Steel and Composite Structures, *Vol. 15, No. 6 (2013) 705-724*
DOI: http://dx.doi.org/10.12989/scs.2013.15.6.705

Fire performance curves for unprotected HSS steel columns

M. Shahria Alam [*1], A.H.M. Muntasir Billah [1], Shahriar Quayyum [2],
Mahmud Ashraf [3], A.N.M. Rafi [4] and Ahmad Rteil [1]

[1] *School of Engineering, The University of British Columbia, Kelowna, BC V1V1V7, Canada*
[2] *Department of Civil, Construction, & Env Engineering, North Carolina State University, Raleigh, NC 27695-7908, USA*
[3] *School of Engineering and IT, The University of New South Wales, Canberra, ACT 2600, Australia*
[4] *IES Associates Windsor, ON, N8Y 4Y6, Canada*

(*Received November 24, 2011, Revised August 29, 2013, Accepted September 16, 2013*)

Abstract. The behaviour of steel column at elevated temperature is significantly different than that at ambient temperature due to its changes in the mechanical properties with temperature. Reported literature suggests that steel column may become vulnerable when exposed to fire condition, since its strength and capacity decrease rapidly with temperature. The present study aims at investigating the lateral load resistance of non-insulated steel columns under fire exposure through finite element analysis. The studied parameters include moment-rotation behaviour, lateral load-deflection behaviour, stiffness and ductility of columns at different axial load levels. It was observed that when the temperature of the column was increased, there was a significant reduction in the lateral load and moment capacity of the non-insulated steel columns. Moreover, it was noted that the stiffness and ductility of steel columns decreased sharply with the increase in temperature, especially for temperatures above 400°C. In addition, the lateral load capacity and the moment capacity of columns were plotted against fire exposure time, which revealed that in fire conditions, the non-insulated steel columns experience substantial reduction in lateral load resistance within 15 minutes of fire exposure.

Keywords: HSS steel column; elevated temperature; moment-rotation; lateral load-deflection; axial capacity

1. Introduction

The temperature of steel rises rapidly when exposed to fire condition, due to its good thermal conductivity. The strength and the stiffness of steel decreases rapidly at elevated temperature, and the typical linear-elastic perfectly plastic stress-strain relationship becomes distinctly nonlinear (Twilt 1991, Poh 1998, Outinen 2007, Gardner *et al*. 2010). At ambient temperature, there is a well defined yield point separating the elastic and plastic portions of the stress-strain curve for steel, but with increasing temperature the behaviour becomes highly nonlinear with both strength and stiffness decreasing. The nonlinear behaviour of steel at fire conditions and the degradation of strength and stiffness influence the behaviour of steel members. Hence, the behaviour of steel

∗Corresponding author, Ph.D., E-mail: shahria.alam@ubc.ca

http://www.techno-press.org/?journal=scs&subpage=8 ISSN: 1229-9367 (Print), 1598-6233 (Online)

members in fire is significantly different from those under ambient temperature. In addition to the change in the material properties of steel, elevated temperatures also induce additional compressive stress due to thermal restraint (Cai and Feng 2010). Hence, in the fire design of steel building structures, the structures are required to have enough strength to resist the service load and the thermal force induced during the fire event. To prevent catastrophic collapse of the building structures, the members need to possess adequate strength at a specified temperature for a specific period of time so that the occupants can safely evacuate from the building (Yang *et al*. 2009).

In steel structures, columns are the main load-carrying members and hence, they are more vulnerable to temperature increase during fire event. In the last two decades, the behaviour of steel columns at elevated temperature has been studied experimentally, theoretically and numerically by a number of researchers. The buckling behaviour of axially and eccentrically loaded slender columns under fire condition was numerically and experimentally studied by Talamona *et al*. (1997) and Franssen *et al*. (1995, 1998). The fire resistance of steel columns has been investigated experimentally by Culver (1972), Ossenbruggen *et al*. (1973), and Poh and Bennetts (1995a, b). Lie (1994), Kodur and Lie (1997), Kodur (1998) performed experimental studies on fire resistance of hollow steel columns filled with plain and fibre-reinforced concrete and developed design equations to predict the fire resistance of structural hollow steel columns. Franssen and Detreppe (1992), Franssen *et al*. (1996) predicted the critical temperature of axially loaded members using a nonlinear computer code. Toh *et al*. (2000), Tang *et al*. (2001) and Yang *et al*. (2006a) examined the structural behaviour of steel columns in fire by loading a series of steel H columns to their limit states at specified temperature levels. The effect of axial or rotational restraint, load ratios, slenderness ratios on fire performance of steel columns have been examined by Ali and O'Connor (2001), Neves *et al*. (2002), Rodrigues *et al*. (2000), Yang and Hsu (2009), and Cai and Feng (2010). Takagi and Deierlein (2007), and Yang *et al*. (2006b) investigated strength of columns at fire conditions. Tao *et al*. (2011) investigated the fire performance of concrete-filled steel tubular (CFST) columns strengthened by CFRP. They concluded that CFST columns strengthened by CFRP can achieve required fire resistance if designed appropriately. Yang and Yu (2013) investigated the creep buckling of steel columns at elevated temperature. Based on their experiment, they proposed a creep strain rate model as the function of a single parameter of the load ratio of temperature to determine the buckling time of steel column due to creep. Recently, Heva and Mahendran (2013) proposed some guidelines for the cold-formed steel compression members subjected to flexural-torsional buckling and fire. They concluded that the current ambient temperature design rules are conservative while the fire design rules are overly conservative.

The previous studies suggested that the capacity (lateral load or moment carrying capacity), stiffness and strength of steel columns reduce significantly during fire exposure due to the degradation of the material properties with time and hence, proper fire protection measures need to be taken to improve the fire performance of steel columns. However, designing steel buildings for good fire performance, better knowledge on the behaviour and collapse of the structural members are required. Steel structures are often prone to fire events and thus require adequate fire protection in order to provide occupants ample time to escape during a fire. Although current design guidelines require passive fire protection on steel columns, very often steel columns are being constructed without any fire protection in many parts of the world. Moreover, a fire event can also take place during a construction period where fire protection is yet to be applied. In the case of a post-earthquake fire event, buildings may experience severe seismic damages, which could also result in unwanted lateral force demand in steel columns. Since fire hazards can substantially reduce the lateral resistance of steel structure (Ali *et al*. 2004), it is important to determine the lateral load resistance of the selected columns to fire exposure and determine how much protection should be provided to get the required hours of fire rating. Although there have been significant studies on the fire performance of steel columns, currently there exists no study or guideline on the performance of steel columns under lateral load during a fire event. Therefore, the present study is unique as it will develop fire performance curves for non-insulated columns under lateral loads, which will provide practitioners with guidelines for the fire safety of steel structures. This study is intended to contribute to the development of performance based fire engineering by developing performance curves Hollow Structural Sections (HSS) at elevated temperature.

The objective of this study is to investigate the effect of temperature increase on the behaviour of steel columns due to fire exposure by using finite element analysis. Hollow square sections (HSS) are commonly used for steel columns and hence, were chosen for this study. The sections were selected from the CISC Handbook (2006). Here, the lateral load-deflection behaviour, moment-rotation behaviour, lateral load capacity and moment capacity, stiffness and ductility of HSS columns were studied at elevated temperatures. These results at different temperatures and fire exposure time are presented where the lateral capacity of various steel columns can be readily known at different fire exposure time.

2. Numerical modeling and analysis

Fig. 1 shows the proposed generic framework for determining the fire performance of steel columns under lateral load at various temperature and axial load level. The developed performance curves of different column sections at different fire exposure times will help the design engineers to select column sections as per the required performance level and fire exposure time. The present

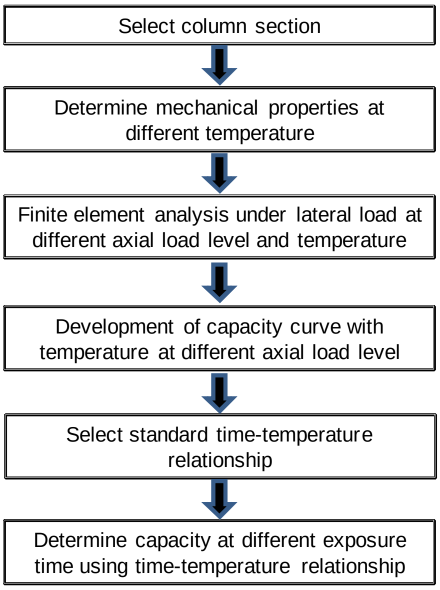


Fig. 1 Proposed generic framework for evaluating fire performance of steel column

study will be able to provide a good understanding of the behaviour of HSS steel columns during fire exposure. The steps for developing the performance curves are as follows:

(1) Select the column section, and load scenario.
(2) Determine the reduced mechanical properties (yield strength and modulus of elasticity) at different temperature level.
(3) Make an analytical model of the selected column.
(4) Analyze the column at different axial load and temperature level.
(5) Obtain the base shear-lateral deflection and moment –rotation relation of the column at different temperature level.
(6) Using the standard time-temperature relation obtain the variation of lateral capacity and moment capacity at different temperature.
(7) Using the standard time-temperature relationship obtain the lateral capacity and moment capacity at different fire exposure time.

