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Evaluation of the Fire Resistance Performance of Interior Anchor Type CFT Columns through Loaded Heating Test

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Abstract

The fire resistance performance of generic CFT columns has been verified through various tests and analyses and the columns are widely used for fire resistance designs abroad. In this study, 3 groups of specimens (Non-fire protection, reinforcement with steel fiber and fire resistance paint) are suggested in order to evaluate the fire resistance performance of interior anchor type concrete-filled steel tubular columns having efficient cross-sections through loaded heating tests. Axial deformation-time relationship and in-plane temperatures are compared to evaluate the fire resistance performance of the specimens associated with variables. Suggested from the fact that the interior anchors exposed to fire exert influence on fire resistance performance due to thermal expansion, the reinforcements using steel fiber and fire resistance paint are verified to mitigate contraction and improve fire resistance performance. The result obtained from the tests of interior anchor type concrete-filled tubular columns is expected to be used for effective fire resistance design in association with previously conducted studies.

Keywords: Fire resistance performance, ACT column, Steel-fiber, Fire resistance paint

1. Introduction

The studies on concrete filled steel tubular columns have been made since early 1990s. Due to the rise in steel price and the development of high-strength materials, the methods to reduce steel amount and use structural members more efficiently have been suggested. The problem inherent in the use of thin steel plates in square CFT columns is local buckling which makes it infeasible for the steel tube to confine the concrete and improve sectional efficiency. Since the tube made of thin steel plates does not enable confinement effect and ductile behavior, stud bolts or anchor bolts are placed in the tube before concrete casting to induce composite effect. The studies on the CFT columns with interior or exterior stiffeners were started in late 1990s.

Choi suggested L-channel bending shown in Fig. 1(a) to stimulate the composite effect between the concrete and steel tube. Placing the welding joints at the center of the 4 sides enhances workability and mitigates stress concentration in the corners. Figure 1(b) shows the cross-section of the column called 'Advanced Construction Technology (ACT)' column developed in 1997. Water press test was carried out to verify the welding performance of the columns and evaluate the safety of their welding throat depth. In addition, the studies on the seismic performance and structural behavior of the columns have been made continuously. Since the construction technology of the ACT columns was certified in Korea in 2011, they have been used in the structures which require high axial force.

Although CFT column is recognized as a fire resistant member thanks to its thermal storage effect, it does not provide the fire resistance performance required for high-rise buildings. In order to deal with this shortcoming, reinforcement using steel fiber or steel bar was developed. Most of the studies dealt with the CFT columns less than 400 mm in width and those without fire protection. The employment of CFT columns without fire protection is considered for economic reasons and many studies have been made on their fire-resistance performance.

The columns used in the buildings in Korea should have 1~3 hours of fire-resistance depending on building size as shown in the Table 1 of fire resistance rating. In

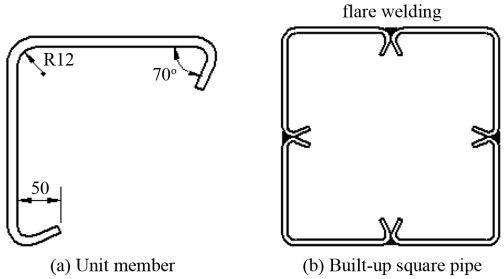


Figure 1. ACT (Advanced Construction Technology) Column (Choi, 2011).

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Table 1. Fire resistance rating for buildings

(Unit : hour)

Occupancy	Story (F) / Height (m)		Column
Common Residential Industrial	12 F or 50 m	Excess	3
		Below	2
	4 F or 20 m	Below	1

this study, reinforcement with steel fiber and fire resistance paint is suggested to secure the fire resistance performance of ACT columns. In order to evaluate the fire resistance performance of non-fire protection columns and those reinforced with steel fiber or fire resistance paint, loaded heating tests were conducted and axial deformation and in-plane temperatures were analyzed.

2. Fire Resistance Performance of ACT Columns Evaluated by Existing Studies

Major studies on the fire resistance of CFT columns were made by the researchers of NRCC (National Research Council of Canada) such as Kodur and Lie (1995) and those in Fuzhou University, China led by Han and sponsored by CIDECT (Comité International pour le Development et l'É tude de la Construction Tubulaire). In the studies made by NRCC, standard fire resistance tests were carried out for CFT columns without reinforcement and those reinforced with steel bars or steel fiber subject to axial load or combined loads generating eccentricity. All of the columns in the studies didn't have fire-protection and both ends had rigid connection. The results of the studies are as follows. The fire resistance performance of the columns without reinforcement (Case 1) should be considered to be lower than what was observed from the test. Although the reinforcement with steel bars (Case 2) improves the fire resistance performance of the CFT columns, the problems of increased costs and limited column size are yet to be solved. Reinforcement with steel fiber solves the problems inherent in Case 1 and Case 2. Kodur and Lie (2005) observed the thermal and dynamic characteristics of the concrete-filled columns reinforced with steel fiber and developed the numerical approach to forecast the fire resistance of the columns. Eq. (1) is for the fire resistance design of the CFT columns subject to axial load (Kodur and Lie, 2000). Table 2 shows the range limit of the equation.

