Steel and Composite Structures, Vol. 13, No. 4 (2012) 309-325
DOI: http://dx.doi.org/10.12989/scs.2012.13.4.309

Determination of limiting temperatures for H-section and hollow section columns

In-Kyu Kwon[1] and Young-Bong Kwon*[2]

[1]*Department of Fire Protection Engineering, Kangwon National University, Korea*
[2]*Department of Civil Engineering, Yeungnam University, Korea*

(Received December 02, 2011, Revised May 14, 2012, Accepted June 14, 2012)

Abstract. The risk of progressive collapse in steel framed buildings under fire conditions is gradually rising due to the increasing use of combustible materials. The fire resistance of such steel framed buildings is evaluated by fire tests. Recently, the application of performance based fire engineering makes it easier to evaluate the fire resistance owing to various engineering techniques and fire science. The fire resistance of steel structural members can be evaluated by the comparison of the limiting temperatures and maximum temperatures of structural steel members. The limiting temperature is derived at the moment that the failure of structural member results from the rise in temperature and the maximum temperature is calculated by using a heat transfer analysis. To obtain the limiting temperatures for structural steel of grades SS400 and SM490 in Korea, tensile strength tests of coupons at high temperature were conducted. The limiting temperatures obtained by the tensile coupon tests were compared with the limiting temperatures reported in the literature and the results of column fire tests under four types of loading with different load ratios. Simple limiting temperature formulas for SS400 and SM490 steel based on the fire tests of the tensile coupons are proposed. The limiting temperature predictions using the proposed formulas were proven to be conservative in comparison with those obtained from H-section and hollow section column fire tests.

Keywords: limiting temperature; fire resistance; structural steels; load ratio; tensile strength tests; column fire tests.

1. Introduction

In recent years, to satisfy the strength requirements for the construction of high-rise and long-span buildings, the use of structural steel has been an increasing trend. However, structural steel members show a rapid reduction in resistance capacity when exposed to severe fire conditions. It is crucial to meet the fire resistance standards required by building regulations (KMOCT 2002) for structural members. There are two different ways to satisfy the building regulations for fire. One way is a prescriptive method which uses building regulations or standards directly and the other technique is a performance based fire engineering method which can evaluate fire development according to building parameters such as fire load density and open area. The latter is considered a more rational method than the former (RIST 2004). In recent years, fire design trends are moving from the prescriptive design to the performance based design in line with numerical research works (Richard and Ma 2004, Saab and

* Corresponding author, Professor, E-mail: ybkwon@ynu.ac.kr

Nethercot 1991, Wang *et al.* 1995, Zalok *et al.* 2009). While the prescriptive fire design, based on building regulations and test methods are used in Korea, fire engineering design currently plays an important role in the U.K., Sweden, Germany, U.S.A., and New Zealand. Furthermore, international co-operation under the leadership of the ISO emphasizes the fire engineering design adopted in ISO TC 92 (DETER 2000, Buchanan 1994, ISO 2009).

Fire engineering design consists of the determination of the design fire and evaluation of the structural stability by using mechanical and thermal properties at high temperatures (Bwalya 2008). The evaluation of structural stability is conducted using two different methods; the first is a simple calculation method which can be executed by comparison between the limiting temperature and the maximum surface temperature of structural members. The maximum surface temperature can be obtained by using the design fire derived from the fire cell. The alternative method is an advanced calculation incorporating structural analysis results with thermal and material histories at high temperatures (NZS3404 1998, SBI 1976, Outinen and Makelainen 2004, Usmani *et al.* 2009, Park *et al.* 2010).

In this paper, in order to determine the limiting temperatures of structural members which are fabricated in Korea, a series of tensile strength tests at high temperatures and column fire tests were carried out. The H-shape, circular hollow section (CHS) and square hollow section (SHS) specimens were fabricated from structural steel plates of grade SS400 and SM490. The nominal yield and ultimate tensile stresses of SS400 are 235 MPa and 400 MPa, and those of SM490 are 315 MPa and 490 MPa, respectively. Simple formulas for the limiting temperatures of SS400 and SM490 steel based on the results of the tensile coupon tests at high temperature are proposed. These proposed limiting temperature formulas were compared with current specifications such as NZS3404 (1998) and BS5950 (1990), and fire test results for columns.

2. Limiting temperatures in current specifications

When structural steel members are exposed to a severe fire, the distance between the molecules of steel structures gradually increases. This causes deformation and reduction of load bearing capacity of the structural steel members. If there is no adequate protection against progressive fire damage, steel members are prone to collapse. Structural members may carry various loads according to the type of structural member and its location in the structure, with the amount of loading depending on the building's design. When a fire occurs in a building which carries comparatively light loads, the structural members may be more sustainable than those of buildings carrying heavier loads. In other words, the stability of the structural members during a fire is dependent upon the loading condition. This means that the lower the load ratio, the higher the fire resistance performance. The researches to predict the limiting temperatures of structural steel members such as beams and space frames under combined design actions were conducted by Wong (2005, 2006).

In the New Zealand Standard NZS3404 (1998), the equation for the limiting temperature adopted is given by

$$T_1 = 905 - 690r_f \tag{1}$$

where T_1 is the limiting temperature (°C) and r_f is the ratio of applied load to yield load $P_y(= AF_y)$. Eq. (1) was derived from several tensile strength tests carried out at high temperatures. The reduction of yield strength with temperature rise can be applied in the range from cold temperatures to 1,000.