A typical 3.2 m long column with hollow square section (HSS) was chosen as the reference column for the finite element (FE) analysis. A typical elevation of the reference steel column is shown in Fig. 2. Fifteen hollow square sections (HSS) were chosen from the CISC Handbook (2006). Table 1 shows the properties of the fifteen HSS columns considered in the analysis. Nonlinear static analysis was performed on the steel columns using a FE package SeismoStruct (Seismostruct 2010). The FE program is capable of predicting large displacement behaviour of structures taking into account, both geometric nonlinearities and material inelasticity. The fibre modeling approach was employed to represent the distribution of material nonlinearity along the length and the cross-sectional area of the member. Three-dimensional beam element was used for modeling the column where the sectional stress-strain state of the elements was obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibres in which the section was subdivided following the spread of material inelasticity within the member cross-section and along the member length.

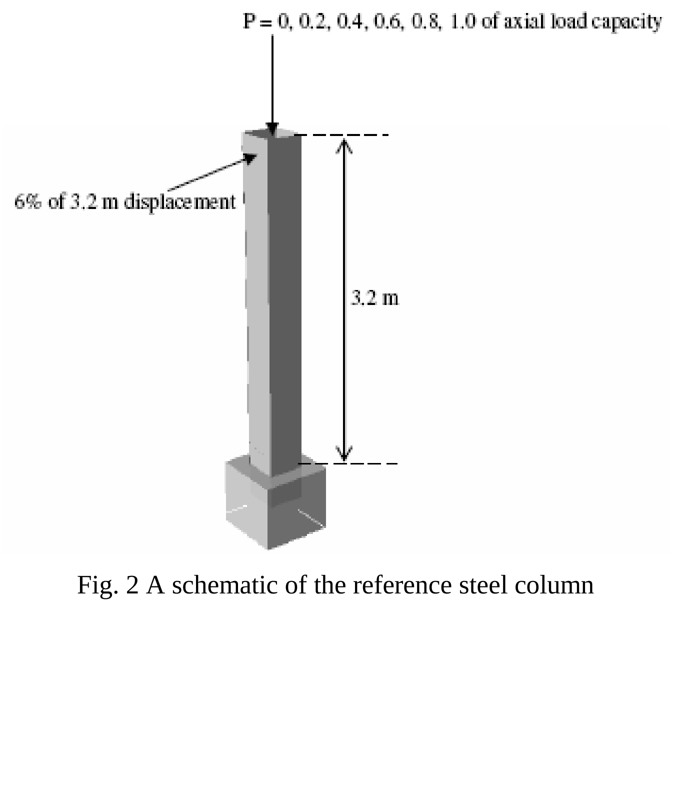


Fig. 2 A schematic of the reference steel column

Table 1 Properties of the CISC HSS columns considered in the study

Designation	Class	Area (mm^2)	Plastic section modulus, Z (10^3 mm^3)	Elastic section modulus, S (10^3 mm^3)	Radius of gyration, r (mm)	Axial load capacity (kN)	Moment capacity (kN.m)
HSS 305×305×16	1	17700	1890	1590	117	5060	595
HSS 305×305×13	1	14400	1560	1330	118	4130	491
HSS 305×305×9.5	3	11000	1210	1040	120	3180	381
HSS 305×305×8	3	9280	1030	886	121	2690	279
HSS 305×305×6.4	4	7480	833	625	121	1780	197
HSS 254×254×16	1	14500	1270	1050	96.1	3720	400
HSS 254×254×13	1	11800	1060	889	97.6	3060	334
HSS 254×254×9.5	2	9090	825	703	99.1	2380	260
HSS 254×254×8	3	7660	702	602	99.9	2020	221
HSS 254×254×6.4	4	6190	571	492	101	1640	155
HSS 203×203×16	1	11200	774	628	75.3	2250	244
HSS 203×203×13	1	9260	651	538	76.9	1910	205
HSS 203×203×9.5	1	7150	513	432	78.4	1510	162
HSS 203×203×8	1	6050	439	373	79.2	1290	138
HSS 203×203×6.4	3	4900	359	308	79.9	1060	113

Table 2 Properties of steel at ambient temperature (20˚C) considered in the study

Properties	Values
Modulus of elasticity	200000 MPa
Yield strength	360 MPa
Ultimate strength	500 MPa
Strain hardening parameter	0.005

Steel was represented by using a bilinear model. The material properties of steel at the ambient temperature are shown in Table 2. At elevated temperature, the strength and the stiffness of the steel is reduced and CSA S16 (2009) has proposed reduction factors for calculating the reduced yield strength and the modulus of elasticity based on the recommendations proposed by Eurocode 3 (2004). Fig. 3 shows the reduced yield strength and the modulus of elasticity of steel proposed by CSA S16 (2009), which was used to represent the material properties at the elevated temperature in this study. To evaluate the performance of the steel columns with the fire exposure time, the ISO 834 standard time-temperature curve (1980) was considered in the analysis (Fig. 4). The steel columns were investigated at different axial load levels, for instance, 0, 0.2, 0.4, 0.6, 0.8, 1.0 of the axial load capacity of the columns. In this study the axial capacities of the columns were calculated using code provided equations (CSA-S16) which possess some reserve strength that provides a partial safety factor towards its resistance. Since partial safety factors and conservative approximations are included in capacity equations, the resulting capacity is a nominal value. Even though we applied axial load equal to column’s axial capacity it still has some reserve capacity which allowed it to carry some additional lateral load. For example, the code calculated capacity of the HSS 305×305×16 section is 5060 kN, whereas the actual capacity is 5282 kN. This 4.2% reserve capacity allowed the column to carry some additional lateral load. The lateral load was applied as a lateral displacement of 6% of the total length of the column i.e., 192 mm. The temperature considered in the analysis varied from 20°C (ambient temperature) to 1000°C.

3. Validation with experimental result

Although the analysis has been performed using a freely available software, the authors have verified the software with several experimental results that consist of static and dynamic loading of structures. For instance, seismic performance of retrofitted RC bridge bent by Billah and Alam (2013), shake table test of SMA reinforced concrete column by Alam *et al*. (2008), quasi-static reversed cyclic loading test of SMA reinforced concrete beam-column joint by Alam *et al*. (2008),

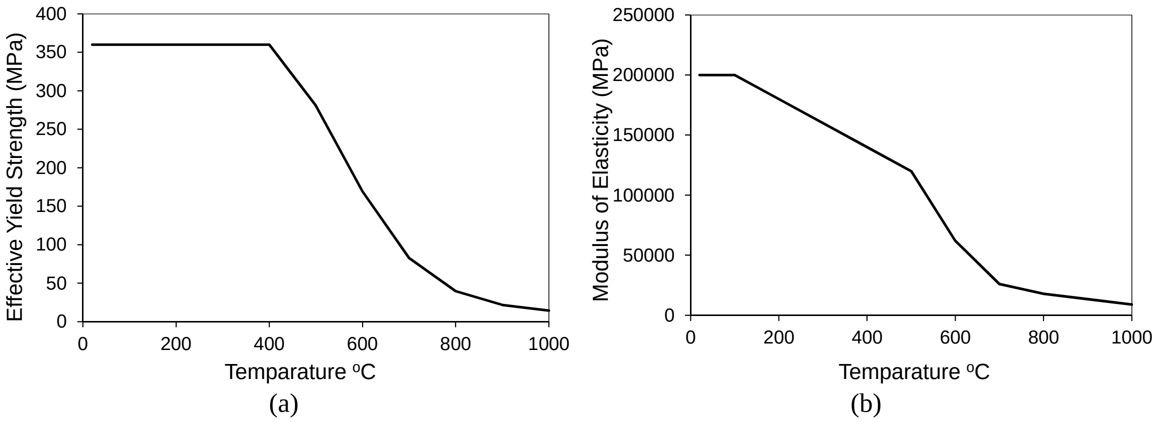


Fig. 3 Yield strength and modulus of elasticity of steel at different temperatures according to CSA S16 (2009): (a) yield strength; (b) modulus of elasticity

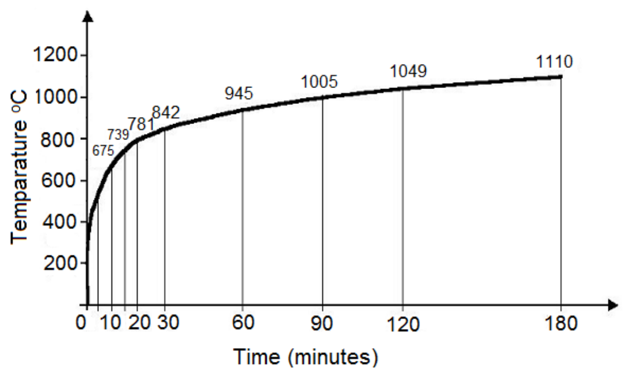


Fig. 4 ISO 834 standard time-temperature curve (1980)

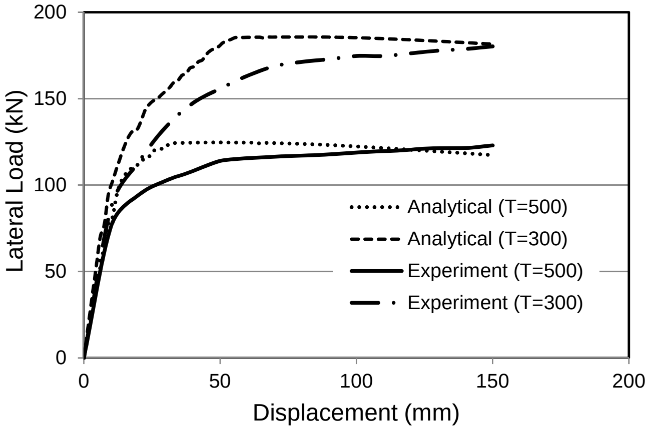


Fig. 5 Comparison with experimental result

and shake table test of a three storey RC frame by Alam *et al*. (2009). Besides, the software SeismoStruct was employed by the winner in the Practitioners category at the recent blind prediction contest deployed by PEER and NEES regarding the shaking-table testing of a full-scale circular bridge pier (PEER 2010).