$$FR = f_1 \frac{(f_{ck}+20)}{(KL-1000)} D^2 \sqrt{\frac{D}{C}} \tag{1}$$

Eq. (2) shows the maximum value of 'C' in equation 1 when the maximum limits are applied. The equation can be used to estimate the axial load ratio for required fire resistance duration. The 'a' factor in the equation is decided according to the aggregate and reinforcement types of the concrete shown in Table 3. Since the concrete deals with the load when the composite column is exposed to high temperatures, applied load 'C' is limited by the compressive load capacity of the concrete (C'_r) as shown in Eq. (3). The maximum is 1.1 times and 1.7 times of (C'_r) in the columns without reinforcement and those reinforced with steel bars or fiber.

$$C \le C_{max}, \; C_{max} = \left(\frac{a(f_c+20)D^{2.5}}{R(KL-1000)} \right)^2 \tag{2}$$

$$C_r = 0.85\varphi_c f'_c A_c \lambda_c^{-2} [\sqrt{1+0.25\lambda_c^{-4}} - 0.5\lambda_c^{-2}] \tag{3}$$

Table 4 summarizes the fire resistance prediction of the ACT specimens to be tested in this study which was obtained by Eqs (1) and (2). The lengths and widths of the columns were applied to the equations even though they exceeded the range limits. The prediction for the specimens reinforced with fire resistance paint shows the fire resistance before painting.

Fire resistance performance is the condition which should be achieved in structural designs involving various variables. Architectural designers use the equations to satisfy the fire resistance required by structural variables such as column length and width, load and concrete strength. Using these equations enables structural design to be economically efficient and cost-effective. CSCC "Fire Protection Bulletins" (20, 21 and 22) includes the design chart for various fire resistance levels to simplify structural design process where C_{max} is related to the effective length associated with various dimensions and concrete strengths. Figure 2(a) is the design graph for square HSS column resisting fire for 1 hour. In order to help the designers and constructors of concrete-filled HSS columns, the National Research Council of Canada

Table 2. Range limit of Eq. (1)

Parameter	Concrete Filling		
	Plain	Steel-fibre reinforced	Bar-reinforced
f'c (MPa)	20 to 40	20 to 55	20 to 55
D (round) (mm)	140 to 410	140 to 410	165 to 410
D (square) (mm)	140 to 305	102 to 305	175 to 305
Reinforcement (%)	N/A	~2% of the concrete mix by mass	1.5% to 5%
Concrete Cover (mm)	N/A	N/A	25
R (minutes)	≤ 120	≤ 180	≤ 180
KL (mm)	2000~4000	2000~4500	2000~4500

Table 3. 'a' factor in Eq. (2)

Aggregate type	Filling type	Steel reinforcement	Circular columns	Square columns
siliceous coarse Aggregate	PC	N/A	0.070	0.060
	FC	~2%	0.075	0.065
	RC	1.5~3%	0.080	0.070
		3~5%	0.085	0.075
Carbonate coarse Aggregate	PC	N/A	0.080	0.070
	FC	~ 2%	0.085	0.075
	RC	1.5~3%	0.090	0.080
		3~5%	0.095	0.085

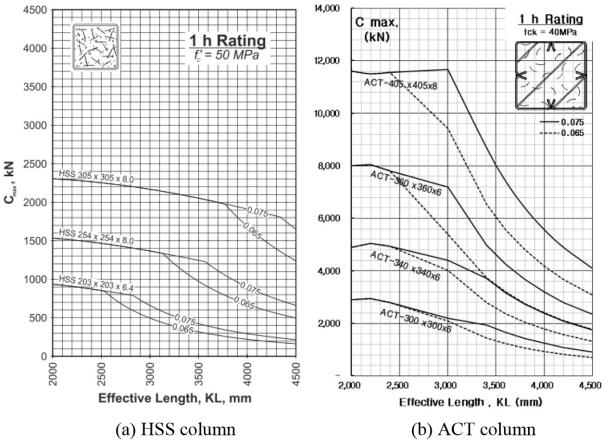
Table 4. Fire resistance prediction

No.	Objects	Named	Parameter	$B \times t \times L$ (mm)	C'_r (kN)	C_{rc} (kN)	Test Load C (kN)	C/C'_r	C/C_{rc}	Eq1-FR (min)
1	Non-fire Protection Steel Fiber concrete	A	Steel-bar Scale down	616×10×4570	6766	26618	8848	1.31	0.33	97
2		B		616×10×4578	8858	23875	8972	1.01	0.38	96
3		C		616×10×4576	7080	23875	7476	1.06	0.31	105
4		E		408×8×3976	2655	11440	3207	1.21	0.28	68
5		1	0%	300×6×3449	3395	6198	1132	0.33	0.18	62
6		2	0.25%	300×6×3449	3395	6198	1132	0.33	0.18	62
7		3	0.375%	300×6×3448	3393	6198	1132	0.33	0.18	62
8		4	0%	300×6×3449	3395	6198	1617	0.48	0.26	51
9		5	0.25%	300×6×3449	3395	6198	1617	0.48	0.26	51
10		6	0.375%	300×6×3445	3387	6198	1617	0.48	0.26	51
11	Fire Resistance Paint	B-1	1.3 mm	616×10×4576	7080	23875	8972	1.27	0.38	96
12		B-2	3.5 mm	616×10×7879	7089	23875	8972	1.27	0.38	96

and the Canadian Steel Construction Council developed and published design guide in 1998. The guide for the fire resistance design of these composite columns will be added. Figure 2(b) is the fire resistance design graph for the ACT columns drawn by the authors of this study based on Fig. 2(a) in the hope that various data is established to be used as fundamental information.