The load ratio equation corresponding to the limiting temperature has been provided in Eurocode3 (1995) as

$$r_f = [0.9674(1 + \exp(T - 482/39.19))]^{-1/3.833} \tag{2}$$

where T is the limiting temperature (°C) and r_f represents the load ratio.

In the U.K., an evaluation method for fire resistance of structural steel is defined in BS5950 Part8 (1990). Fire resistance can be evaluated by the comparison of the limiting temperature of the structural steel and the design temperature derived from fire tests. The limiting temperatures for different load ratios of structural steel members are shown in Table 1.

3. Fire tests

3.1 Tensile coupon tests at high temperature

To obtain the yield and ultimate tensile strengths and the elongation of the structural steel at cold and high temperatures, tensile coupon tests were conducted in accordance with KS B 0802 (2003) and KS D 0026 (2002) respectively. The standard chemical compounds and mechanical properties of structural steel of grades SS400 and SM490 are summarized in Table 2. The shape of the tensile coupon tested at high temperatures is shown in Fig. 1. The tensile strength tests were executed in the range between room temperature and 900 at an interval of 100. Three specimens were tested at every designated temperature and the average temperature was determined to be the limiting temperature. The test device used for the tensile coupon tests was composed of a 250 kN UTM and a furnace as shown in Fig. 2.

Tensile coupon tests were executed using two different control methods, load control was applied from the start of testing to yield strength and displacement control was used from yield strength to

Table 1 Limiting temperatures of structural members (°C)

Classifications	Load ratios					
	0.7	0.6	0.5	0.4	0.3	0.2
Compression members						
$\lambda \le 70$	510	540	580	615	655	710
$70 < \lambda \le 180$	460	510	545	590	635	635
Members in bending supporting concrete slabs or composite slabs						
Unprotected members or protected members comply to standard	590	620	650	680	725	780
Other protected members	540	585	625	655	700	745
Members in bending not supporting concrete slabs						
Unprotected members or protected members comply to standard	520	555	585	620	660	715
Other protected members	460	510	545	590	635	690
Members in tensions	460	510	545	590	635	690

λ: slenderness ratio (L_e/i)

Table 2 Chemical components and mechanical properties

Steel grade	Components (%)					Mechanical properties			
	C	*Si*	*Mn*	*P*	*S*	Thickness (mm)	Yield strength (MPa)	Tensile strength (MPa)	Elongation (%)
SS400	0.157	0.238	0.80	0.019	0.008	$16 \geq t$ $16 < t \leq 40$ $t > 40$	245 235 215	400~510	17 21 23
SM490	0.117	0.404	1.38	0.016	0.004	$16 \geq t$ $16 < t \leq 40$ t > 40	325 315 295	490~610	17 21 23

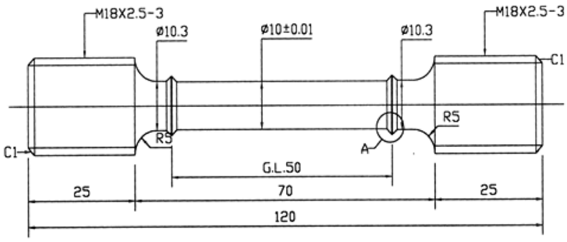

Fig. 1 Tensile coupon tested at high temperatures (unit: mm)

Fig. 2 Testing machine for tensile coupon tests at high temperatures

rupture of the tensile coupons. The loading speed and tolerance of the temperature within the furnace were applied differently according to the furnace temperature during test and are summarized in Table 3.

The stress versus strain curves for tensile coupons of SS400 and SM490 steel are shown in Figs. 3 and 4 respectively. The symbol R.T in the figures is an acronym for room temperature. As shown in Fig. 2, the yield and ultimate tensile strengths of SS400 steel decreased with the temperature rise, with the exception that the ultimate strengths measured at 200°C and 300°C increased. For SM490 steel, the yield and tensile strengths also decreased as the temperature rose, with the exception that the tensile strength measured at 300°C was higher than that measured at 200°C. Elongations for both SS400 and

Table 3 Conditions for tensile coupon tests at high temperatures

Classifications of temperature	Loading speed		Tolerance of temperature
	to yield strength	from yield strength	
Cold	17.0 MPa/sec	20.0 %/min	
High	7.0 MPa/sec	7.5 %/min	300~600°C = ±3°C 600~900°C = ±4°C