This section presents the validation of the finite element model for predicting the standard fire behaviour of steel columns under lateral load. The finite element model was used to predict the standard fire behaviour of wide-flange steel columns tested by Choe *et al*. (2011). They tested W10 × 68 column at 300 and 500 degree Celsius temperature at axial load level of 0.15 and 0.3. They applied a lateral displacement of 150 mm (10% of test length) at the column top. They presented the lateral force-displacement-temperature relationship of the column. In order to validate the numerical model adopted in this study, the column was modeled using finite element software and analysed under same lateral load and axial load level of 0.3 under both 300 and 500 degree Celsius temperature. Fig. 5 shows the lateral force-displacement-temperature relationship obtained from both experimental and numerical study. From figure it is evident that the numerical model was capable of predicting the initial stiffness and ultimate capacity very well. The results obtained from numerical analysis showed good agreement with the experimental result in predicting the ultimate capacity with a small variation of 3.5% and 1.62% for 300 and 500 degree Celsius temperature, respectively. In predicting the initial stiffness the numerical results varied by 9.65% and 10.49%, respectively for 300 and 500 degree Celsius temperature. Although the behaviour is a little different after yielding, this is mainly due to the inability of the software to capture the softening of steel sections under increasing exposure time. Nonetheless, the present study focuses on the capacity and the initial stiffness of the steel columns under elevated temperature.

4. Experimental approach

Nonlinear static analysis was performed on the fifteen HSS column sections as mentioned in Table 1 and the complete results of two columns (HSS 254×254×16 and HSS 305×305×16) have

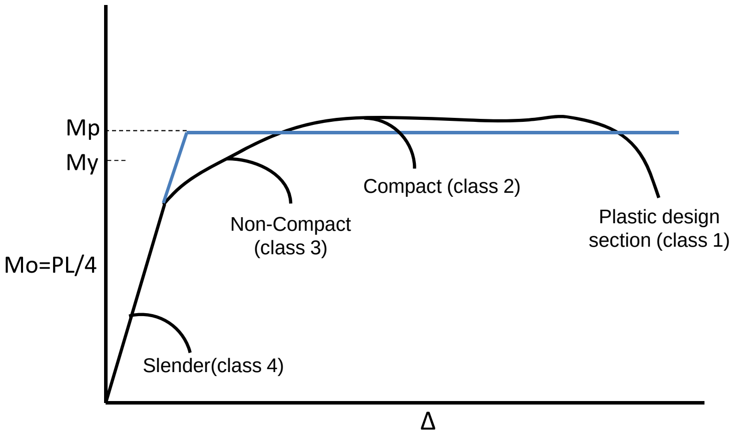


Fig. 6 Load-deflection response of different steel sections

been presented as a case study. However, the variation in moment capacity and lateral load capacity of columns with increasing temperature is presented for six sections (HSS 254×254×16, HSS 305×305×16, HSS 254×254×8, HSS 305×305×8, HSS 254×254×6.4 and HSS 305×305×6.4). Among these six columns two column sections were taken from Class 1, two column sections were taken from class and the rest two column sections were from Class 4. Class 1 and Class 2 sections have similar moment carrying capacity, therefore only Class 1 section has been considered as they will produce similar results in terms of capacity versus temperature. According to CSA S16-09, the available steel sections can be classified into four classes as shown in Fig. 6. CSA defined Class 1 sections as those which can be subjected with a bending moment equal to plastic moment, and given adequate cross-sectional stiffening. Class 2 section can be subjected with a bending moment equal to plastic, however, cannot undergo any local rotation. Class 3 section can be subjected with a bending moment equal to yield moment. The cross section starts buckling after the most outer fibres have yielded. Class 4 section fails locally before yield moment can be obtained. These columns were chosen as they produced the most conservative results among the fifteen columns and hence, the results can be taken as a guideline for the behaviour of HSS steel columns under fire conditions. The results of the nonlinear finite element analysis are presented in the following sections.

4.1 Lateral load-deflection behaviour

Figs. 7 and 8 show the lateral load-deflection behaviour of HSS 305×305×16 and HSS 254×254×16 columns, respectively for different temperatures at various axial load levels. It can be observed that their lateral load-deflection behaviour at different axial load levels is significantly influenced by the temperature due to the degradation of the material properties of steel with increase in temperature. It was noted that up to a temperature of 400°C, the lateral load-deflection behaviour of steel columns were quite close to each other, but after that there was a significant reduction in the lateral load capacity of the columns. This can be explained by the fact that up to 400°C, the effective yield strength of steel is the same, whereas the modulus of elasticity reduces by 30% (Fig. 3).

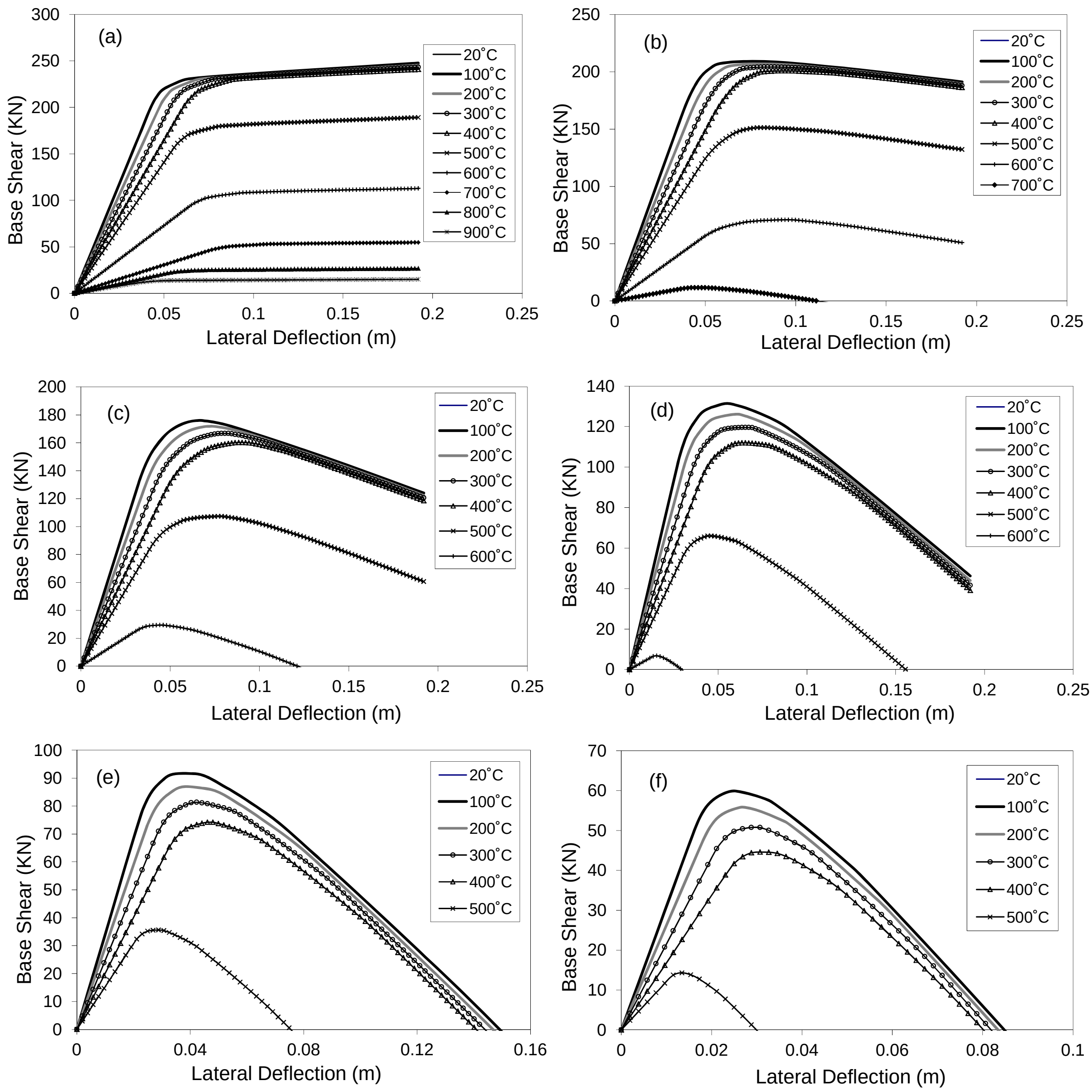


Fig. 7 Lateral load-deflection behaviour of HSS 305×305×16 column at different axial load levels: (a) No axial load; (b) Axial load = 0.2 × axial load capacity; (c) Axial load = 0.4 × axial load capacity; (d) Axial load = 0.6 × axial load capacity; (e) Axial load = 0.8 × axial load capacity; (f) Axial load = axial load capacity

For HSS 305×305×16 column with no axial load (Fig. 7(a)), the lateral load capacity was 247.6 kN at ambient temperature. At 400°C, the lateral load capacity reduced to 240.9 kN, which indicated a reduction in the lateral load capacity by only 2.7%. Whereas, at 500°C, the lateral load capacity was found 189.1 kN, which was a 24% reduction in the lateral load capacity. When the column was subjected to a temperature of 900°C, the lateral load capacity became only 15 kN, i.e., a 94%

reduction in the lateral load capacity. Similar behaviour was also observed for different axial load levels (Fig. 7). HSS 254×254×16 column also showed similar behaviour as observed from Fig. 8.