3. Methods to Secure the Fire Resistance of Interior Anchor Type CFT Columns

3 groups of ACT specimens were tested in this study to evaluate fire resistance performance. The first group consisted of the ACT columns without fire protection with the target of 1 hour fire resistance. The specimens in the second group were reinforced with steel fiber for 2 hours of fire resistance. The third group consisted of the specimens reinforced with fire resistance paint for 3 hours of fire resistance. The specimens in the last group had the same shape with those in the first group. Errors were observed during the loaded heating test due to the problems associated with boundary condition and the state of heating furnace. In spite of repeated tests, it was difficult to get clear answers to the fire resistance performance of the ACT columns and additional analysis is yet to be made. In this paper, the results of a study performed subsequent progress will be made in conjunction with the experiment in this chapter is to arrange the results of a study performed.



(a) HSS column (b) ACT column

Figure 2. Fire resistance design graphs for composite columns.

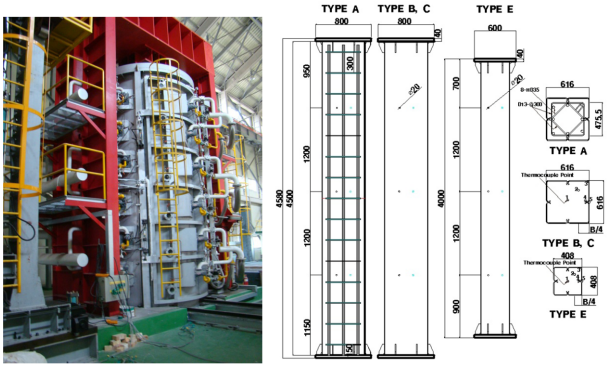
3.1. Non-fire protection column (1 hour)

3.1.1. Experiment overview

All of the 4 non-fire protection columns were made of the same steel (Fy = 325 MPa) and concrete (42.5 MPa). The variables in the test were reinforcement with steel bars, load ratio and column dimensions. Table 5 shows the specimens and their specification. 5 thermo couples were installed to measure the temperatures and deformation of the steel and the concrete. The test was carried out in accordance with KS-F 2257-1 standard heating curve and Eq. (4) and a 1,000 ton heating furnace shown in Fig. 3(a) was used. End-plates (800×800×40 mm) were placed at the top and bottom of the specimens to place them at

Table 5. ACT specimens without fire protection

Type	*B* (mm)	*t* (mm)	*L* (mm)	Load Ratio	Test Load *C* (kN)	Main Parameter
A	612	10	4554	0.5	8848	Steel-Bar reinforced 8-D35
B	610	10	4512	0.6	8972	Load ratio Increase
C	611	10	4485	0.5	7476	Basic
E	402	8	3979	0.5	3207	Scale Down



(a) 10,000 kN heating furnace (b) Specimens details

Figure 3. Heating furnace, specimen details and location of thermo couples.

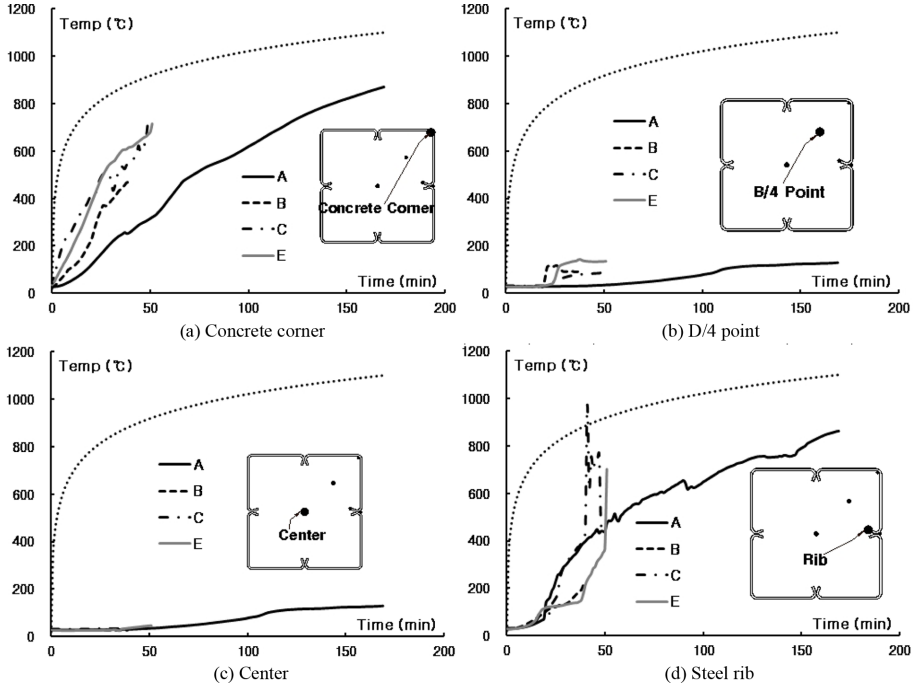
the heating furnace. 6 vent holes consisting of 3 sets were made to release pressure inside the columns. The dimension of the vent holes was 20 mm. Figure 3(b) shows specimen details including the location of thermo couples and vent holes.

$$T = 345\log 10_{10}(8t+1)+20 \quad (4)$$

3.1.2. Experiment results

Figure 4 shows the temperatures at the different locations of the columns. As shown in Fig. 4(a), the temperatures measured at the corners of the concrete rose rapidly. Among the 4 specimens, specimen A that was reinforced with steel bars showed relatively stable temperature rise. The temperatures at 1/4 point from the surface did not exceed 100°C as shown in Fig. 4(b). The temperatures of the interior anchors in specimens B, C and E rose rapidly from 50 minutes due to rapid contraction as shown in Fig. 4(d). Deformation was accelerated when gaps were made between the steel tube and the concrete and interior anchors were separated from the concrete due to the expansion of the steel tube exposed to fire. In the specimen A reinforced with steel bars, it took more than 100 minutes to reach 600°C.