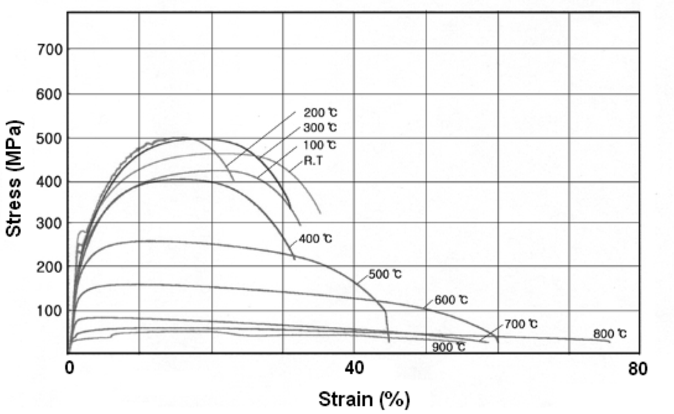

Fig. 3 Stress versus strain curves of SS400 steel

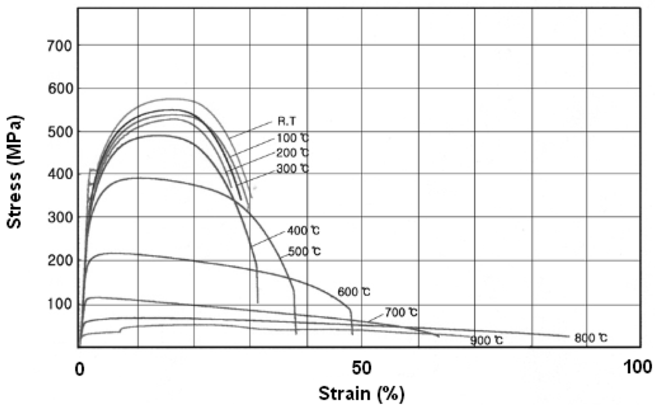

Fig. 4 Stress versus strain curves of SM490 steel

SM490 steel coupons measured showed little difference up to 400°C. However, at temperatures higher than 400°C, the temperature rise caused a significant increase in elongation.

The stress-strain curves obtained from the coupon tests conducted at room temperature or under the temperature of 300°C showed distinctive yield points. However, when the ambient temperature for the coupon test was higher than 300°C, the yield point was not clearly shown in the stress-strain curve. In that case, the 1.0% offset method was adopted to determine the yield stress (AIJ 1999).

The regressions for the yield strength and elastic moduli of SS400 and SM490 steel corresponding to

Table 4 Yield strength and elastic modulus for SS400 steel

Classification	Temperature (°C)	Regressions (MPa)	Remarks
Yield strength	$20 \leq T < 200$	240	value at room temperature
	200°C $\leq T$	$302 - 0.31T$	$R^2 = 0.96$
Elastic modulus	$20 \leq T < 200$	210,000	value at room temperature
	200°C $\leq T$	$261578 - 257.89T$	$R^2 = 0.90$

Table 5 Yield strength and elastic modulus for SM490 steel

Classification	Temperature (°C)	Regressions (MPa)	Remarks
Yield strength	$20 \leq T < 200$	330	value at room temperature
	200°C $\leq T$	$+426 - 0.48$	$R^2 = 0.97$
Elastic modulus	$20 \leq T < 200$	210,000	value at room temperature
	200°C $\leq T$	$+261980 - 259.9T$	$R^2 = 0.99$

the different temperature ranges are shown in Tables 4 and 5 respectively. In the temperature range from 20°C to 200°C, both limiting temperature and the elastic modulus for SS400 and SM490 remain constant. However, at the temperatures higher than 200°C, the limiting temperature and elastic modulus are proposed to decrease linearly with the temperature rise.

Simple load ratio formulas based on the tensile coupon test results are given by

For SS400 steel

$$r_f = 1.0 \qquad (T < 200) \tag{3a}$$

$$r_f = 1.26 - \frac{T}{774.2} \qquad (200 \leq T) \tag{3b}$$

For SM490 steel

$$r_f = 1.0 \qquad (T < 200) \tag{4a}$$

$$r_f = 1.29 - \frac{T}{687.5} \qquad (200 \leq T) \tag{4b}$$

where T is the limiting temperature (°C) and r_f represents the load ratio.

The load ratio versus temperature curves for SS400 and SM490 steel and the tensile coupon test results are shown in Fig. 5. The load ratio in the figure is defined to be a ratio of the yield strength measured at each designated temperature and that for room temperature. As shown in Fig. 5, there were no differences in load ratios up to approximately 200°C for SS400 and SM490 steel. However, the load ratios decreased linearly with temperature as the temperature was increased further. The slope on the load ratio versus temperature curve for SM490 steel was slightly steeper than that for SS400 steel.

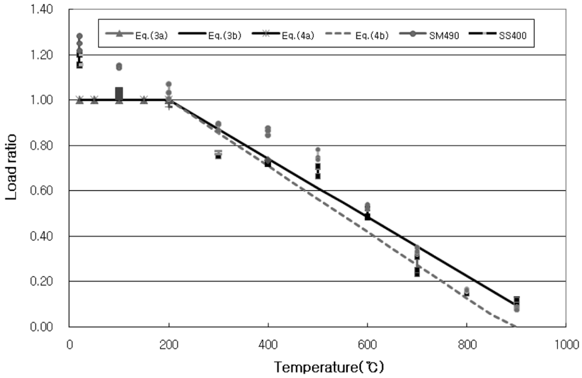

Fig. 5 Load ratio versus temperature curves

3.2 Fire tests of columns

A series of fire tests of steel columns were conducted using a 3000 kN testing machine equipped with vertical furnace (as shown in Fig. 6) in accordance with the standard fire curve defined in KS F 2257-1 (2005) and KS F 2257-7 (2005). The boundary conditions for the column ends were hinges.