4.2 Stiffness and ductility

It was noted that with the increase in the temperature, the ductility (ability to undergo large deformation without collapse) and the stiffness of the columns decreased. Ductility is measured as the ratio of the displacements at ultimate and yield load, whereas the stiffness is measured by the initial slope of the load-displacement curve. At ambient temperature the stiffness at no axial load was 2632 kN/m and 4466 kN/m, respectively for HSS 254×254×16 and HSS 305×305×16 columns. Tables 3 and 4 present the ductility and stiffness of the columns at different temperatures under varying axial load levels, respectively. It was observed that with the increase in temperature, the ductility of both columns decreased sharply (Table 3). At 400°C and 1000°C, the ductility of the columns decreased by 27% and 73%, respectively (on an average), compared to the ductility at the ambient temperature when there was no axial load on the columns. On the other hand, in presence of axial loads equal to 20%, 40%, 60%, 80% and 100% of the axial load capacity, the average reduction in the ductility of the columns were 23%, 14%, 15%, 21%, and 20%, respectively, at

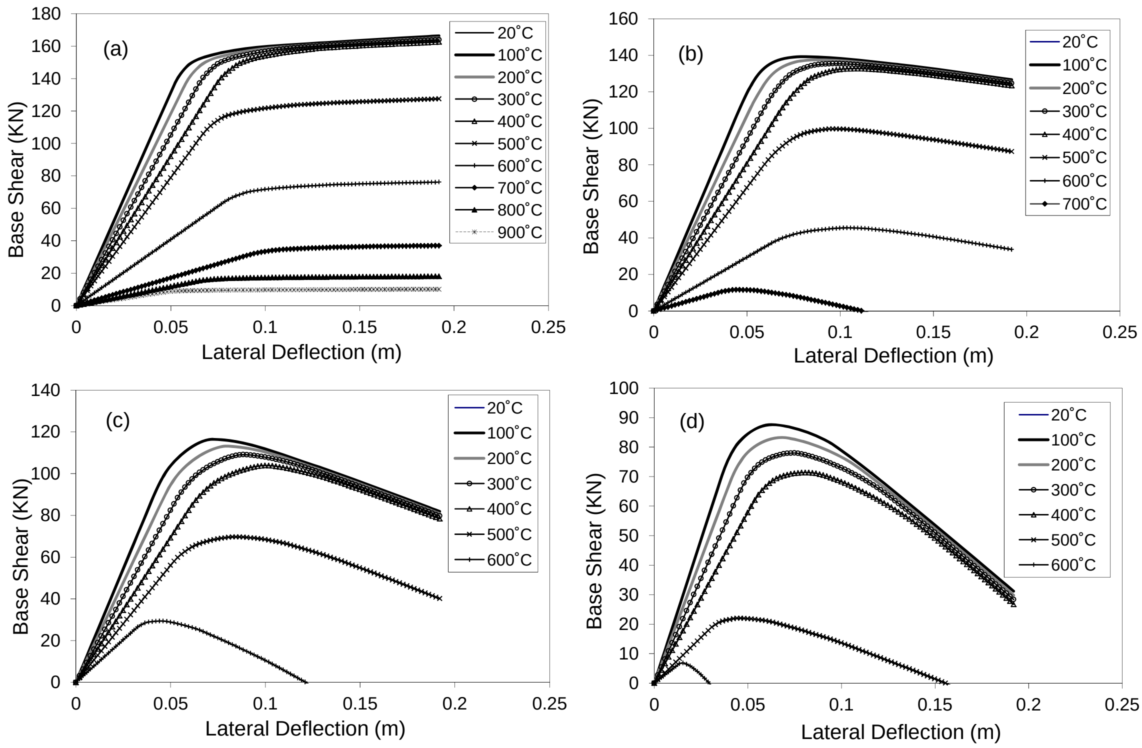
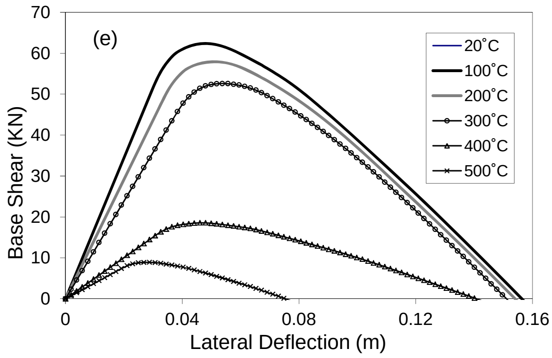


Fig. 8 Lateral load-deflection behaviour of HSS 254×254×16 column at different axial load levels: (a) No axial load; (b) Axial load = 0.2 × axial load capacity; (c) Axial load = 0.4 × axial load capacity; (d) Axial load = 0.6 × axial load capacity; (e) Axial load = 0.8 × axial load capacity; (f) Axial load = axial load capacity



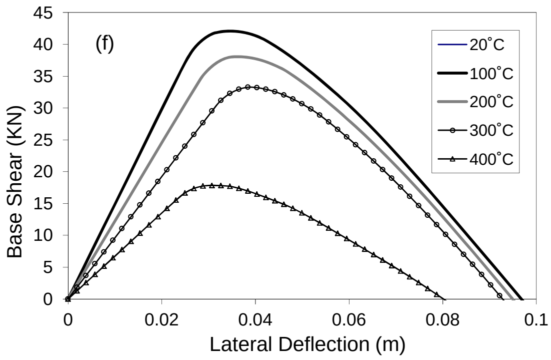


Fig. 8 Continued

400°C. It was also observed that the ductility decreased with the increase in the axial load level for the same temperature. This decrease in ductility due to the increase in axial load was as high as 46% on an average.

Similar behaviour was also observed for the stiffness of the columns with temperatures at different axial load levels (Table 4). It was observed that with the increase in temperature, the stiffness of both columns decreased sharply. At 400°C and 1000°C, the stiffness of the columns decreased by 27% and 95%, respectively (on an average), compared to the stiffness at the ambient temperature, when there was no axial load on the columns. On the other hand, in presence of axial loads equal to 20%, 40%, 60%, 80% and 100% of the axial load capacity, the reduction in the stiffness of the columns were 32%, 35%, 40%, 58%, and 64% (on an average), respectively at

Table 3 Variation of ductility of HSS columns with temperature at different axial load levels

Temp. (°C)	Ductility											
	HSS 254×254×16						HSS 305×305×16					
	Axial load level compared to its capacity						Axial load level compared to its capacity					
	0	0.2	0.4	0.6	0.8	1.0	0	0.2	0.4	0.6	0.8	1.0
20	3.49	1.64	1.46	1.44	1.41	1.40	3.84	1.74	1.45	1.44	1.38	1.38
100	3.49	1.64	1.46	1.44	1.41	1.40	3.84	1.74	1.45	1.44	1.38	1.38
200	3.20	1.49	1.45	1.33	1.32	1.16	3.56	1.45	1.42	1.30	1.26	1.16
300	2.74	1.39	1.42	1.30	1.29	1.15	3.20	1.27	1.35	1.28	1.15	1.13
400	2.55	1.36	1.32	1.27	1.13	1.14	2.82	1.25	1.20	1.18	1.08	1.08
500	2.4	1.25	1.20	1.17	1.09	-	2.80	1.23	1.10	1.00	-	-
600	2.13	1.23	1.18	1.00	-	-	2.74	1.23	-	-	-	-
700	1.90	1.05	-	-	-	-	2.46	1.00	-	-	-	-
800	1.33	-	-	-	-	-	1.0	-	-	-	-	-
900	1.00	-	-	-	-	-	1.0	-	-	-	-	-
1000	1.00	-	-	-	-	-	1.0	-	-	-	-	-

Table 4 Variation of stiffness of H columns with temperature at different axial load levels