Figure 5 shows the axial deformation of the specimens. The thermal expansion of the steel tubes upon heating caused axial expansion of the specimens. The deformation reached climax in 20 minutes of heating when the furnace temperature was 600~700°C. Then, axial expansion stopped and contraction deepened due to the deterioration in the load capacity of the steel tubes and the local buckling of the tubes at their ends and the load to the exterior tubes was transferred to the concrete and interior tubes. There-



(a) Concrete corner (b) D/4 point

(c) Center (d) Steel rib

Figure 4. Temperature of ACT columns without fire protection.

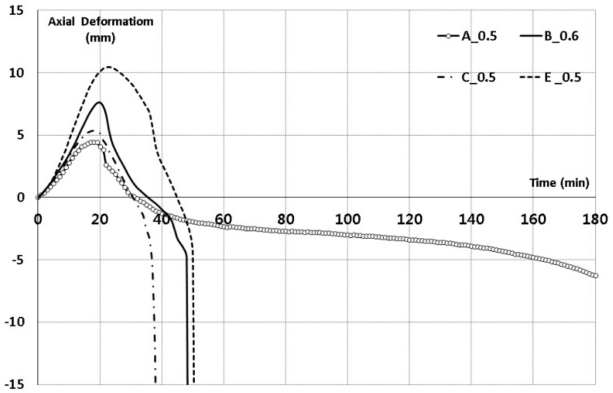


Figure 5. Axial Deformation of the ACT columns without fire protection.

fore, all the specimens except that reinforced with steel bars showed rapid contraction. In specimen A reinforced with steel bars, the concrete continued to deal with the load and fire resistance duration was 180 minutes.

Table 6 summarizes the weight, length and width of the specimens before and after the test. Figure 6 shows the failure mode of the specimens at the termination of the test. The difference in weight was the biggest in specimen A because the moisture in the concrete vaporized and the surface of the steel tubes was oxidized during the 3 hours of heating.

3.1.3. Analysis and discussion

Resisting fire for about 50 minutes, ACT columns without fire protection did not reach the goal of 60 minutes. Since the load ratio in the specimens was 50~60%, the load applied to the concrete was too great. As shown in Table 4, C/C'_r was bigger than 1. It is deduced that the required fire resistance duration of 60 minutes can be achieved if the load ratio is adjusted downward.

The temperatures of interior anchors rose rapidly. After the termination of the test, it was observed that the damage to the interior anchors was more severe than that to the corners, seemingly because the expansion of the anchors caused cracks at the concrete and reduced contraction significantly. To verify this conjecture, the condition of the concrete inside the steel tube was examined. As shown in Fig. 7, the concrete in specimen B and C fell to pieces and the damage covered more than 40% of the concrete cross-sections.

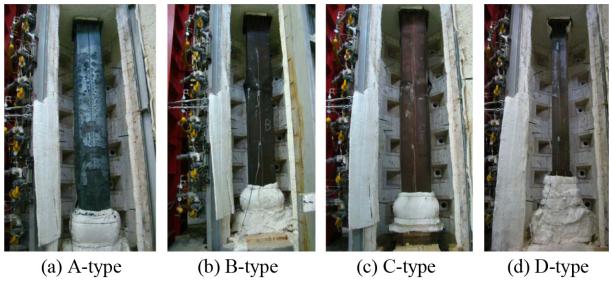


Figure 6. Failure mode of the specimens without fire protection.

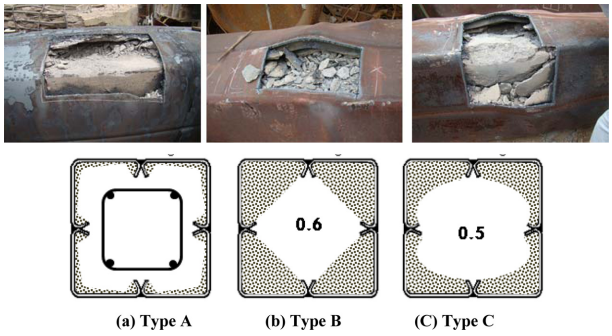


Figure 7. The condition of the concrete after heating.

3.2. Steel-fiber reinforced column (2 hours)

One of the findings from the test of non-fire protection columns was that fire resistance performance is improved if the concrete remains in good condition. However, reinforcing the ACT column with steel bars involves complicated work process though it improves fire resistance performance greatly. Therefore, steel fiber was mixed with the concrete to increase fire resistance up to 2 hours. Among the various types of steel fiber, crimped-end wire type which is widely used for structural purpose and does not clump together was selected. The shape of the fiber is shown in Table 7. Table 8 shows the compressive, tensile and bending properties of the steel fiber concrete of different curing ages obtained from the material test which was conducted before the loaded heating test.

