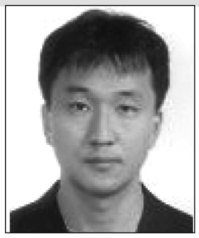
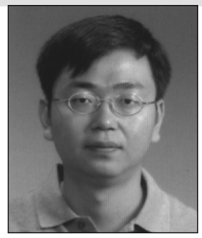
SC합성기둥의 성능기반 내화구조설계

Performance Based Structural Design of SC Composite Columns under Fire Condition


강 성 덕* 최 재 성** 김 인 기***

* 현대종합설계 구조설계팀 부소장
** 현대종합설계 구조설계팀 소장
*** 현대종합설계 대표이사

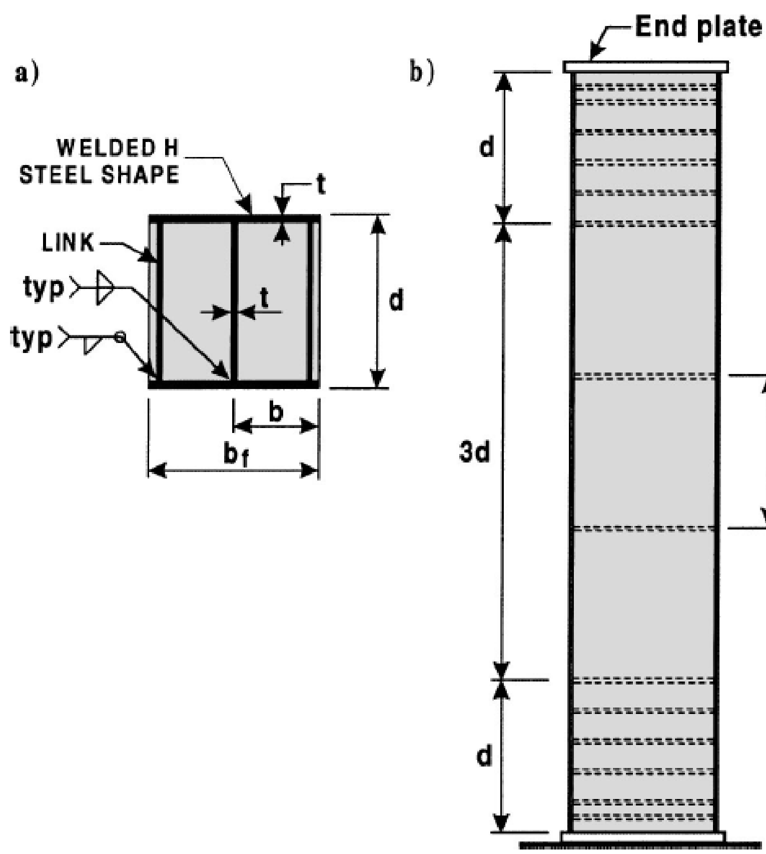
1. Introduction

Composite columns are very efficient structurally in resisting compression loads and are gaining popularity in multistory buildings. Such structural forms blend the advantages of concrete and steel which significantly improves the load-bearing capacity at both ambient and elevated temperatures. For concrete encased composite columns under fire conditions, the concrete which has a much lower heat conduction coefficient noticeably retards the rise of temperature in encased steel. By encasing these columns with concrete and stiffening with transverse links, the load-bearing capacity of the composite columns can be increased substantially.

As such, a new type of steel and concrete composite column consisting of a thin-walled, I-shaped steel section with concrete being poured between the flanges of the steel section has recently been developed and proposed (Fig. 1). The steel section features very slender plates exceeding the width-to-thickness ratio limits for non-compact sections. Transverse links between the flanges are spaced at regular intervals to enhance the resistance of the flanges to local buckling.

The proposed composite column is intended to carry only axial loads in multistory buildings, the lateral loads being resisted by other structural systems such as shear walls. The main advantages of this system over traditional composite design, concrete-filled tubes(CFT) and steel-reinforced concrete columns(SRC), have been presented elsewhere (Vincent, 2000), along with a series of tests on partially encased composite columns and a proposed design equation accounting for local flange buckling(Tremblay et al., 1998; Tremblay et al., 2000; Chicoine et al., 2002).

This study is presented the experimental and analysis studies on the fire resistance of the SC-Column under axial compression at elevated temperatures. A comprehensive study has been conducted to investigate the fire performance of the behavior and strength of the SC composite column made with concrete and steel H-section stiffened with transverse links. This paper presents the results of a parametric study using the heat transfer analysis and compares these results with experimental results to check the accuracy of the proposed



(a) Cross section (b) Elevation view

Fig. 1 Partially encased SC composite column

parametric studies.