Temp. (°C)	Stiffness/Stiffness at ambient temperature with no axial load (%)											
	HSS 254×254×16						HSS 305×305×16					
	Axial load level compared to its capacity						Axial load level compared to its capacity					
	0	0.2	0.4	0.6	0.8	1.0	0	0.2	0.4	0.6	0.8	1.0
20	100%	91%	83%	74%	65%	57%	100%	98%	91%	84%	77%	69%
100	100%	91%	83%	74%	65%	57%	100%	98%	91%	84%	77%	69%
200	90%	81%	77%	64%	53%	47%	95%	88%	80%	73%	66%	59%
300	80%	71%	63%	54%	45%	37%	84%	77%	70%	63%	56%	47%
400	70%	61%	53%	44%	26%	19%	74%	67%	59%	52%	47%	38%
500	60%	51%	43%	40%	14%	-	63%	56%	49%	42%	34%	27%
600	31%	22%	20%	19%	-	-	33%	25%	18%	11%	-	-
700	13%	11%	-	-	-	-	14%	6%	-	-	-	-
800	9%	-	-	-	-	-	9%	-	-	-	-	-
900	7%	-	-	-	-	-	7%	-	-	-	-	-
1000	5%	-	-	-	-	-	5%	-	-	-	-	-

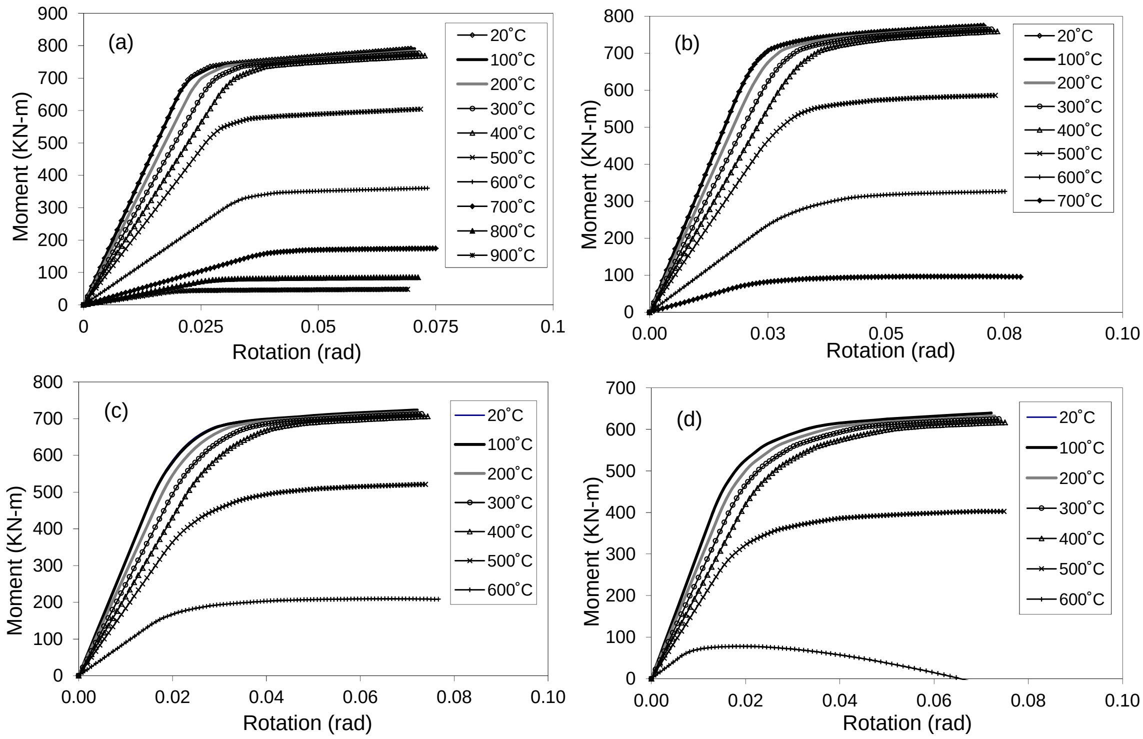


Fig. 9 Moment-rotation behaviour of HSS 305×305×16 column at different axial load levels: (a) No axial load; (b) Axial load = 0.2 × axial load capacity; (c) Axial load = 0.4 × axial load capacity; (d) Axial load = 0.6 × axial load capacity; (e) Axial load = 0.8×axial load capacity; (f) Axial load = axial load capacity

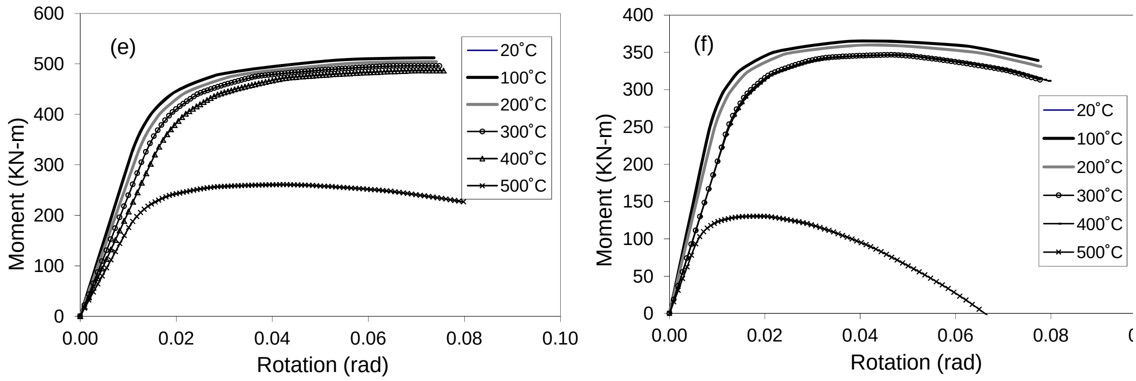


Fig. 9 Continued

400°C. It was also observed that the stiffness decreased with the increase in the axial load level for the same temperature. This decrease in stiffness due to the increase in axial load was as high as 60% on an average.

4.3 Moment-rotation relationship

Figs. 9 and 10 show the moment-rotation behaviour of HSS 305×305×16 and HSS 254×254×16 columns, respectively for different temperatures at various axial load levels. It was observed that the moment-rotation behaviour of steel columns at different axial load levels was significantly influenced by higher temperatures. The moment-rotation behaviour of columns declined with increase in temperature due to the degradation of the material properties of steel with increase in temperature at each axial load level. It was noted that up to a temperature of 400°C, the moment-

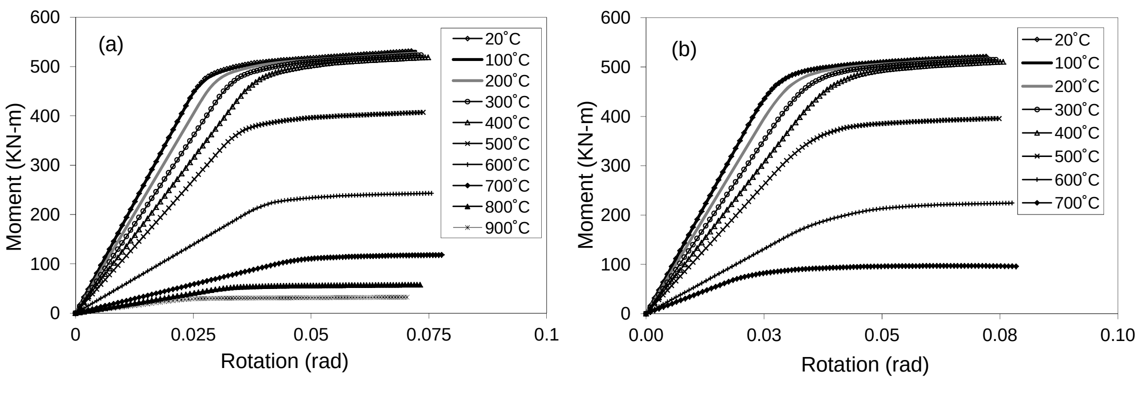


Fig. 10 Moment-rotation behaviour of HSS 254×254×16 column at different axial load levels: (a) No axial load; (b) Axial load = 0.2×axial load capacity; (c) Axial load = 0.4×axial load capacity; (d) Axial load = 0.6×axial load capacity; (e) Axial load = 0.8×axial load capacity; (f) Axial load = axial load capacity