3.2.1. Experiment overview

All of the 6 ACT columns reinforced with steel fiber had the same cross-section and length. The variables in the test were steel fiber mixing ratio and load ratio. Table

Table 6. The dimensions of ACT columns without fire protection before and after heating (A: Before Heating , B: After Heating)

	A Weight (kg)	B Weight (kg)	Weight gap (kg)	A Length (mm)	B Length (mm)	Length gap (mm)	A Width (mm)	B Width (mm)	Axial Expansion (mm)
A	5050	4750	300	4570	4554	16	612	640	4.5
B	4820	4740	80	4578	4512	66	610	632.5	5.3
C	4860	4750	110	4576	4485	91	611	635.5	7.6
E	2070	2000	70	3979	3922	57	402	1670	10.4

Table 7. Shape and dimension of steel fiber

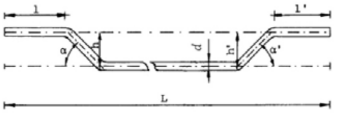
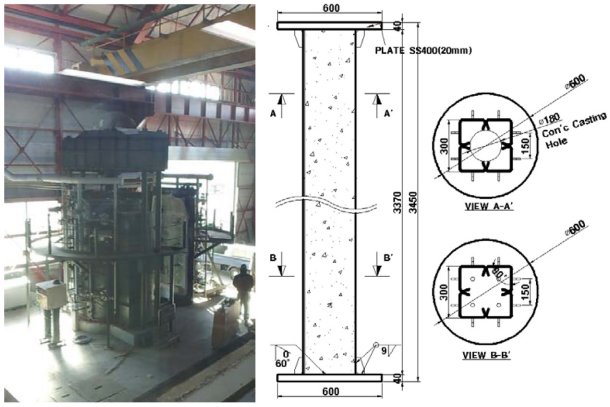
Shape		
	L_f (mm)	60
	D_f (mm)	0.75
	$V_f = l_f / d_f$	80
	Density (g/mm^3)	0.0078

Table 8. Properties of steel fiber of different curing ages

Result of Material test	days	Mixing Ratio(%)		
		0	0.25	0.375
Compressive Strength (MPa)	7	19	23	26
	14	37	30	35
	28	41	42	46
Tensile Strength (MPa)	7	1.7	1.7	2.5
	14	2.3	2.6	2.9
	28	3.0	3.5	3.5
Bending Strength (MPa)	7	2.4	2.5	2.9
	14	2.6	3.7	4.1
	28	2.8	2.9	4.4

9 summarizes the properties of the specimens. Installation and location of thermo couples, location and dimension



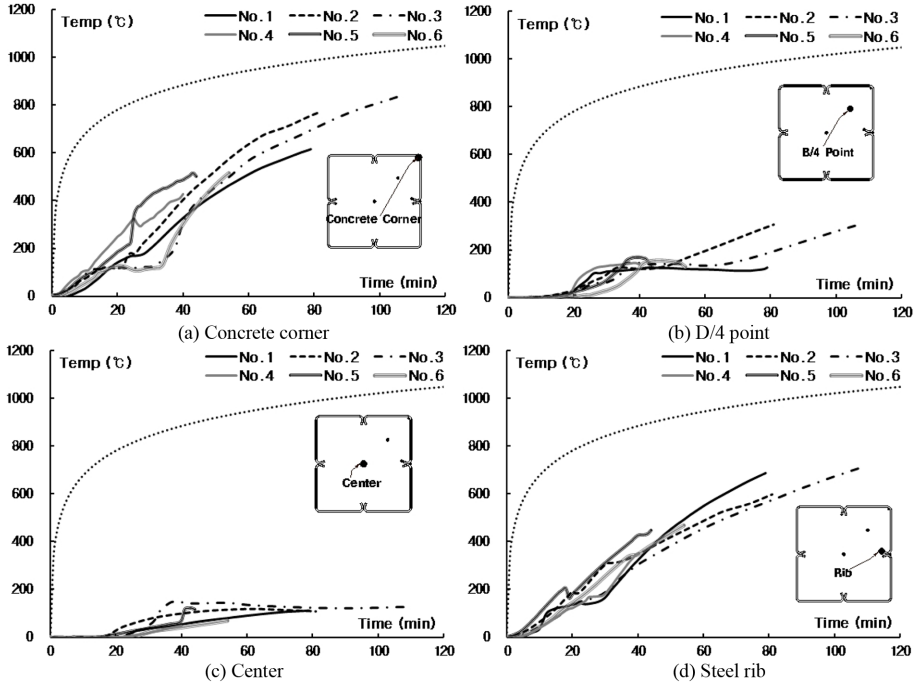
(a) 3,000 kN heating furnace (b) Specimens details

Figure 8. Heating furnace, specimen details and location of thermo couples.

of vent holes and standard heating curve (Eq. (3)) were the same with the test of the columns without fire protection. A 300 ton heating furnace shown in Fig. 8(a) was used in the test. End-plates (600×600×40 mm) were placed at the top and bottom of the specimens to place them at the hea-

Table 9. ACT specimens reinforced with steel fiber

Type	Steel fiber Mixing Ratio	B (mm)	t (mm)	L (mm)	Load Ratio	Test Load C (kN)
1	0	300	6	3449	0.35	1132
2	0.25	300	6	3449	0.35	1132
3	0.375	300	6	3448	0.35	1132
4	0	300	6	3449	0.5	1617
5	0.25	300	6	3449	0.5	1617
6	0.375	300	6	3445	0.5	1617



(a) Concrete corner (b) D/4 point

(c) Center (d) Steel rib

Figure 9. Temperature of steel fiber reinforced ACT Columns.