Limiting temperatures derived from the tensile coupon test results at high temperatures were used as reference data for the evaluation of the fire resistance of the structural members. Structural members are manufactured through several fabricating procedures including cutting and welding. These procedures may cause initial imperfections with steel columns and beams. Therefore, there will be differences in limiting temperatures obtained from tensile coupon fire tests and full-scale column fire tests. To investigate the differences in those limiting temperatures, load-bearing fire tests of H-section and square and circular hollow section columns were conducted. To measure the surface temperature of the steel columns at failure, three thermo-couples were attached on the top, middle and bottom positions

Fig. 6 Testing equipment for column fire tests

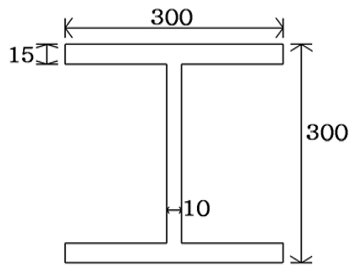

Fig. 7 H-300×300×10×15 section (unit: mm)

along the overall column (KS F 2257-1 2005, KS F 2257-7 2005).

3.2.1 Limiting temperatures of H-section columns

A series of fire tests were carried out on H-300×300×10×15 section columns of 3500 mm length. The structural steel grade was SS400 of which the nominal yield and ultimate tensile stresses were 235 MPa and 400 MPa respectively. The section geometries and dimensions of the H-section columns tested are shown in Fig. 7. Since the width-to-thickness ratios of the flanges and web were 10 and 17 and were less than the elastic local buckling limits in current specifications such as AISC specifications (2005) or Eurocode3 (1995), local buckling of the flanges or web would not occur during fire testing of the columns. However, since the overall slenderness ratio of 46.6 was smaller than the elastic Euler buckling limit, an inelastic column buckling was expected to occur during fire testing.

The limiting temperatures of the steel columns were determined as the surface temperature measured when the load-bearing criteria for both deflection and rate of deformation were exceeded simultaneously. The reference axial compressive load was determined to be 1,690 kN based on the allowable stress design method (KBC 2005). In the KBC, the allowable compressive stress is given by Eq. (5) and the allowable load can be computed multiplying it by the gross section area.

$$f_{ca} = \frac{\left[1 - 0.4\left(\frac{\lambda}{\lambda_p}\right)^2\right] \cdot F_y}{\frac{3}{2} + \frac{2}{3}\left(\frac{\lambda}{\lambda_p}\right)^2} = 140.8\ (\mathrm{kN/cm^2}) \tag{5}$$

$$P = A \cdot f_{ca} = 119.8 \times 140.8 = 1690\ \mathrm{kN} \tag{6}$$

where λ = slenderness ratio, λ_p = limit slenderness ratio, and F_y = specified yield strength.

To obtain the limiting temperatures according to the different load ratios, load-bearing fire tests of columns were conducted under the constant axial load of 1690 kN, 1352 kN, 1014 kN, and 845 kN. The applied loads were 100%, 80%, 60%, and 50% of the reference load respectively. The compressive loads were applied 15 minutes before the fire test started.

As the temperature was increased, the axial deflection due to linear expansion gradually increased up to the limiting temperature. At the limiting temperature, the column buckled in an overall buckling mode and failed abruptly. The shapes of the column before and after fire tests are shown in Fig. 8. As shown in Fig. 8, the column failed in the minor axis flexural buckling mode and typical kinks were formed in the flanges and web at the column center.

Fig. 8 Test column before and after fire test of H-section

Table 6 Test results for specimens at failure

Specimens	Load (kN)	Deflection (mm)	Deformation rate (mm/min.)	Elapsed time (minutes)	Surface temperature	Elongation (%)
H-100	1690	50.9	44.7	12	564	15.4
H-80	1352	59.0	72.1	13	604	13.1
H-60	1014	71.7	84.1	16	646	12.7
H-50	845	74.0	83.8	16	657	10.0

The results measured at the moment when the load bearing criteria of specimens were exceeded are summarized in Table 6. In the table, the number in the specimen title indicates the load ratio in percentage. The load-bearing criteria for the deflection was 35 mm and that for deformation rate was 10.5 mm/min, which correspond to length L/100 and 3L/1000 per minute respectively.

The deflection and the rate of deformation versus elapsed time curves are shown in Figs. 9 and 10. For the specimens under the axial load of 50% and 60% of the reference load, the criteria of load bearing capacity was exceeded at 16 minutes after the start of the fire test and the deformations were almost constant from the start to 16 minutes as shown. In the case of the specimens loaded with 100% and 80% of the reference load, a rapid rate of deformation occurred after 12 minutes and 13 minutes from the start of fire test. Referring to the test results in Figs. 9 and 10, it can be concluded that the fire resistance of H-section columns depends mainly upon the applied loads.