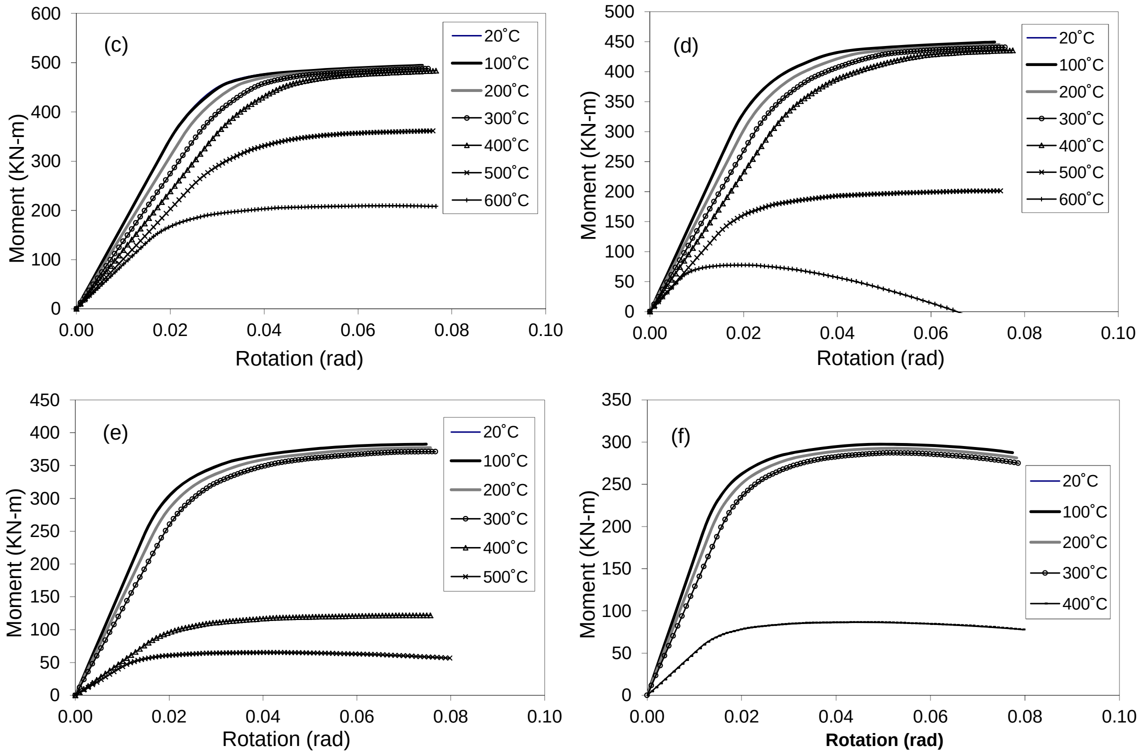


Fig. 10 Continued

rotation behaviour of the steel columns were quite close to each other (within 3%), but after that temperature, there was a significant reduction in moment capacity of the columns. This can be explained by the fact that up to 400°C, the effective yield strength of steel is the same, whereas the modulus of elasticity reduces by 30% (Fig. 4).

For HSS 254×254×16 column with no axial load, the moment capacity was 527.4 kN.m at ambient temperature. At 400°C the moment capacity reduced to 519 kN.m, which indicated a reduction in the moment capacity by only 1.6%, whereas at 500°C the moment capacity decreased to 407.1 kN.m, i.e., a 23% reduction in the moment capacity. When the column was subjected to 900°C, the moment capacity became only 32.2 kN.m, which was only 6% of the original moment capacity. Similar behaviour was also observed at different axial load levels (Fig. 10). Similar behaviour of HSS 305×305×16 column was also observed under increasing temperature (Fig. 9).

4.4 Moment and lateral load capacity

Fig. 11 shows the variation of the moment capacity of the columns with temperature and axial load levels. For all of the columns, it was evident that with the increase in the axial loads, the moment capacity of the columns decreased at the same temperature, and also, the columns could not withstand higher temperature in presence of higher axial loads. For example, HSS 305×305×16 column could resist 604 kN.m of moment at 500°C, when there was no axial load on the column. When the column was subjected to an axial load equal to the axial capacity of the column at the

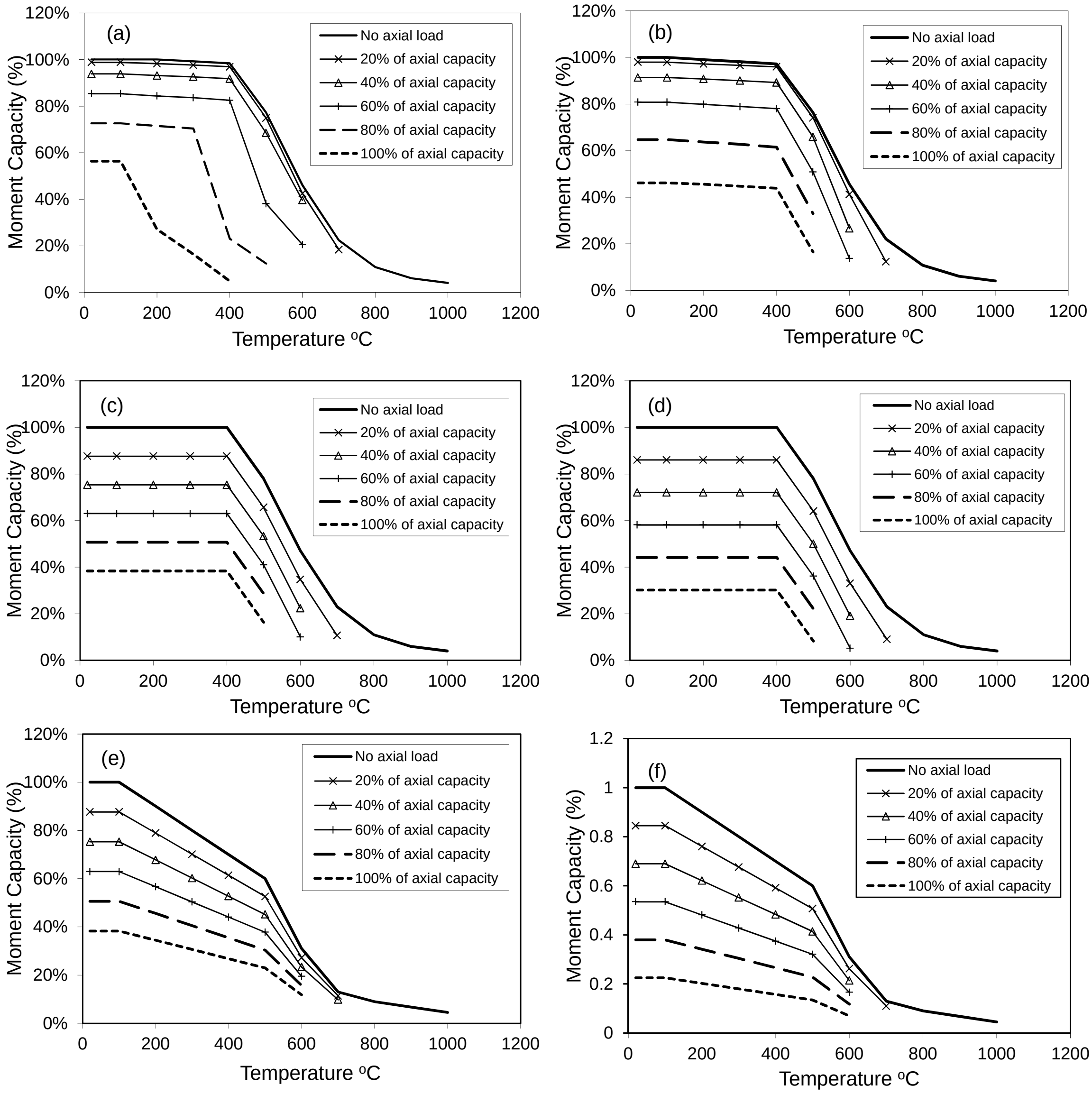


Fig. 11 Moment capacity of columns with temperature at different axial load levels: (a) HSS 254×254×16; (b) HSS 305×305×16; (c) HSS 254×254×8; (d) HSS 305×305×8; (e) HSS 254×254×6.4; (f) HSS 305×305×6.4

same temperature, the column could resist only 130.4 kN.m moment. This indicated about 78% decrease in the moment capacity of the column. From the comparison of behaviour of different classes of section it can be concluded that for Class 1 and Class 3 sections there is not any rapid decrease in moment capacity up to 400°C, however, in the case of Class 4 section moment capacity decreases rapidly after 100°C. It was also observed that with the increase in the axial loads, the moment capacities of the Class 3 and Class 4 sections decrease more rapidly compared to that of Class 1 section. For example, with the increase in axial load from 0% to 40% of the capacity, the moment capacity of HSS 305×305×16 (Class 1) column reduces by only 9%, whereas, the reduction in moment capacity for HSS 305×305×8 column (Class 3) is 28% and for HSS 305×305×6.4 column (Class 4) is 31%.

Figs. 12 and 13 show the variation of the moment and the lateral load capacity of the non-insulated columns with fire exposure time at different axial load levels, respectively. From Fig. 13 it was observed that the effect of axial force in the lateral load capacity of columns varies