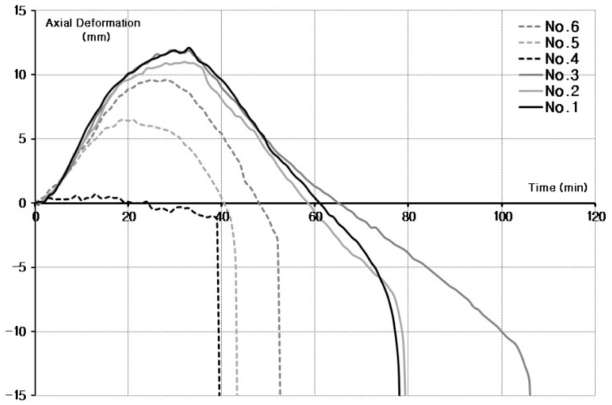


Figure 10. Axial deformation of Steel-fiber reinforced ACT columns.

ting furnace. Figure 8(b) shows specimen details.

3.2.2. Experiment results

Table 9 shows the temperatures at the different locations of the concrete. The temperatures at each point were similar among the 6 specimens, meaning that mixing ratio and load ratio do not have a material influence on temperature distribution. The temperatures inside the concretes were below 200 as shown in Figs. 9(b) and (c). The temperatures at the corners of the concrete shown in Fig. 9(c) were higher than those at the interior anchors shown in Fig. 9(d). It is deduced that the thermal storage effect of the concrete mitigates temperature rise at the interior anchors.

Figure 10 shows the axial deformation of the ACT columns reinforced with steel fiber. It is observed from the curves that higher mixing ratio leads to more gradual contraction and improved fire resistance performance. Specimens 4, 5 and 6 where load ratio was 0.5 resisted fire for less than 60 minutes, whereas fire resistance duration of the specimens 1, 2 and 3 where load ratio was 0.35 was over 60 minutes.

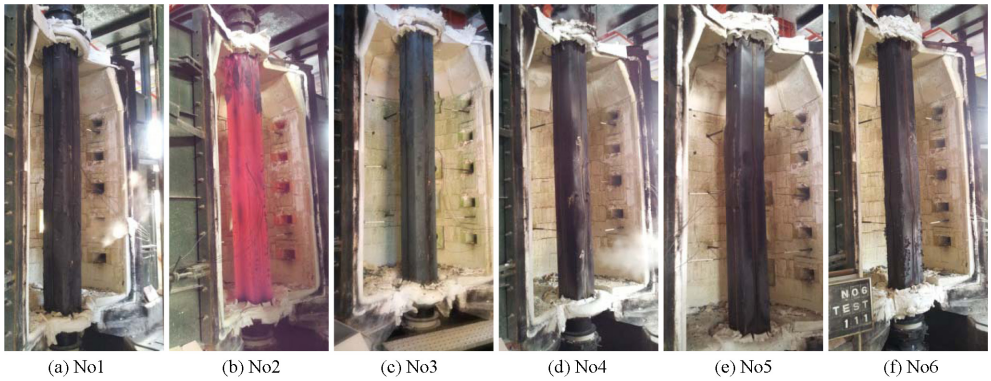
Table 10 summarizes the weight, length and width of the specimens before and after the test. Figure 11 shows the failure mode of the specimens at the termination of the test. The average differences in weight and length were approximately 50 Kg and 60 mm. The steel tube of specimen 4 hardly expanded, but it seems to have been caused by the defection of the tube. Additional review and analysis are needed to find the reason.

3.2.3. Analysis and discussion

Figure 12 shows non-dimensional expression of the fire resistance performance of the ACT columns reinforced with steel fiber. When the load ratio was 0.35, reinforcing the concrete with 0.375% steel fiber improved fire resistance by 35%. When the load ratio was 0.5, fire resistance was improved by 29%. However, reinforcing with 0.25% steel fiber did not make significant improvement in fire resistance. It is deduced that steel fiber mixing ratio of more than 0.375% improves fire resistance performance of the columns because the steel fiber mixed with the concrete prevents cracks and deals with a certain degree of load.

Table 10. The dimensions of ACT columns reinforced with steel fiber before and after heating (A: Before heating, B: After heating)

	A Weight (kg)	B Weight (kg)	Weight gap (kg)	A Length (mm)	B Length (mm)	Length gap (mm)	A Width (mm)	B Width (mm)	Axial Expansion (mm)
1	1036	984	52	3449	3388	52	300	312	12
2	1032	968	64	3449	3380	70	302	315	11
3	1042	1000	42	3448	3378	70	300	317	12
4	1036	1008	28	3449	3390	60	300	319	0.7
5	1030	984	46	3449	3380	70	301	320	7
6	1042	1000	42	3445	3382	63	301	318	10



(a) No1 (b) No2 (c) No3 (d) No4 (e) No5 (f) No6

Figure 11. Failure mode of the specimens reinforced with steel fiber.