The parametric studies, such as section size, load ratio and fire protection thickness, provide information of fire resistance of composite columns. Numerical predictions of temperature distribution during heating agree reasonably well with experimental data. It was observed that during heating all specimens underwent concrete spalling at mid-height, which noticeably decreased the fire resistance. Critical times of composite columns under compression in fire are also predicted according to Eurocode 3 and 4 Part 1.2 and ISO834 standard fire time-temperatures.

2. Material Characteristics at Elevated Temperatures

2.1 The EC3 Model

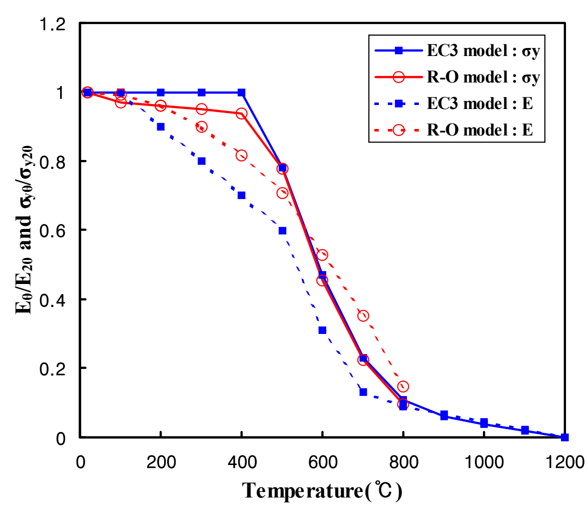


Fig. 2 Ratio relationship of yield stress and modulus of elasticity at elevated temperatures

The EC3 constitutive model for steel at elevated temperatures modifies the stress at a given strain using an equation, which is much more complicated than the Ramberg-Osgood equation. The stress-strain curves are made up from

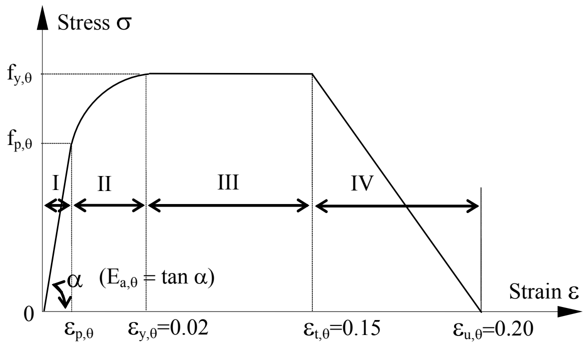


Fig. 3 Stress-strain relationship for structural steel at elevated temperature according to EC3

three sections. These are linear elastic for small strains, an elliptically curved part for intermediate strains and a constant ultimate stress for strains above 2%. There is provision for strain-hardening at high strains in the lower temperature range, but this is neglected here. In Fig. 2, it can be seen that the EC3 model gives a much higher yield stress than the Ramberg-Osgood model but EC3 model gives a much lower than the Ramberg-Osgood model for modulus of elasticity. The EC3: Part 1.2 presents the stress-strain relationship of structural steel at elevated temperature as a set of linear-elliptical curves as shown in Fig. 3.

2.2 The Ramberg-Osgood Model

The Ramberg-Osgood mode for steel at elevated temperatures modifies the strain at a given stress by the use of three temperature-dependent parameters, E_θ, $\sigma_{y\theta}$ and n_θ. If ε_θ represents strain and represents stress at temperature θ, the Ramberg-Osgood equations used for approximation of the British test data(Kirby and Preston, 1988) expressed as,

$$\varepsilon_\theta = \left(\frac{\sigma_\theta}{E_{20}}\right) + \left(\frac{3}{7}\frac{\sigma_\theta}{\sigma_{y20}}\right)^{50} \quad \text{for } 20^\circ C \le \theta \le 80^\circ C \tag{1}$$

$$\varepsilon_\theta = \left(\frac{\sigma_\theta}{E_\theta}\right) + \frac{1}{100}\left(\frac{\sigma_\theta}{\sigma_{y\theta}}\right)^{50} \quad \text{for } 80^\circ C \le \theta \le 800^\circ C \tag{2}$$

where,

ε_θ is the strain at temperature θ;

σ_θ is the stress at temperature θ;

σ_{y20} is the yield stress at ambient temperature ;

E_{20} is the modulus of elasticity of steel at ambient temperature ;

$\sigma_{y\theta}$ is the effective yield stress, which is taken as the 0.2% proof stress at temperature.