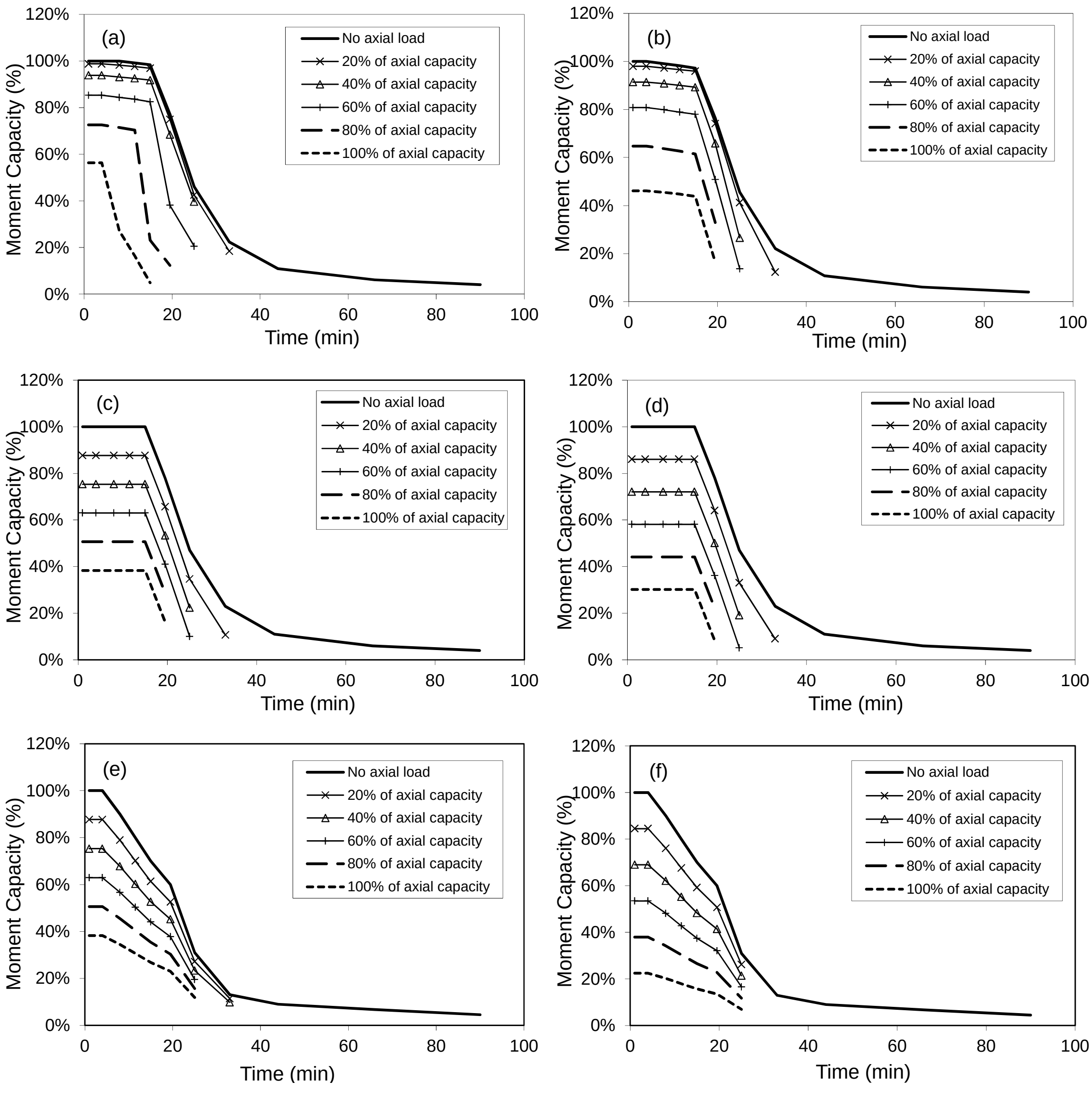


Fig. 12 Moment capacity of columns with fire exposure time at different axial load levels: (a) HSS 254×254×16; (b) HSS 305×305×16; (c) HSS 254×254×8; (d) HSS 305×305×8; (e) HSS 254×254×6.4; (f) HSS 305×305×6.4

for different classes of sections. For example at room temperature, for an increase in axial load from 0% to 40% of axial load capacity, the lateral load capacity of HSS 254×254×16 (Class 1) column reduced from 100 percent to 94 percent while the lateral load capacity of both of the HSS 254×254×8 (Class 3) and HSS 254×254×6.4 (Class 4) columns reduced from 100 percent to 75 percent. From both of the Figs. 12 and 13 it was observed that with the increase in fire exposure time and axial load, the moment capacity and the lateral load capacity of the non-insulated columns dropped off significantly. It is evident that when there was no axial load on the columns,

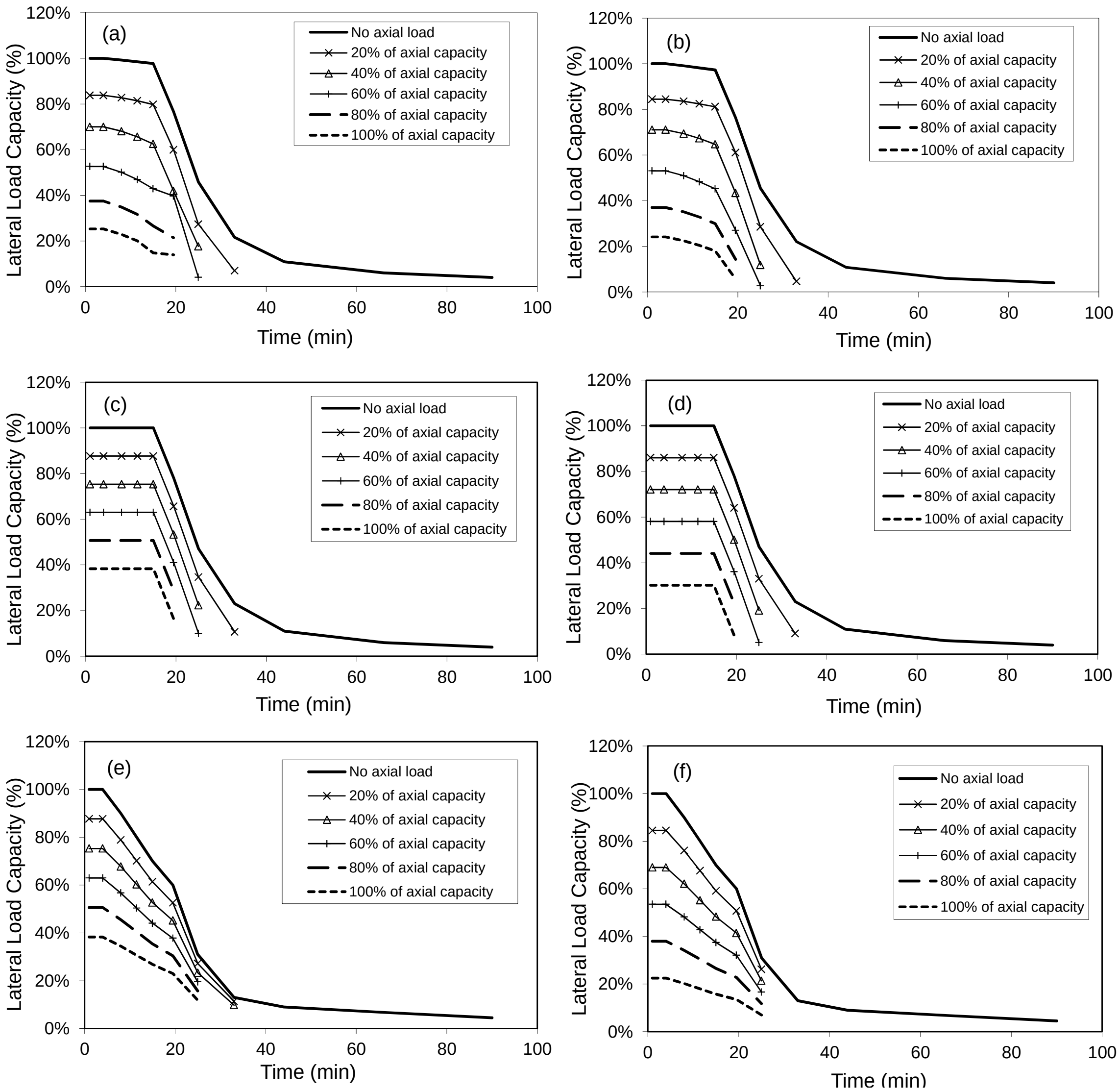


Fig. 13 Lateral load capacity of columns with fire exposure time at different axial load levels: (a) HSS 254×254×16; (b) HSS 305×305×16; (c) HSS 254×254×8; (d) HSS 305×305×8; (e) HSS 254×254×6.4; (f) HSS 305×305×6.4

they could withstand the fire exposure for a long time before failure, whereas with the increase in the axial load levels, the columns could withstand the imposed stresses for lesser amount of fire exposure time. It was also observed that Class 1 and Class 3 sections can sustain fire for a longer time than Class 4 section. For example Class 1 and Class 3 sections can withstand fire without reduction in capacity for 15 minute but for Class 4 section there is a drastic reduction in the capacity of the column only after 4 minutes of exposure to fire. This indicates that with the increase in fire exposure time and axial load levels, the moment capacity and the lateral load capacity of the columns decrease significantly and hence, proper fire insulating material should be used as coating on the steel columns to help the columns withstand the imposed forces for a longer fire exposure time.

5. Conclusions

The present study investigates the performance of unprotected HSS steel columns at elevated temperatures and thus, assesses their behaviour during a fire event. The results of the study lead to the following conclusions.

- The lateral load-deflection behaviour and the moment-rotation behaviour of steel columns at temperatures greater than 400˚C get significantly affected due to the degradation of the material properties of steel at elevated temperature. It was observed that the lateral load capacity and the moment capacity get reduced by more than 20% and 90% at temperatures of 500°C and 900°C, respectively.
- There is a significant reduction in the ductility and the stiffness of steel columns at elevated temperature due to fire conditions. The reduction in ductility and stiffness can be as high as 46% and 60% compared to the ductility and stiffness of the columns at the ambient temperature.
- With the increase in axial loads in columns, the moment capacity and the lateral load capacity decrease at the same temperature, and also, the columns cannot withstand higher temperature in presence of higher axial loads.
- The effect of axial load in the moment capacity and lateral load capacity of non-insulated columns is different for different classes of sections.
- Class 3 and Class 4 sections exhibit a rapid reduction in moment and lateral load capacity with increasing axial load level than Class 1 and Class 2 sections.
- The degradation in moment capacity and lateral load capacity of the steel columns are mainly governed by the degradation of material properties of steel with increasing temperature and the geometric properties of the column section.
- Fire exposure time plays an important role in the failure of steel columns during fire events. It has been noted that Class 1 and Class 3 sections at different axial load levels can withstand fire exposure for 15 minutes without any drastic reduction in moment or lateral load capacity, whereas for Class 4 section there was a drastic reduction in moment and lateral load capacity only after 4 minutes of fire exposure.