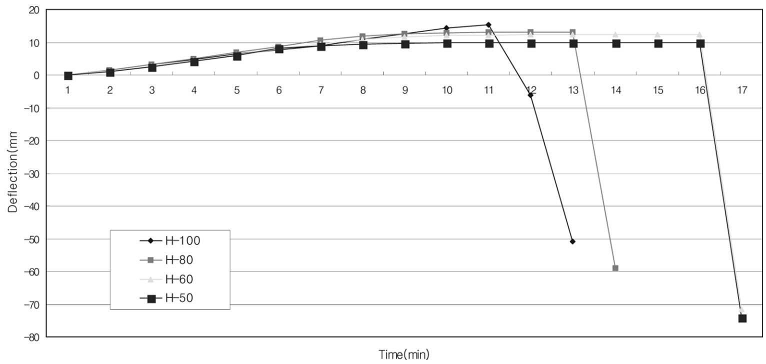

Fig. 9 Deflection versus elapsed time curves

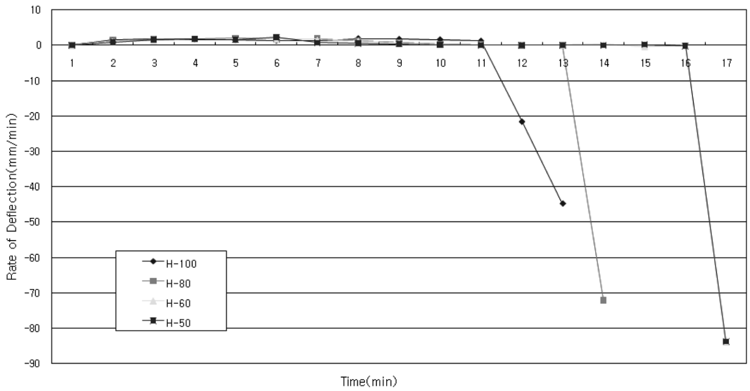

Fig. 10 Deformation rate versus elapsed time curves

The surface temperatures were obtained from the fire tests by using a standard fire curve and an analysis using STR-FR, which is a program to predict the fire resistance of steel columns (Kwon 2009), these are compared in Fig. 11. The surface temperature increased linearly with the elapsed time. The surface temperature of the columns subjected to high loads was generally higher than that of the columns under lower load measured at the same elapsed time. Though test results are slightly higher than the predictions made by STA-FR, they show good agreement overall. Referring to the relations of the surface temperature and load ratio in Fig. 11, it was found that the column specimen bearing higher axial load failed at earlier time and at lower temperature than that with lower load ratio.

The average and maximum temperatures of the H-section columns at fire resistance of 11 minutes, 12 minutes, and 15 minutes are given in Table 7. The average temperature of the H-100 specimen is quite similar to the allowable temperature defined in KSF 2257-7 (2005) at 538ºC. BS5950: Part 8 suggests the limiting temperature of columns according to the load ratios. The limiting temperatures corresponding to the load ratios of 0.7, 0.6, and 0.5 are 510ºC, 540ºC, and 580ºC respectively. These are slightly conservative in comparison with the limiting temperatures derived from the fire tests of H-section columns.

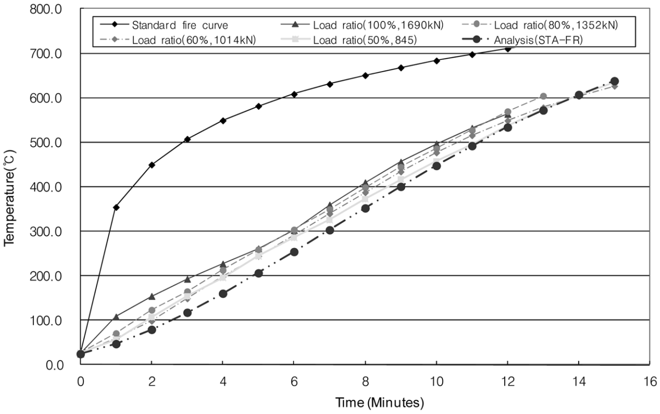

Fig. 11 Surface temperatures versus time curves for columns

Table 7 Surface temperatures of H-section columns at fire resistance time (°C)

Classification	Temperature (average/maximum: °C)					
	Section 1	Section 2	Section 3	Section 4	Average	Maximum
H-100	524/561	560/602	478/529	567/626	532	626
H-80	571/601	569/619	523/571	615/699	570	699
H-60	618/652	647/692	579/611	663/699	627	699
H-50	646/676	630/673	579/621	684/728	635	728

3.2.2 Limiting temperatures of hollow section columns

In order to investigate the effects of the cross sectional shape on the limiting temperatures of columns, a series of load-bearing fire tests were executed on circular hollow section (CHS) columns and square hollow section (SHS) columns in addition to H-section columns. The details of CHS and SHS columns tested are summarized in Table 8. The diameter of CHS columns and the width and depth of SHS columns are exterior dimensions. Since the width-to-thickness ratio of SHS columns and diameter-to-thickness ratio of CHS columns were smaller than the local buckling limit in the current specifications (EC3 1995, AISC 2005), local buckling of columns would not occur during fire testing.

The fire test method and evaluation method for hollow section columns are the same as those used for H-section columns. The reference loads were calculated based upon allowable stress design and are given in Table 9. The number at the end of specimen title indicates the percentage ratio of the applied load to the reference load, i.e., CHS-100 indicates that a circular hollow section column was tested under 100% of the reference axial load.

To obtain the limiting temperatures for the load ratios, four fire tests for the CHS and the SHS columns were conducted under constant axial load, which were 100%, 80%, 60%, and 50% of the reference load. The test loads were applied 15 minutes before the fire test started.

The structural behavior of hollow section columns during testing was quite similar to that of H-section columns. As the surface temperature was increased, the axial deflection due to linear expansion increased gradually up to the limiting temperature. At the limiting temperature, overall buckling of columns occurred and final failure followed immediately. The shapes of CHS and SHS columns before and after the fire tests are shown in Fig. 12. As shown in Fig. 12, the columns failed in the overall buckling mode and typical kinks were formed in the concave sides at the column center for both SHS and CHS sections.