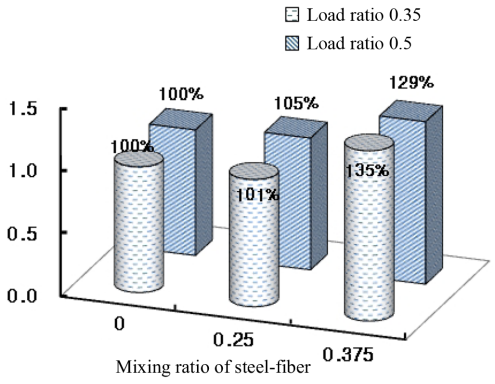


Figure 12. Fire resistance performance associated with different mixing ratios.

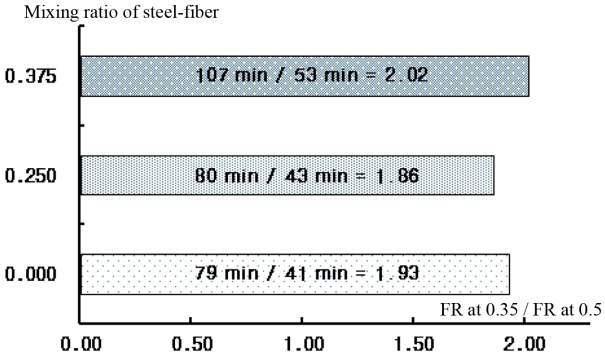


Figure 13. Fire resistance performance associated with different load ratios.

Figure 13 shows the fire resistance of the columns having the same mixing ratio to see the influence of load ratio. When the load ratio was adjusted downward from 0.5 to 0.35, fire resistance was almost doubled. In particular, the fire resistance improved more than twice as much when the mixing ratio was 0.375, showing that load ratio is the major factor in determining fire resistance performance. The longest fire resistance duration observed in the ACT columns reinforced with steel fiber was 107 minutes. It is deduced that the fire resistance for 120 minutes can be secured if the load ratio is adjusted downward.

3.3. Fire-proof paint column (3 hours)

The columns employed in high-rise buildings should satisfy 3 hour fire resistance requirement. However, it is difficult to secure fire resistance for 3 hours only with steel bar or steel fiber reinforcement because high-rise buildings should deal with greater load. Therefore, fire resistance paint was applied to the ACT columns and their fire resistance was examined. While this method of reinforcement has advantages such as no requirement of finishing touches after painting and easy application of the paint even to joint ends and small spaces, it costs more than fire resistance spray or board. Therefore, thinner layer of the fire resistance paint was applied. Table 11 shows the specimen details. The strength of the members, location of thermo couples, equipment and standard heating curve used in the 2 specimens were the same with those in non-protected specimen B. Figure 14 shows the application of the paint and the specimen after application.



(a) Painting of column (b) Boundary condition

Figure 14. Application of fire resistance paint and boundary condition.

3.3.1. Experiment results

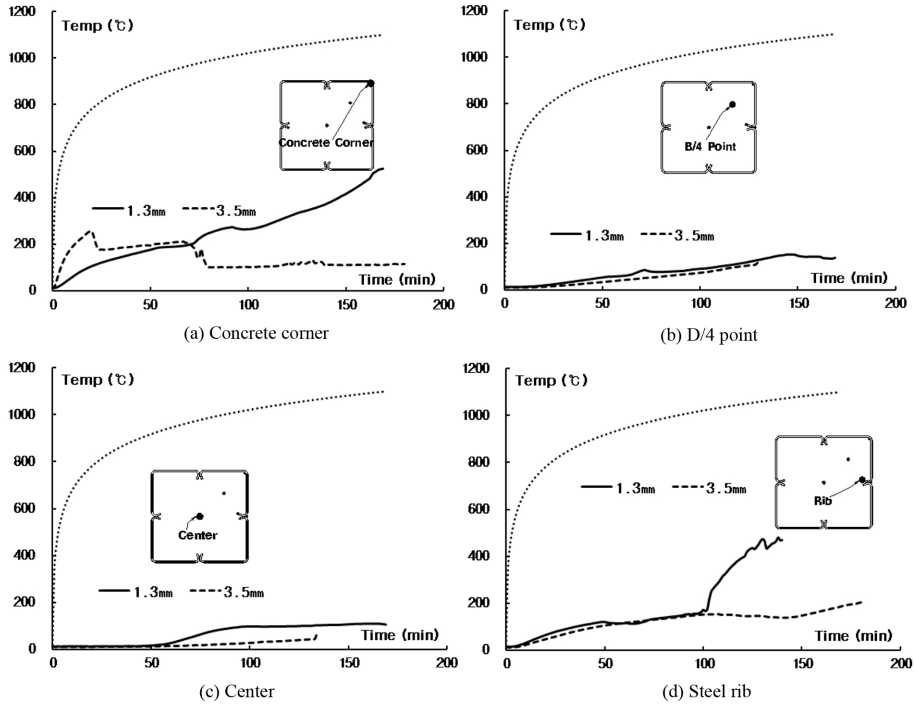
The graphs in Fig. 15 show the temperatures of the ACT columns reinforced with fire resistance paint. It is shown that the temperatures are significantly lower than those observed in the specimens without fire protection and those reinforced with steel fiber. The temperatures were below 400°C even after 120 minutes. Temperature started to rise after 120 minutes in the specimen with 1.3 mm-thick-layer of fire resistance paint. The highest temperature was lower than 100°C in the specimen with 3.5 mm-thick-layer of the paint.