2.3 The EC4 Model

The strength and deformation properties of uniaxially stressed concrete at elevated temperatures shall be obtained from the stress-strain relationships in Eurocode2 Part 1.2. Table 1 gives for elevated concrete temperatures.

Table 1 Values for the two main parameters of the stress-strain relationships of normal weight concrete(NC) and light weight concrete(NC) and light weight concrete(LC) at elevated temperatures(EC4 Part 1.2)

Concrete temperature (℃)	$k_{c,\theta} = f_{c,\theta} / f_{c,20}$		$\varepsilon_{cu,\theta}\, x10^3$
	NC	LC	NC
20	1	1	2.5
100	0.95	1	3.5
200	0.90	1	4.5
300	0.85	1	6.0
400	0.75	0.88	7.5
500	0.60	0.76	9.5
600	0.45	0.64	12.5
700	0.30	0.52	14.0
800	0.15	0.40	14.5
900	0.08	0.28	15.0
1000	0.04	0.16	15.0
1100	0.01	0.04	15.0
1200	0	0	15.0

3. Experimental Program

The parametric studies, such as section size, load ratio and fire protection thickness, provide information of fire resistance of composite columns. Numerical predictions of temperature distribution during heating agree reasonably well with experimental data. It was observed that during heating all specimens underwent concrete spalling at mid-height, which noticeably decreased the fire resistance. Critical times of composite columns under compression in fire are also predicted according to Eurocode3 and 4 Part 1.2 and ISO834 standard fire time-temperatures.

3.1 Test Specimen

This study presents the results of an experimental study and analysis for the behavior of SC-Columns under compression at elevated temperatures. According to the Korean Industrial Standard KS2257-1, which is the standard used in the evaluation of fire resistance performance of SC-Column in this study.

The three parameters cross-section size, presence of fire protection clad, and load ratio were used in this experiment. Two specimens for each cross-section size, a total of seven specimens, were manufactured

Prior to testing, it is assumed that the entire surface area of the specimens is exposed to the heat of a fire, whose temperature follows that of the ISO834 fire standard curve of Fig. 4. The ISO834 fire standard time-temperature curve is given by:

$$T = 345 \log10 (8t+1) + T_0 \tag{3}$$

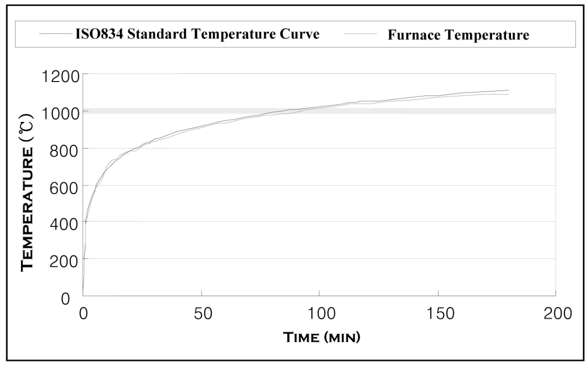


Fig. 4 Standard fire curve and furnace temperature

Where T : Heating temperature

T_{20} : Ambient temperature

t : Fire exposure time

In all, a total of 5 specimens were tested without fire protection. For each composite column, the fire exposure time, fire resistance, and temperature is recorded. A schematic view of a SC-Composite Column is shown in Fig. 5. To

