Experimental Study on Ultimate Strength of Steel Columns under Fire Load

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Abstract

During the past decade, researches on the behavior of steel columns at elevated temperature have been carried out experimentally and theoretically. Most of the works were focus on the critical temperature of steel columns under specified service load or the fire resistance duration of protected steel columns in fire conditions. The knowledge related to the ultimate strength of unprotected steel columns is limited. With the advancement of metal production and environmental concern, a new type of structural steel, Fire-resistant steel (FRS), has been developed to conquer inherent weakness of conventional steel at elevated temperature. The requirement of fire-protection used in the fire-resistant steel columns can be reduced or even lifted as compared to conventional steel columns. In order to adopt the fire resistance design of steel structures into the performance-based design, the ultimate strength of steel columns at different temperature levels is needed. A series of steel columns, including the columns made from conventional steel and fire-resistant steel, was loaded to fail under specified temperature to investigate the structural behavior of steel columns under fire conditions. This is attempted to establish the reduction effects of column strength resulting in the increasing temperature. Based on the research results, a design guideline is proposed to determine the buckling curves of steel columns under fire events.

1. Introduction

With the growing popularity of steel structures adopted in high rise buildings, the fire safety of the steel structures becomes one of the main issues in recent years, especially after the collapse of World Trade Center in New York. Because columns are critical load-carrying members in a structural system, research works on the behavior of steel columns at elevated temperature have been carried out experimentally and theoretically. Some researchers [1~7] examined the effect of

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boundary condition in the fire resistance of steel columns through experimental fire tests and numerical analysis. Some [8~10] proposed a model describing the load capacity of steel column during fire. However, most of the previous works were focus on determining the critical temperature of steel columns under specified service load. The knowledge related to the ultimate load-carrying capacity of unprotected steel columns at elevated temperature is limited.

Normally, fire-protection material need to be used for steel columns in order to meet the fire resistance duration required in the specifications. The fire-protection material is recommended to be reduced or excluded with the environmental concern. To conquer inherent weakness of conventional steel at elevated temperature and to solve the population problem induced by the fire-protection material, a new type of structural steel, Fire-resistant steel (FRS), has been developed. Fire test has been conducted to prove Fire-resistant steel have better performance than conventional structural steel at elevated temperature [11~12]. The requirement of fire-protection in the fire-resistant steel can be released or relaxed as compared with conventional steel structures. However, there is a lack of knowledge on the ultimate load-carrying capacity of unprotected fire-resistant steel column at elevated temperature.

In traditional steel structures design at room temperature, the ultimate strength of load-carrying members is determined first, either by experimental or analytical approach. The resistance of steel members is derived by multiplying a reduction factor. Such as, the safety factor (F.S.) used in the allowable stress design (ASD) and the safety index (β) adopted in the limit state design (LSD) or load and resistant factor design (LRFD). In the concept of performance-based design, designer is allowed to have more freedom to select the allowable loads corresponding to different temperature levels. In order to adopt the fire-resistant design of steel structures into the traditional design or performance-based design, there is an urgent need to determine the ultimate strength of steel columns at different temperature levels.

In this study, a total of 15 steel columns were tested under different temperature levels to investigate the structural behavior of steel columns in fire conditions. Among these specimens, 11 specimens were hot-rolled steel columns made from conventional structural steel, SM 490 or A36. Others were welded by newly developed fire-resistant steel. Based on the experimental results, a simple model is proposed to determine the ultimate strength of steel columns under fire events. The mechanical properties of steel at elevated temperature are also examined because they are the more relevant factors affecting the behavior of steel columns under fire events.

2. Experimental Study

2.1 Material properties of structural ste el at elevated temperature

Tensile tests were performed at different temperature levels to examine the material properties of fire-resistant steel and conventional structural steel, SM490, in fire conditions. In the tensile test, the specimens were loaded after specimens reaching the steadily specific temperature levels. The material properties of steels at different elevated temperature were determined from the recorded stress-strain curves. The 0.2% offset method was adopted to determine the yielding strength at each elevated temperature. The material properties derived from the test results are illustrated in Table 1. The nominal yielding strength of both the fire-resistant steel and conventional steel used in this study are 343 MPa at room temperature. The comparison of the reduction factor of yielding strength between the tensile tests and those of major design specifications is listed in Table 2 and also shown in Figure 1.