Table 8 CHS and SHS column details

Specimens	Steel grade	Size (mm)	Thickness (mm)	b/t or d/t	Length (mm)
CHS Columns	STK 400	ø355.6	9.2	38.7	3500
SHS Columns	SPSR 400	300×300	9.0	31.3	3500

Table 9 Applied loads for CHS and SHS columns

Specimens	Loads (kN)	Specimens	Loads (kN)
CHS-100	1560	SHS-100	1630
CHS-80	1240	SHS-80	1300
CHS-60	930	SHS-60	980
CHS-50	780	SHS-50	810

(a) CHS column

(b) SHS column

Fig. 12 CHS and SHS columns before and after fire tests

The test results measured at the moment when the load bearing criteria of the specimens were exceeded are summarized in Table 10. In the table, the number in the specimen title indicates a load ratio as a percentage of the reference load. The load-bearing criterion for the deflection was 35 mm and that for the deformation rate was 10.5 mm/min, which correspond to L/100 and 3L/1000 per minute respectively.

The deflection and the deformation rate versus elapsed time curves are shown in Figs. 13 and 14 for CHS columns and Figs. 15 and 16 for SHS columns. The elapsed times when the load-bearing criteria was exceeded were 13 minutes for CHS-100, 15 minutes for CHS-80, 18 minutes for CHS-60 and 19 minutes for CHS-50 specimens. Those for SHS columns were 13 minutes for SHS-100, 17 minutes for SHS-80, 18 minutes for SHS-60 and 20 minutes for SHS-50 specimens. There was no significant

Table 10 Test results for specimens at failure

Specimens	Load (kN)	Deflection (mm)	Deformation rate (mm/min)	Elapsed time (min)	Surface Temperature (°C)	Elongation (%)
CHS-100	1560	45.1	61.1	14	592	16.9
CHS-80	1240	42.9	54.8	16	569	18.4
CHS-60	930	59.6	33.8	19	684	21.7
CHS-50	780	57.3	15.1	20	624	23.5
SHS-100	1630	47.3	20.9	15	566	16.3
SHS-80	1300	46.3	60.3	18	590	18.0
SHS-60	980	51.3	67.0	19	635	20.2
SHS-50	810	58.0	67.6	21	668	21.5