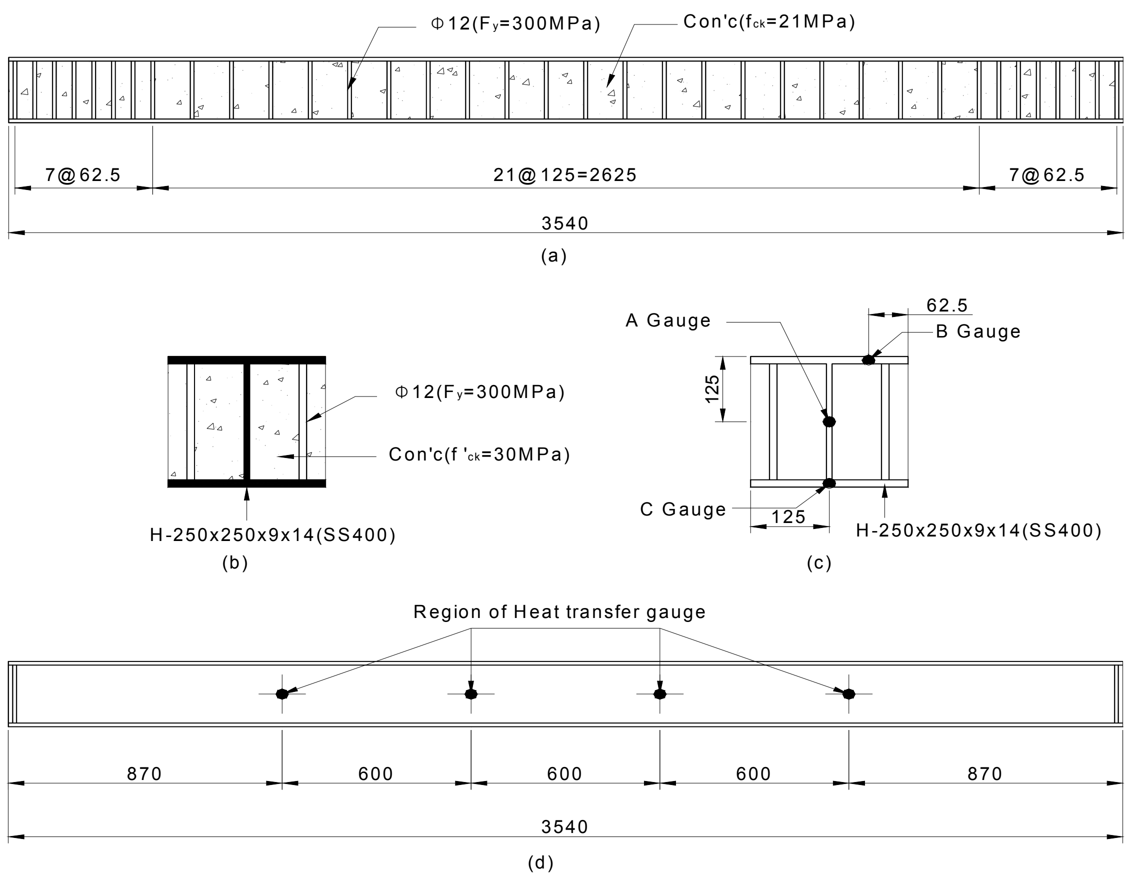


Fig. 5 SC Composite column specimen details and dimensions

ensure that the column acts as a composite section, steel link bars are placed between the flanges. These bars are welded .They were attached to the flanges of the steel section at regular spacing of 62.5 or 125mm. Thermocouples were built into each test specimen to record steel and concrete temperatures. Sufficient thermocouples were included to enable the temperature profiled of the cross-section to be obtained.

3.2 Test Conditions and Procedures

These composite columns were tested at an age of 28 days after concrete casting. The average compression strengths of concrete at that age were 24.3 and 23.8MPa. All composite columns were of rectangular cross section. The components of the column were dimensioned so that their lengths' was 3,540mm excluding the plate thickness. Prior to testing, the top surfaces of the partially encased composite columns were ground smooth and flat using a grinding with diamond cutters. This was to ensure that the load was applied across the section to the steel and concrete.

The tests were carried out by exposing the columns to heat in a furnace specially built for testing loaded columns in Korea. A hydraulic jack with a capacity of 300 tons produces a load along the axis of the test column. Deflection and deflection rate were measured by installing LVDT's at the very center of the position where the highest modification is expected to occur, as shown in Fig. 6(a). The furnace temperatures are measured by the thermocouples are average temperature used as the criterion for controlled in such a way that the temperature in the furnace followed as the ISO-834 standard curve.