The yielding strength of conventional steel derived on the basis of the test results is quite different from those specified in Eurocode 3 [13] up to temperature level of 500^{o}C. The yielding strength of conventional steel has been overestimated in the Eurocode 3 for temperature below 500^{o}C. The yielding strength of conventional steel fit well to the values specified in Eurocode 3 at higher temperatures. While, the values specified in Eurocode 3 fit well the yielding strength of fire-resistant steel made in Japan. It is noted that the means adopted to determine the yielding strength of steel are different from each specifications or researches, as indicates Table 2. The higher the offset strain used, the higher the yielding strength is. The difference between the 0.5% offset strain and 2.0% offset strain is up to 20%, as calculated in BS code [13]. A generally accepted method to calculate the yielding strength of structural steel at elevated temperature is needed as the basis to evaluate the performance of steel structure at fire events.

Table 1

Material Properties of fire-resistant steel (FR-steel) and conventional steel (SM490) at elevated temperatures

T(^{o}C)	FR-steel		SM490	
	Yielding strength (MPa)	Young's Modulus (MPa)	Yielding strength (MPa)	Young's Modulus (MPa)
Room	393	208938	417	228502
100	386	231313	389	218410
200	365	212094	357	181753
300	306	236187	296	172173
400	285	171023	279	171677
500	260	198312	238	151103
600	200	153106	185	137122
700	114	70880	98	110116

Table 2 Reduction Factor of yielding strength at elevated temperature [13~14]

T(^{o}C)	This study		Japan		Eurocode 3 (2.0%)	ECCS (0.5%)	BS	
	SM 490 (0.2%)*	FR-steel (0.2%)	Con.Steel (0.2%)	FR-steel (0.2%)			(0.5%)	(2.0%)
Room	1.00	1.00	1.00	1.00	1.00	0.99	1.00	1.00
100	0.93	0.95	1.00	1.01	1.00	0.95	0.97	1.00
200	0.86	0.79	1.00	1.07	1.00	0.88	0.95	1.00
300	0.71	0.74	1.00	0.99	1.00	0.78	0.85	1.00
400	0.67	0.67	0.78	0.98	1.00	0.65	0.80	0.97
500	0.57	0.65	0.56	0.83	0.78	0.48	0.62	0.78
600	0.44	0.52	0.33	0.54	0.47	0.27	0.38	0.47
700	0.23	0.29	0.11	0.26	0.23	0.16	0.19	0.23

Note: *inside the ( ) represents the strain offset for determining the yielding strength.

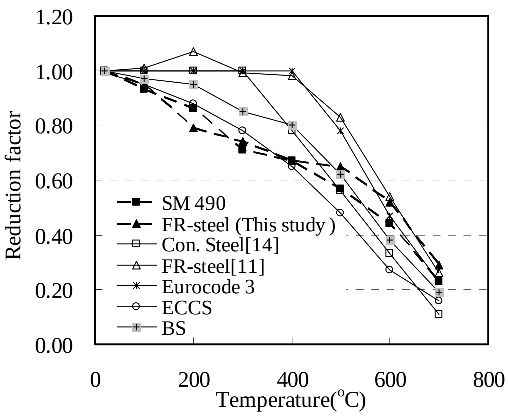


Figure 1 Reduction factors of yielding st rength of steel at elevated temperature

2.2 Experimental tests plan

A total of 15 steel columns were loaded to failure at specified temperatures to examine the structural behavior in fire conditions. Among these column specimens, 7 out of 15 are hot-rolled columns, made from conventional steel, SM 490. Four of them are welded by fire-resistant steel columns. The nominal yielding strength of these two types of steel is 343 MPa at ambient temperature. The rest four specimens are hot-rolled steel columns, made from ASTM A36 conventional steel, with nominal yielding strength of 245 MPa at ambient temperature. The dimensions of the specimens are listed in Table 3. All the specimens are classified as non-compact section according to the regulations of Limit State Design specifications [16].

The specimen was heated up to the specific temperature and then axial load was applied. The temperature selected in this study varied from room temperature up to 600°C, as listed in Table 3. After reaching its ultimate strength, the test was stopped when the strength decreased to 70% of the ultimate load. During the heating process, the specimen expanded freely without any restraint to its thermal expansion. To monitor the temperature distribution of the stub columns and to ensure the temperature of the specimens remains steadily at desired level, thermocouples were installed on each specimen, following BS476 regulation [15]. Figure 2 shows the test device in this study.