As shown in the Fig. 16 of axial deformation-time relationship graphs, the specimen with 3.5 mm-thick-layer of fire resistance paint displayed gradual axial expansion for the 180 minutes of heating and the steel tube did not contract. As the fire resistance paint was exposed to fire, it formed fire-blocking layers. The layers turned white and finally became like black lumps of charcoals. Since temperature rise was delayed, the specimen resisted fire for 180 minutes. In the specimen with 1.3 mm-thick-layer of fire resistance paint, the steel tube expanded gradually and resisted fire for 170 minutes.

As shown in Fig. 17, the center of the specimen with

Table 11. ACT specimens reinforced with fire resistance paint

Type	Thickness of Paint cover	*B* (mm)	*t* (mm)	*L* (mm)	Load Ratio	Test Load *C* (kN)
B-1	1.3 mm	616	10	4576	0.6	8972
B-2	3.5 mm	616	10	4579	0.6	8972



(a) Concrete corner (b) D/4 point

(c) Center (d) Steel rib

Figure 15. Temperature of ACT columns with fire resistance painting.

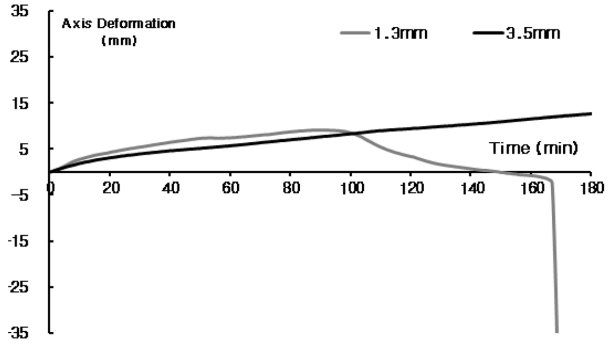


Figure 16. Axial Deformation of ACT columns with fire resistance painting.



(a) 1.3 mm (b) 3.5 mm (c) Tumescent depth

Figure 17. Failure mode of ACT columns with fire resistance painting.

1.3 mm-thick-layer of fire resistance paint turned black, while the 3.5 mm-thick-layer of fire resistance paint swelled with tiny cracks on it. In the latter, the swell was about 14 mm deep, indicating fire resistance duration of over 180 minutes. It is deduced that 1.8~3.5 mm-thick-layer of fire resistance paint is appropriate for economically efficient fire resistance design. Additional studies should be conducted to find the optimal thickness of fire resistance painting. For the verification method, before the simulation analysis between the result of the experiment and variable interpretation is needed. The prime variable is thickness of the steel tube, since the effect of reinforced fire resistance paint in concrete-filled steel tube and general steel tube is different. Therefore, the thickness of the paint can differ by thermal conduction of the steel tube. These results could not be included in this study, but continuous research will be in progress.

4. Conclusion

In this study, ACT columns without fire protection and those reinforced with steel fiber or fire resistance paint were fabricated for loaded heating test to verify the fire resistance performance of interior anchor type CFT columns. The conclusion of the study to enable economically efficient fire resistance design is as follows.

1) In the ACT specimens without fire protection, the interior anchor was exposed to high temperatures and the fire resistance performance of the columns was deteriorated due to thermal expansion. Since the cracks at the concrete reduced contraction rapidly, load ratio should be adjusted downward to below 0.5 to secure the fire resistance duration of more than 60 minutes.

2) As the amount of steel fiber mixed with the concrete

increased, the contraction of the columns was more gradual and fire resistance performance was improved. It was found that the steel fiber deals with the load after thermal expansion and delays the cracking at the concrete.

3) The temperatures at the ACT columns reinforced with fire resistance paint were stable and noticeably lower until 180 minutes compared with the other 2 groups of specimens. A member with 3.5 mm thickness of the fire-resistant paint has shown only axial expansion for 180 minutes and constructional deformation has not appeared. In other words, strength reduction by high-temperature has not happened and could get satisfying result of three hours of fire resistive performance.

4) Effective measure to satisfy the planned fire-resistant time for each ACT columns is needed. To verify the possibility to be realized, evaluating the outcome of one hour (non-fire protection), two hours (steel fiber reinforced), and three hours (fire resistance paint) has been done and it has definitely shown the potential of fire-resistant ability.

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Notation

FR = Fire Resistance (min) ≤ 180 min
f'_c = Compressive of Concrete (20~55 MPa)
KL = Effective Buckling Length (2000~4500 mm)
D = Width or Diameter of Column (141.3~404.6 mm)
C = Applied Load (kN)
f_1 = Factor of Aggregate (Siliceous: 0.06, Carbonate: 0.07)
ϕ_c = 0.60
A_c = Area of Concrete Section (mm^2)
$\lambda_c = \frac{KL}{r_c}\sqrt{\frac{f'_c}{\pi^2 E_c}}$
γ_c = radius of gyration
E_c = Coefficient of Elasticity of Concrete
T = Heating Temperature (°C)
t = time (min)
L_f = Length of Steel fiber
D_f = Diameter of Steel fiber
V_f = Reinforcing Index ($V_f = l_f / d_f$)