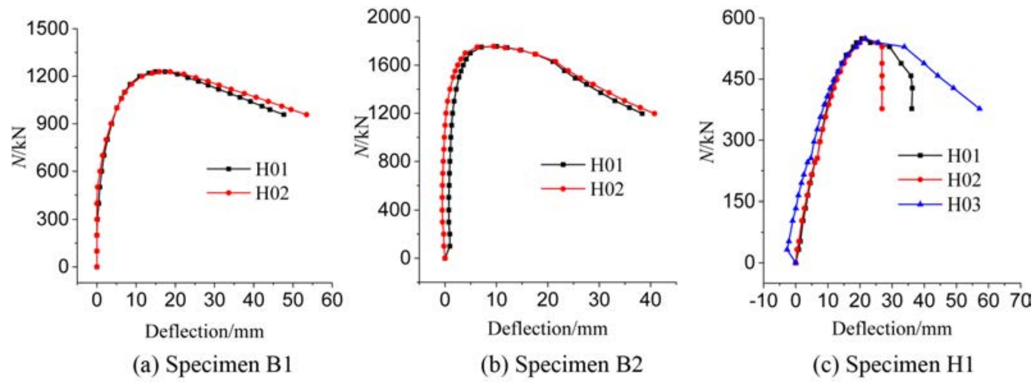


Figure 17. Load-deflection curves.

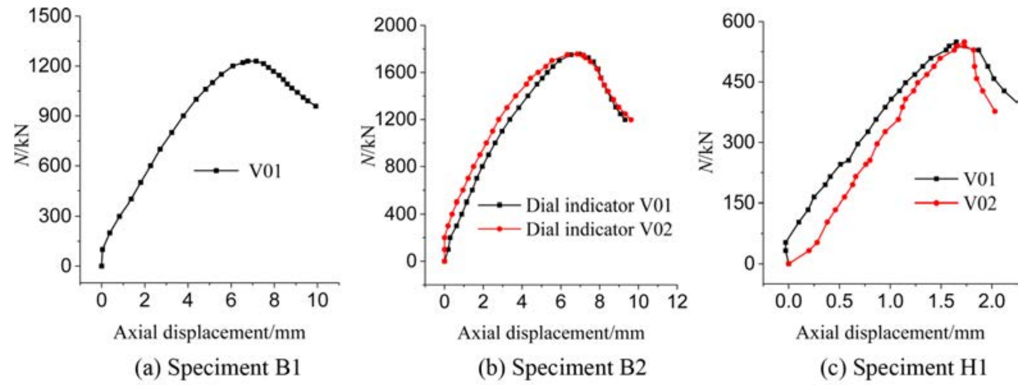


Figure 18. Load-axial displacement curves.

modes in the steel columns were measured during the compressive tests by horizontal LVDT H01 H02 and dial indicator V01 and V02. The applied load versus mid-height deflection curves and load-axial displacement curves of all specimens are plotted in Figs. 17 and 18, respectively. In the early stage of load-deflection curves, the specimens exhibit reduction of stiffness. For box-section specimens, the data generated from the LVDTs arranged on the same specimen are similar, which indicates that only flexural buckling occurs on the specimen and almost no torsion deformation was seen. However, for H-section specimen, there are some discrepancies of the three measured deflections. This may be attributed to local buckling on the flange or web at the mid-height of column. It is also seen that the axial displacement obtained from the dial indicators V01 and V02 are quite consistent with each other.

10. Summary and Conclusions

Main conclusions drawn from a serious of tests and related analytical studies on fire response of high strength steel structures can be summarized as follows:

(1) Different steel exhibits different reduction factor of mechanical properties and the use of code provisions for the fire resistance design of high strength steels may lead to conservative or unsafe predictions.

(2) The deterioration of mechanical properties of high strength steels is not obvious for the temperature exposed lower than 600°C, and the deterioration exhibits significant decline for the exposure temperature exceeding 600°C.

(3) At a given temperature, the creep strain at a higher stress level is larger than that at a lower stress level. For the creep deformation at high temperature, the value is large enough and most specimen did not experience fracture.

(4) The reduction factors of the residual stress decreases when the exposed temperature increases. The residual stresses reduce to 20% when the exposed temperature exceeds 600°C.

(5) The stability coefficient of high strength Q460 steel columns is lower than that of mild steel columns at the similar temperature and slenderness ratio. The critical temperature of high strength steel columns is much higher than that of mild steel at the similar temperature and slenderness ratio.

(6) The applied load and restraint stiffness have considerable influences on fire resistances of high strength steel columns. The columns with only axial restraints failed by flexure buckling about the weak axis whereas the columns with both axial and rotational restraints and subjected to large magnitude of the applied load failed by flexural tor-

sional buckling.

(7) The proposed equivalent stiffness method can accurately predict structural response of high strength steel beams with non-uniform temperature distribution.

(8) With the same width-to-thickness ratio or height-to-thickness ratio, the buckling load of high strength columns decreases while temperature increases. In general, the degradation of strength is 50% with the temperature of 450°C. And it decreases more than 80% as the temperature reaches 650°C. Both the strength and stiffness of the specimens drop rapidly between ambient temperature and 650°C.

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