Table 3 Dimensions of test specimens

Dimensions of specimens	L(cm)	T(°C)
H 300×300×10×15 Conventional steel, SM490, $F_{y,nominal}$=343 MPa	100	Room
	100	300
	100	400
	100	450
	100	500
	100	550
	100	600
H 200×200×10×10 Fire-resistant steel, $F_{y,nominal}$=343 MPa	60	Room
	60	400
	60	500
	60	600
H 175×175×7.5×11 Conventional steel, A36, $F_{y,nominal}$=245 MPa	130	Room
	130	400
	130	500
	130	600

Figure 2 test device

3. Discussions of test results

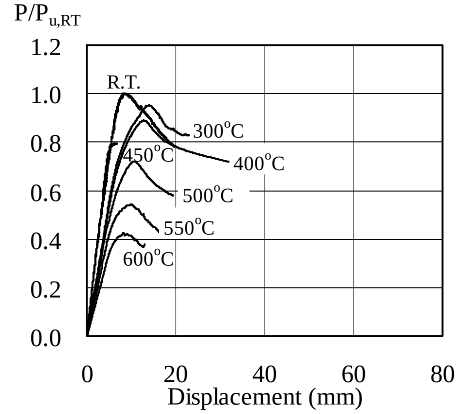
3.1 Structural behavior of steel columns at elevated temperature

Figure 3 shows the normalized load-displacement curves of column specimens at elevated temperature, where (a) is the results derived from H 300×300×10×150; (b) represents the behavior of fire-resistant steel H200×200×10×10; and (c) is the behavior of H175×175×7.5×11.

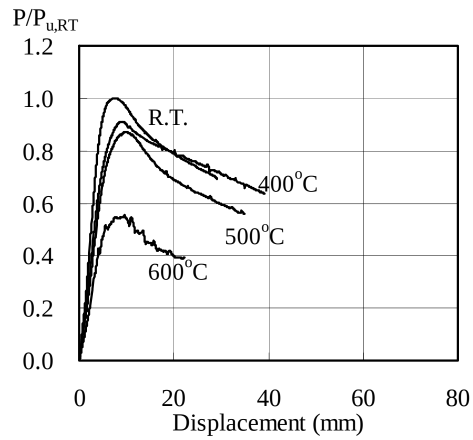
The normalized load-displacement curves decrease with temperature increases. When temperature is equal to or less than 500°C, the reduction of ultimate strength is less than 30% of the strength at ambient temperature. However, the load-carrying capacities of the steel columns decrease significantly as temperature reaches 600°C. There is approximately 30% of the ultimate strength left as temperature reach 600°C for column specimen H175×175 ×7.5×11. Among three different specimen series, the temperature effects in the strength reduction of specimens H200×200×10×10 is less than others. This is the advantage of the steel columns made from fire-resistant steel.

The column specimens H175×175×7.5 ×11 perform more ductile behavior either at room temperature or at elevated temperature, compared to other two series of specimens. This is mainly because the inherent properties of the steels. That is, the higher the material strength, the less the ductility.

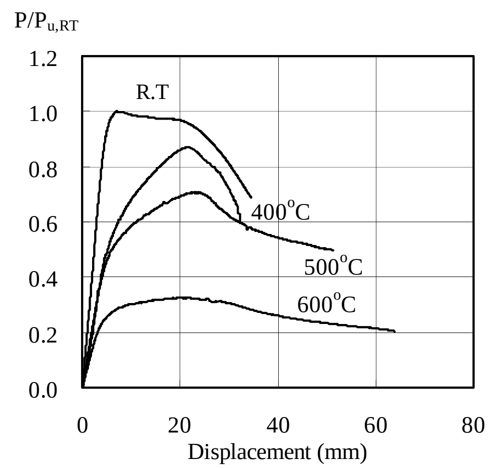
It is observed that the failure mode of column specimens H175×175×7.5×11 is different when loaded at room temperature and at elevated temperature. Columns specimens H175×175×7.5×11, with slenderness ratio of 30, is designed as inelastic buckling columns at room temperature. However, instead of inelastic global buckling, the column specimens failed due to local buckling at elevated



(a). H 300×300×10×15



(b) H 200×200×10×10



(c). H 175×175×7.5×11

Figure 3 Load-displacement curves of column specimens at elevated temperature

temperature. Figure 4 compares the failure modes of these specimens. At elevated temperature, the release of residual stresses [12] increases the inelastic buckling strength of steel column. Consequently, the failure mechanism of the steel column changed at elevated temperature.