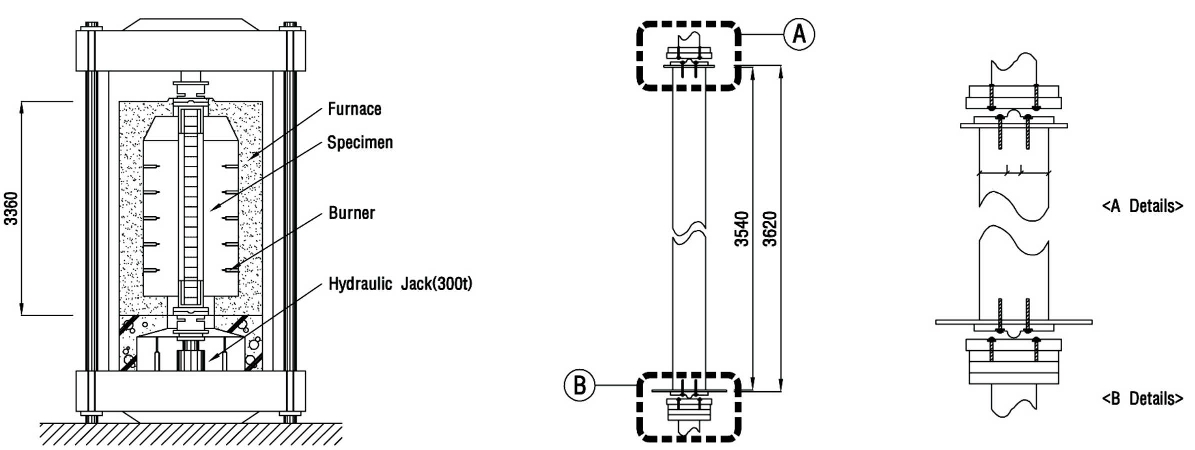
The columns were installed in the furnace by securing the end plates to a loading head at the top and a hydraulic jack at the bottom. All the columns have pinned-end conditions, as shown in Fig. 6(b).

4. Experiments and Analyses for SC-Columns in Fire

4.1 Failure Mode of Specimens

SC-Column A specimen performed achieved a fire resistance of 28 minutes. The test was discontinued because of the rate of deflection was excessive and total collapse was imminent. An examination of the specimen after the test showed that failure had occurred due to overall buckling at the center of the column. Little spalling of the concrete had taken place and little local buckling of the steel flanges were observed. The SC-Column B performed similarly to SC-Column A, 38 minutes. SC-Column C achieved a fire resistance of 67 minutes. An examination of the specimen showed that an overall buckling failure had occurred about the center of the column. No spalling of the concrete had taken place and very little local buckling had occurred.

The behavior of the SC-Column D and SC-Column E appeared to be very similar. It was observed that the tested specimens all failed by compression or overall buckling. The columns were subjected to loads of 0.3, 0.5 and 0.7 times. In



(a) Schematic view of the test set-up

(b) Details of end plates

Fig. 6 SC composite column test set-up

Table 2 Results of experimental test specimens

Specimen	Section Size	Load (KN)	Load Ratio	Failure Time (min)
SC-Column A	H-250x250x9x14	1452	0.7	28
SC-Column B	H-250x250x9x14	1217	0.5	38
SC-Column C	H-250x250x9x14	306	0.3	67
SC-Column D	H-190x190x12x6	301	0.5	35
SC-Column E	H-300x300x12x6	239	0.5	63

(a) SC-Column A

(b) SC-Column B

(c) SC-Column C

(d) SC-Column D

(e) SC-Column E

Fig. 7 Failure mode of SC-Column specimens

all the tests there was no serious sign of concrete spalling. It is expected that as the steel starts to expand at the early stage of heating, compressive stress in the concrete will be decreased, then the steel may locally buckle, which transfers additional load onto the concrete. In a final limit state, the steel may do additional load onto the concrete, until the concrete fails in a brittle manner.

The influences of the section dimension and load ratio on the temperature of the composite columns are given in Table 2. For example, with the same load ratio, SC-Column D specimen takes about 35min to reach a failure, and 63 min in the specimen SC-Column E.

Fig.s 7(a)~(e) show the failed picture after test for specimens of Partially Encased Composite Columns of steel H-columns under compression in fire. It can be found that specimens occurs the overall buckling after axial deformation in center height part of column. It was observed that during heating all specimens underwent concrete spalling at mid-height, which noticeably decreased the fire resistance.

In specimens without fire protection, fire resistance performance increased along with the increase in cross-section size, and the time to maximum temperature of the member increased as the cross section size increased. The column section size has a relatively significant influence on the fire resistance of the column.

Fire resistance performance increased as the load ratio decreased as the time to allowable deflection and time to allowable deflection ratio decreased as the load ratio decreased. Temperature rise after fire exposure was slower as the cross-section size is larger. Fire resistance performance

Table 3 Comparison between test and analysis results

Model	Test				Analysis	Difference (%)
	Load Ratio	Time(min)	Temp.(℃)	Pmn, θ(KN)	Pmn, θ(KN)	
SC-Column A	0.7	28	358	1452	1541.84	6.2
SC-Column B	0.5	38	494	1217	1316.92	8.2
SC-Column C	0.3	67	676	306	329.71	7.7
SC-Column D	0.5	35	521	301	275.73	8.8
SC-Column E	0.5	63	634	239	213.88	11.7

increased along with the increase in cross section size.