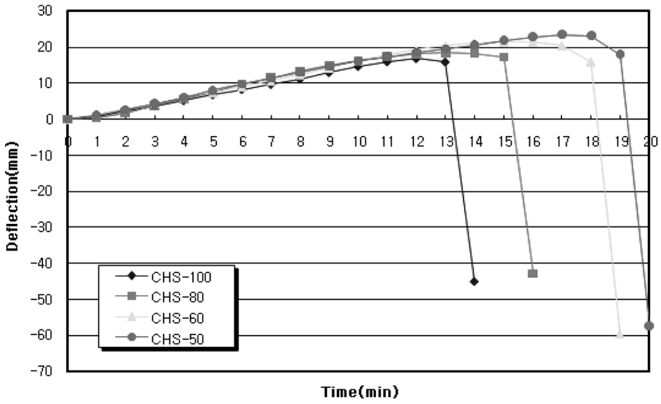

Fig. 13 Deflection versus elapsed time curves for CHS columns

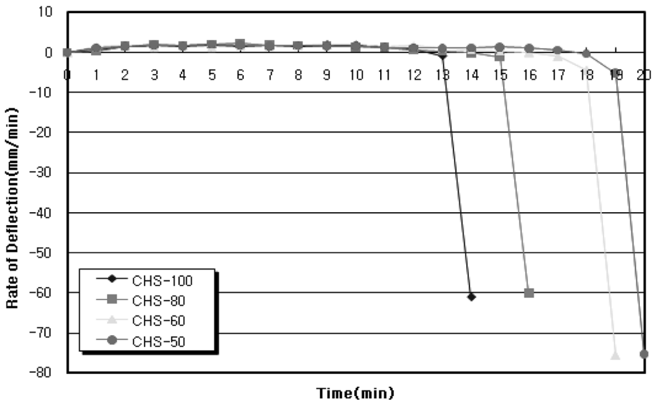

Fig. 14 Deformation rate versus elapsed time curves for CHS columns

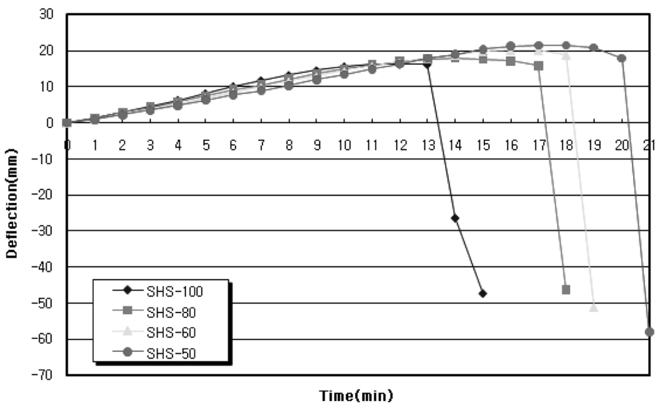

Fig. 15 Deflection versus elapsed time curves for SHS columns

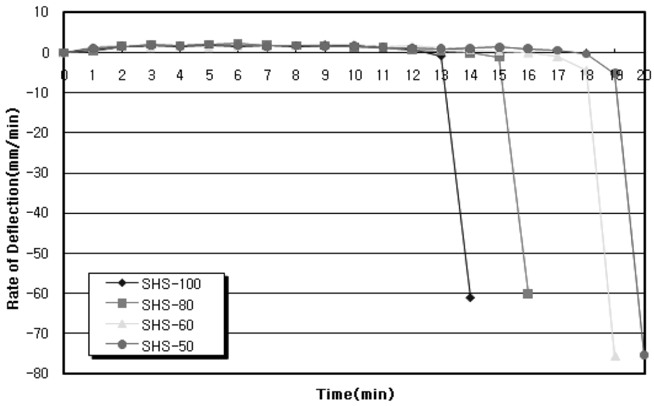

Fig. 16 Deformation rate versus elapsed time curves for SHS columns

difference between the structural behavior of CHS and SHS columns during fire tests. According to the results of the fire tests of CHS and SHS columns in the figures, it can be said that the columns under a large axial load failed faster than those under a smaller applied axial load. Consequently, it can be concluded that the fire resistance of hollow section columns depends mainly upon the load ratio.

The surface temperatures of the columns measured at failure are summarized in Table 11. The temperatures were not significantly different between the measured locations. However, the surface temperatures became higher for the columns with a decrease of the load ratio.

3.3 Comparison of limiting temperatures

A simple limiting temperature equation for SS400 steel columns can be derived from Eq. (3a) and (3b) and is given by

$$T = 975.20 - 774.2r_f \tag{7}$$

where T is a limiting temperature and r_f represents a load ratio.

Similarly, the limiting temperature equation for SM490 steel columns is given by

Table 11 Surface temperatures of columns at fire resistance time (°C)

Specimens	Load (kN)	Locations			Average	Maximum
		Top	Middle	Bottom		
CHS-100	1560	558	607	535	567	607
CHS-80	1240	498	563	573	545	573
CHS-60	930	577	706	676	653	706
CHS-50	780	582	606	633	607	633
SHS-100	1630	570	529	550	550	570
SHS-80	1300	562	570	570	567	570
SHS-60	980	630	613	593	612	630
SHS-50	810	670	631	649	650	670

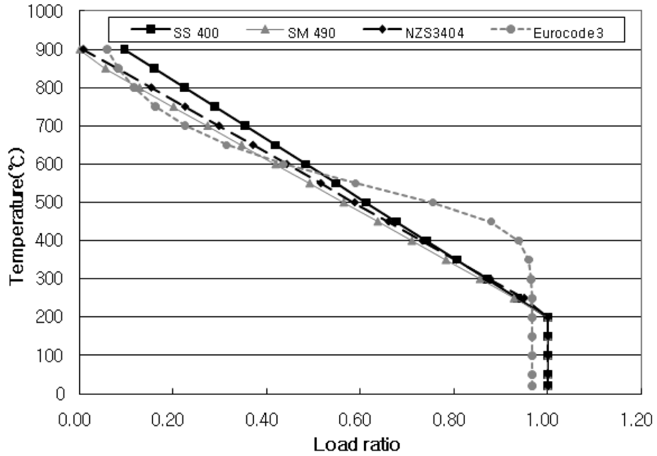

Fig. 17 Comparison of limiting temperature formulas and specifications

$$T = 886.88 - 687.5r_f \tag{8}$$

The load ratios calculated by Eqs. (7) and (8) are compared with those of NZS3404 and Eurocode3 in Fig. 17. As shown in Fig. 17, the predictions of load ratios by Eq. (7) and NZS3404 (1998) for SS400 steel show good agreement. The load ratios predicted by Eq. (8) for SM490 steel are slightly higher than those predicted by NZS3404 (1998). The load ratios predicted by Eurocode3 (1995) in the range of 200ºC to 600ºC are unconservative in comparison with the load ratios predicted by Eqs. (7) and (8). However, in temperatures higher than 600, Eurocode3 (1995) can predict the load ratios conservatively in comparison with Eqs. (7) and (8), and NZS3404 (1998).

The limiting temperatures predicted by Eqs. (7) and (8) are compared with fire test results of tensile coupons, H-section, SHS and CHS columns in Table 13. The limiting temperatures predicted by NZS 3404 (1998) and BS5950 are also included in Table 13 for comparison. Referring to the limiting temperatures in Table 13, the predictions by NZS3404 (1998) and Eq. (7) show a good agreement. The limiting temperatures predicted by NZS3404 (1998) are conservative in comparison with column fire test results but unconservative for tensile coupon test results. The predictions by Eq. (7) are also slightly unconservative in comparison with the limiting temperatures obtained by the fire tests of tensile coupons. However, for the columns tested, the limiting temperatures predicted by Eq. (7) are conservative in comparison with the fire test results. The limiting temperatures derived for H-sections are quite similar to those predicted by the BS5950. However, since the BS5950 adopts the limit state design, the load ratio referred should be converted in allowable stress design. For example, the load ratio of 0.7 in BS5950 is corresponding to 1.0 in the allowable stress design.