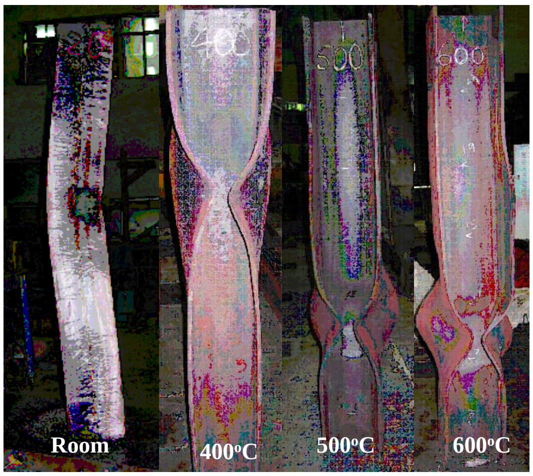


Figure 4 Failure mode of H 175×175×7.5×11 specimens

3.2 Ultimate strength of steel columns at elevated temperature

The reduction factors of ultimate strength of column specimens at elevated temperature is shown in Figure 5 and also listed in Table 4. The ultimate strength of column specimen decreases with temperature increases. The ultimate strength drops significantly from temperature 500 ^{o}C to 600 ^{o}C, as shown in Figure 5. The reduction factors of the ultimate strength are almost the same between the column specimens made from two different conventional steels. While, the reduction factors are higher for those specimens made from fire-resistant steel. At temperature level of 600^{o}C, the reduction factors of specimens made from fire-resistant steel are 12% and 22% higher than the specimen made from conventional steel of SM 490 and A36, respectively.

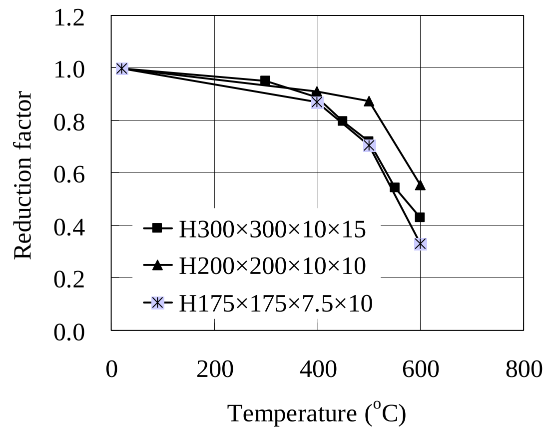


Figure 5 Reduction factors of ultimate strength of column specimens at elevated temperature

Table 4 Reduction factors of ultimate strength of column specimens at elevated temperature

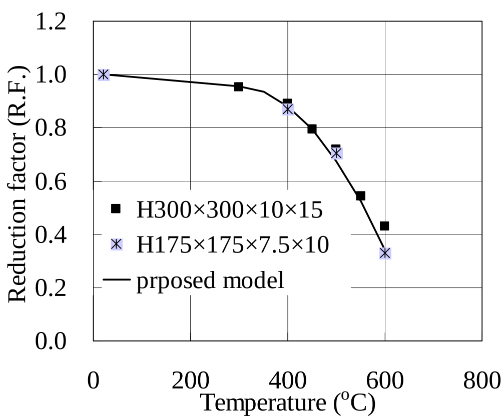
Dimensions of specimens	T(^{o}C)	Reduction factors
H 300×300×10×15 Conventional steel, SM490, $F_{y,nominal}$=343 MPa	Room	1.00
	300	0.95
	400	0.89
	450	0.80
	500	0.72
	550	0.54
	600	0.43
H 200×200×10×10 Fire-resistant steel, $F_{y,nominal}$=343 MPa	Room	1.00
	400	0.91
	500	0.87
	600	0.55
H 175×175×7.5×11 Conventional steel, A36, $F_{y,nominal}$=245 MPa	Room	1.00
	400	0.87
	500	0.71
	600	0.33

3.3 Design recommendation of steel columns at elevated temperature

Based on the experimental results, it is found that the fire resistance of steel compression members at elevated temperature seems to be overestimated by the non-dimensional parameter $\bar{\lambda}_{\theta}$ in EC3-1.2 [17] or the coefficient α in the fire limit state design of axially compression member in China [18]. Researches related to the ultimate strength of unprotected conventional steel columns at elevated temperature are needed in order to truly represent the structural behavior of steel columns in fire.

Based on the experimental results of this study, a simple design model (Eq.1) is proposed to determine the reduction factors of steel columns at elevated temperature, corresponding to the steel columns made from conventional steel. The ultimate strength of steel columns at specified temperature level can be obtained by multiplying the reduction factor to the ultimate strength derived at room temperature.