4.2 Comparison with Experimental and Analytical Results

The heat transfer analysis was carried out using the general FE package ANSYS program. As heat transfer is the movement of heat by conduction, convection, and radiation, and as temperature inside an object varies by position and time, heat distribution was analyzed by the function of position and time. The SC-Column was simplified as 2-D by assuming that the same size of fire is applied for the entire section of the member.

Specific heat and thermal conductivity, representing thermal characteristics of the materials used in the heat transfer analysis for the SC-Column, are functions of temperature and are used for nonlinear analysis. In this study, thermal characteristics of the steel and the concrete presented by Eurocode 3 and 4 Part 1.2 were used. The comparisons between experimental and analytical results of this study are summarized in Table 3.

5. Conclusions

By using partially encased composite columns, which can be formed on or off-site, the need for expensive fire protection to the columns can be reduced or eliminated.

The high temperature stress-strain relationships of the steel and concrete used herein are the constitution models of the Eurocode3 and 4 Part 1.2. Comparisons are also made with the results of fire tests and analyses of this study. The test results of this study give a good agreement with the analytical results.

The composite columns under compression in fire had occurred mainly the overall buckling after axial deformation in center part of the column height before the yield failure. The buckling and yield stress, and the failure load of composite columns under compression in fire are almost invariable in $\theta \leq 400$℃ but are decreased very rapidly in $\theta > 400$℃.

References

1. Chabot M, Lie TT. Experimental studies on the fire resistance of hollow steel columnsfilled with bar reinforced concrete. Institute for Research in Construction, National Research Council of Canada, Ottawa, Ont., IRC International Report No. 628; 1992.
2. Lie TT, Chabot M. Experimental studies on the fire resistance of hollow steel columns filled with plain concrete. Institute for Research in Construction, National Research Council of Canada, Ottawa, Ont., IRC International Report No. 611; 1992.
3. Prion HGL, Baraka M. Thin-walled tubes filled with high strength concrete. Canadian Journal of Civil Engineering 1994; 21(1); 207-218.
4. Sakumoto Y, Okada T, Yoshida M, S. Tasaka S. Fire resistance of concrete filled fire resistance steel tube columns. Journal of Materials in Civil Engineering 1994; 6(2): 169-184.
5. Lie TT. Fire resistance of circular steel columns filled with bar-reinforced concrete. ASCE Journal of Structural Engineering 1994; 120(5); 489-1509.
6. Kodur VKR, LieTT. Fire performance of concrete-filled hollow steel columns. Journal of Fire Protection Engineering 1995; 7(3); 9-98.
7. Kodur VKR, Lie TT. Experimental studies on the fire resistance of circular hollow steel columns filled with steel-fibre-reinforced concrete. Institute for Research in Construction, National Research Council of Canada,

Ottawa, Ont., IRC International Report No. 691; 1995.
8. Chabot M, Lie TT. Experimental studies on the fire resistance of hollow steel columns filled with bar-reinforced concrete., Institute for Research in Construction, National Research Council of Canada, Ottawa, Ont., IRC International Report No. 628; 1992.
9. Lie TT, Chabot M. Experimental studies on the fire resistance of hollow steel columns filled with plain concrete. Institute for Research in Construction, National Research Council of Canada, Ottawa, Ont., IRC International Report No. 611; 1992.
10. Han LH, Yang YF, Xu L. (2003), An experimental study and calculation on the fire resistance of concrete-filled SHS and RHS columns. Journal of Construct Steel Research 2003 59(4); 427-452.
11. Lin-Hai Han, Jing-Si Huo (2003), Concrete-filled hollow structural steel columns after exposed to ISO-834 fire standard, Journal of Structural Engineering, Vol. 129, No. 1, pp. 68-78.
12. Y. Sakumoto, T. Yamaguchi, T. Okada, M. Yoshida, S. Tasaka, H. Saito (1994), Fire resistance of fire-resistance steel columns, Journal of Structural Engineering, Volume 120, Issue 4, pp. 1103-1121.
13. T.T. Lie and V.K.R. Kodur (1996), Fire resistance of steel columns filled with bar-reinforced concrete, Journal of Structural Engineering, Volume 122, Issue 1, pp. 30-36.

[담당 : 김명한, 편집위원]