Based on the results of this study, it can be concluded that during fire condition, there is a significant reduction in the ductility, stiffness, and the lateral load capacity and the moment capacity of steel columns, which may cause failure at an axial load level far below the axial load capacity of the columns. This reduction in capacity, ductility and stiffness of the columns must be accounted for during the design of steel columns and proper insulation should be provided to increase the fire rating so that it will provide adequate time before failure and will allow occupants for safe exit from the building. Most of the current design guidelines for steel frames recommend passive protection against fire for steel sections. Since, the present study considered only the performance of unprotected steel columns, further study is necessary to develop the performance curves for protected steel columns with various fire retardant coatings.

Acknowledgments

The financial contribution of the Natural Sciences and Engineering Research Council (NSERC) of Canada through ENGAGE program was critical to conduct this research and has been gratefully acknowledged.

References

Alam, M.S., Youssef, M.A. and Nehdi, M. (2008), "Analytical prediction of the seismic behaviour of superelastic shape memory alloy reinforced concrete elements", *Eng. Struct.*, **30**(12), 3399-3411.

Alam, M.S., Youssef, M.A. and Nehdi, M. (2009), "Seismic performance of concrete frame structures reinforced with superelastic shape memory alloys", *Smart Struct. Syst., Int. J.*, **5**(5), 565-585.

Ali, F. and O'Connor, D. (2001), "Structural performance of rotationally restrained steel columns in fire", *Fire Safety J.*, **36**(7), 679-691.

Ali, F., Nadjal, A., Silcock, G. and Abu-Tair, A. (2004), "Outcomes of a major research on fire resistance of concrete columns," *Fire Safety J.*, **39**(6), 433-445.

Billah, A.H.M.M. and Alam, M.S. (2013), "Performance-based prioritization for seismic retrofitting of RC bridge bents", *Struct. Infrastruct. Eng.* DOI: 10.1080/15732479.2013.772641 [In Press]

Cai, J. and Feng, J. (2010), "Thermal buckling of rotationally restrained steel columns", *J. Construct. Steel Res.*, **66**(6), 835-841.

Choe, L., Varma, A.H., Agarwal, A. and Surovek, A. (2011), "Fundamental behaviour of steel beam-columns and columns under fire loading: Experimental evaluation", *ASCE J. Struct. Eng.*, **137**, 954-966.

CISC 2006, *Handbook of Steel Construction*, (9th Edition), Canadian Institute of Steel Construction (CISC), Toronto, ON, Canada.

CSA S16 (2009), *Design of Steel Structures*, Canadian Standards Association, Mississauga, ON, Canada.

Culver, C. (1972), "Steel column buckling under thermal gradients", *ASCE J. Struct. Eng.*, **92**(8), 1853-1865.

European Committee for Standardisation (CEN) (2004), Eurocode 3, *Design of Steel Structures: Part 1.2: General Rules*, "Structural fire design", Brussels.

Franssen, J.M. and Dotreppe, J.C. (1992), "Fire resistance of columns in steel frames", *Fire Safety J.*, **19**(2-3), 159-175.

Franssen, J.M., Schleich, J.B. and Cajot, L.G. (1995), "A simple model for the fire resistance of axially-loaded members according to Eurocode 3", *J. Construct. Steel Res.*, **35**(1), 49-69.

Franssen, J.M., Schleich, J.B., Cajot, L.G. and Azpiaxu, W. (1996), "A simple model for the fire resistance of axially loaded members-comparison with experimental results", *J. Construct. Steel Res.*, **37**(3), 175-204.

Franssen, J.M., Talamona, D., Kruppa, J. and Cajot, L.G. (1998), "Stability of steel columns in case of fire: Experimental evaluation", *ASCE J. Struct. Eng.*, **124**(2), 158-163.

Gardner, L., Insausti, A., Ng, K.T. and Ashraf, M. (2010), "Elevated temperature material properties of stainless steel alloys", *J. Construct. Steel Res.*, **66**(5), 634-647.

Heva, Y.B. and Mahendran, M. (2013), "Flexural-torsional buckling tests of cold-formed steel compression members at elevated temperatures", *Steel Compos. Struct., Int. J.*, **14**(3), 205-227.

ISO (International Standards Organization) (1980), ISO 834: *Fire Resistance Tests, Elements of Building*

Construction, Switzerland.
Kodur, V.K.R. (1998), "Performance-based fire resistance design of concrete-filled steel columns", *J. Construct. Steel Res.*, **51**(1), 21-36.
Kodur, V.K.R. and Lie, T.T. (1997), "Evaluation of fire resistance of rectangular steel columns filled with fibre-reinforced concrete", *Can. J. Civil Eng.*, **24**(3), 339-349.
Lie, T.T. (1978), "Fire resistance of structural steel", *Eng. J.*, American Institute of Steel Construction, 116-125.
Lie, T.T. (1994), "Calculation of the fire resistance of steel hollow structural section columns filled with plain concrete", *Can. J. Civil Eng.*, **21**(3), 382-385.
Neves, I.C., Valente, J.C. and Correia Rodrigues, J.P. (2002), "Thermal restraint and fire resistance of columns", *Fire Safety Journal*, **37**(8), 753-771.
Ossenbruggen, P., Aggarwal, V. and Culver, C. (1973), "Steel column failure under thermal gradients", *ASCE J. Struct. Eng.*, **99**(4), 727-739.
Outinen, J. (2007), "Mechanical properties of structural steels at high temperatures and after cooling down", Doctoral Dissertation, Helsinki University of Technology, TKK-TER-32, Helsinki.
PEER (2010), http://nisee2.berkeley.edu/peer/prediction_contest. Accessed on September 5, 2011.
Poh, K.W. and Bennetts, I.D. (1995a), "Analysis of structural members under elevated temperature conditions", *ASCE J. Struct. Eng.*, **121**(4), 664-675.
Poh, K.W. and Bennetts, I.D. (1995b), "Behaviour of steel columns at elevated temperatures", *ASCE J. Struct. Eng.*, **121**(4), 676-684.
Poh, K.W. (1998), "Behaviour of load-bearing members in fire", Ph.D. Thesis, Monash University, Clayton.
Seismostruct, V5.0.5 (2010), www.seismosoft.com
Rodrigues, J.P.C., Cabrita Neves, I. and Valente, J.C. (2000), "Experimental research on the critical temperature of compressed steel elements with restrained thermal elongation", *Fire Safety J.*, **35**(2), 77-98.
Takagi, J. and Deierlein, G.G. (2007), "Strength design criteria for steel members at elevated temperatures", *J. Construct. Steel Res.*, **63**(8), 1036-1050.
Talamona, D., Franssen, J.M., Schleich, J.B. and Kruppa, J. (1997), "Stability of steel columns in case of fire: Numerical modeling", *ASCE J. Struct. Eng.*, **123**(6), 713-720.
Tang, C.Y., Tan, K.H. and Ting, S.K. (2001), "Basis and application of a simple interaction formula for steel columns under fire conditions", *ASCE J. Struct. Eng.*, **127**(10), 1206-1213.
Tao, Z., Wang, Z.B., Han, L.H. and Uy, B. (2011), "Fire performance of concrete-filled steel tubular columns strengthened by CFRP", *Steel Compos. Struct., Int. J.*, **11**(4), 307-324.
Toh, W.S., Tan, K.H. and Fung, T.C. (2000), "Compressive resistance of steel columns in fire: The Rankine approach", *ASCE J. Struct. Eng.*, **126**(3), 398-405.
Twilt, L. (1991), TNO-report BI-91-015: Stress-strain relationships of structural steel at elevated temperatures: Analysis of various options & European proposal, Delft, TNO.
Wang, W.Y. and Li, G.Q. (2009), "Behaviour of steel columns in a fire with partial damage to fire protection", *J. Construct. Steel Res.*, **65**(6), 1392-1400.
Yang, K.C. and Yu, Z.H. (2013), "Experimental research on the creep buckling of fire-resistant steel columns at elevated temperature", *Steel Compos. Struct., Int. J.*, **15**(2), 163-173.
Yang, K., Chen, S. and Ho, M. (2009), "Behaviour of beam-to-column moment connections under fire load", *J. Construct. Steel Res.*, **65**(7), 1520-1527.
Yang, K. and Hsu, R. (2009), "Structural behaviour of centrally loaded steel columns at elevated temperature", *J. Construct. Steel Res.*, **65**(10-11), 2062-2068.
Yang, K.C., Lee, H.H. and Chan, O. (2006a), "Performance of steel H columns loaded under uniform temperature", *Journal of Constructional Steel Research*, **62**(3), 262-270.
Yang, K.C., Lee, H.H. and Olen, C. (2006b), "Experimental study on fire-resistant steel H columns subjected to fire load", *J. Construct. Steel Res.*, **62**(6), 544-553.

CC