R.F.= 0.4+0.0038T(^oC)-6E-6T^2(^oC)… Eq.1



It is recommended to establish the database of the temperature effects in the strength of fire-resistant steel members, in order to adopt fire-resistant steel into the fire resistant design of steel structures and to apply fire-resistant steel in practice.

4. Conclusions

A total of 15 steel columns were loaded to fail under specified temperature in this research. The objective of this research is attempted to establish the buckling curves of steel column at elevated temperature. Based on this study, it is found that the governing failure mode of steel columns at elevated temperature can be different from that of at room temperature. More researches need to be conducted in order to establish the knowledge related to the structural behaviors of steel columns at elevated temperature. In addition, the strength reduction of steel columns made from Fire-resistant steel is less than those made from the conventional steel. Furthermore, some of the parameters used in the fire design specifications to determine the strength reduction of steel compression members in fire conditions are not conservative. It is tentatively suggested to adopt the reduction factors derived from this experimental result to determine the load-carrying capacities of steel columns in fire. A series of experimental works related to the fire resistant capacity of steel columns at elevated temperature are still ongoing to establish the buckling curves of steel columns at different temperature levels with different slenderness ratios.

5. Acknowledgement

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References

1. Y.C. Wang, J.M. Davies. An experimental study of non-sway loaded and rotationally restrained steel column assemblies under fire conditions: analysis of test results and design calculations, J. Construct. Steel Res. **2003**, 59, 291-313.
2. Y.C. Wang, J.M. Davies. Fire tests of non-sway loaded and rotationally restrained steel column assemblies, J. Construct. Steel Res. **2003**, 59, 359-383.

3. Faris Ali, David O'Connor. Structural performance of rotationally restrained steel columns in fire. Fire Saf J. **2001**, 36, 679-691.
4. J.M. Franssen, Failure temperature of a system comprising a restrained column submitted to fire. Fire Saf J. **2000**, 34, 191-207.
5. J.C. Valente, I.C. Neves. Fire resistance of steel columns with elastically restrained axial elongation and bending. J. Construct. Steel Res. **1999**, 52, 319-331.
6. I.Cabrita Neves, J.C. Valente, J.P. COrreia Rodrigues. Thermal restraint and fire resistance of columns. Fire Saf J. **2002**, 37, 753-771.
7. J.P. Correia Rodrigues, I. Cabrita Neves, J.C.Valente. Experimental research on the critical temperature of compressed steel elements with restrained thermal elongation. Fire Saf J. **2000**, 35, 77-98.
8. J.M. Franssen, J.B. Schleich, L.G. Cajot, W. Azpiazu. A simple model for the fire resistance of axially loaded members – comparison with experimental results. J. Construct. Steel Res. **1996**, 37, 175-204.
9. K.W. Poh, I. D. Bennetts, Behavior of steel columns at elevated temperatures, Journal of Structural Engineering, Vol. 121, No. 4, April **1995**, pp. 676-684.
10. J.M. Franssen, J.B. Schleich, L.G. Cajot. A simple model for the fire resistance of axially-loaded members according to Eurocode 3. J. Construct. Steel Res. **1995**, 35, 49-69.
11. Y. Sakumoto, T. Okada, T. Yamaguchi, M. Yoshida, S. Tasaka, H. Saito. Fire Resistance of Fire-Resistant Steel Columns, Journal of Structural Engineering, Vol. 120, No. 4, April **1994**, pp. 1103-1121.
12. S.J. Chen, S.T. Chen, and C.C. Fang. Structural behavior of bolted and welded fire-resistant steel in fire. Report of China Steel Company. **1999**. (In Chinese).
13. G.Q. Li. Fire design of steel structures. **1999**. (In Chinese).
14. Recommendation for fire resistant design of steel structures, JSSC, **1999**. (In Japanese).
15. BS 476-20, Fire tests on building materials and structures-part 20: Methods for determination of the fire resistance of elements of construction (general principles), **1987**.
16. Structural steel design specifications – Limit state design, **2001**. (In Chinese).
17. Leen Twilt. The new Eurocode on fire design of steel structures. Proceedings of the International Seminar on Steel Structures in Fire, Shanghai, **2001**, 241-254.
18. Guo-Qiang Li, and Guo-Biao Lou. The principles of fire resistance fore steel structures in China. Proceedings of the International Seminar on Steel Structures in Fire, Shanghai, **2001**, 29-39.