The maximum surface temperatures obtained from the fire tests of H-section, CHS and SHS columns were higher than those obtained from tensile coupon tests at high temperature. The main reason is that the steel columns designed according to the current design specifications generally have an adequate safety margin. Therefore, when the structural stability was evaluated by the load ratio, the limiting temperatures obtained from the column fire tests are more practicable. However, for practical use of the limiting temperature equations proposed, it is necessary to evaluate fire resistance for columns with various section types, boundary conditions and steel grades in further studies.

Table 13 Comparison of limiting temperatures

Load ratios	Limiting temperatures (°C)							
	Tensile coupons	H-section columns	CHS columns	SHS columns	Eq. (7)	Eq. (8)	NZS3404	BS5950
1.0	150	532	567	550	200	199	215	540
0.8	323	570	545	567	356	337	353	580
0.6	477	627	653	612	511	474	491	615
0.5	554	635	607	650	588	543	560	655

4. Conclusions

To investigate the limiting temperatures of columns, which are the maximum endurance temperatures for the structural members exposed to fire conditions under axial loads, a series of tensile coupon tests at high temperature and column fire tests were conducted. Simple equations to predict the limiting temperature for grade SS400 and SM490 steel were proposed based on the tensile coupon test results at high temperatures. The limiting temperature formulas were compared with current standards and the fire test results of H-section, CHS and SHS columns. From the experimental study, the following conclusions are drawn:

1. The simple limiting temperature formula proposed for SS400 steel was proven conservative in comparison with column fire test results.
2. The limiting temperatures obtained from the tensile strength tests at high temperature were very close to those predicted by NZS3404.
3. The limiting temperatures obtained from the fire tests on H-section columns showed good agreement with those defined in BS5950.
4. Since the columns designed by the current design specifications generally had adequate safety margins, the limiting temperatures obtained from the fire tests of H-section, SHS and CHS columns were higher than those from tensile coupon tests.

Reference

Architectural Institute of Japan (AIJ) (1999), "*Recommendations for Fire Resistance Design of Steel Structures*".
American Institute of Steel Construction (AISC) (2005), "*Specification for Structural Steel Buildings*", Chicago, Il, USA.
Bwalya, A. (2008), "An overview of design fires for building compartments", *Fire Technology*, **44**(2), 167-184.
BSI, BS 5950: Part8 (1990), "*Structural Use of Steelwork in Buildings*".
Buchanan, A. H. (1994), "*Fire Engineering Design Guide*".
CEN, Eurocode 3 (1995), "*Design of Steel Structures Part 1.2: General Rules Structural fire design*".
DETR (2000), "*The Building Regulations 1991, Fire Safety, Approved Document B*".
ISO TC92/SC4, WG12 (2009), "*Structures in fire*".
Korean Ministry of Construction and Transportation (2005). "Korean Building Codes".
Korean Ministry of Construction and Transportation (2002), "*Development of Fire Engineering Technique*".
Korean Standard Association, KS D 0026 (2002), "*Method of elevated temperature tensile test for steels and heat-resisting alloys*".
Korean Standard Association, KS B 0802 (2003), "*Method of tensile test for metallic materials*".
Korean Standard Association, KS F 2257-1 (2005), "*Methods of fire resistance test for elements of building*

construction-general requirements".
Korean Standard Association, KS F 2257-7 (2005), "*Methods of fire resistance test for elements of building construction-beam, column*".
Kwon, I.K. (2009), "Development of Analytic Program for Calculation of Fire Resistant Performance on Steel Structures", *Journal of Korean architectural institute*, **21**, 201-208.
Outinen, J. and Makelainen, P. (2004), "Mechanical properties of structural steel at elevated temperature and cooling down", *Fire and Materials*, **28**(2-4), 237-251.
Park, S.Y., Park, W.S. Kim, H.Y. and Hong, G.P. (2010), "Study on the Analytical Method for Fire Resistance Calculation of Asymmetric Slim floor Beam", *J. of Korean Institute of Fire Sci. & Eng*., **24**, 31-37.
Research Industry of Science & Technology (2004), "*Development of fire engineering technique of structural steels* (III)".
Richard Liew, J.Y. and Ma, K.Y. (2004), "Advanced analysis of 3D steelwork exposed to compartment fire", *Fire and Materials*, **28**(2-4), 253-267.
Saab, H.A. and Nethercot, D.A. (1991), "Modelling steel frame behavior under fire conditions", *J. Eng. Struct.*, **13**(4), 371-382.
SBI (1976), "*Fire Engineering Design of Steel Structures*".
Standards New Zealand, NZS3404: Part1:1997 (1998), "*Steel Structures Standard*".
Usmani, A., Roben, C. and Al-Remal, A. (2009), "A Very Simple Method for Assessing Tall Building Safety in Major Fires", *Journal of Steel Structures*, **9**(1), 17-28.
Wang, Y.C., Lennon, T. and Moore, D.B. (1995), "The behavior of steel frame subject to fire", *Journal of Constructional Steel Research*, **35**, 291-322.
Wong, M.B. (2005), "Modelling of axial restraints for limiting temperature calculation of steel members in fire", *Journal of Construction Steel Research*, **61**(5), 675-687.
Wong, M.B. (2006), "Effect of Torsion on Limiting Temperature of Steel Structures in Fire", *J. Struct. Eng.*, **132** (5), 726-732.
Zalok, E., Hadjisophocleous, G.V. and Mehaffey, J.R. (2009), "Fire loads in commercial premies", *Fire and Materials*, **33**, 63-78.

CC