




# An Experimental Evaluation on the Fire Resistance and Post-fire Structural Performance of Concrete Filled Steel Tube Columns

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## Abstract

In this study, the test was conducted in two steps to evaluate the fire resistance performance and post-fire structural performance of the concrete-filled steel tube column (CFT column). As a first step, fire resistance tests were conducted on a CFT column with fire-resistant paint applied. As a second step, structural performance tests were conducted on the identical test specimens after 2–4 months of fire resistance test to evaluate the residual strength after the fire. As a result of the fire resistance test, it has been confirmed that almost the same fire resistance performance is secured even if the fire-resistant paint is applied with 60% of the thickness of the fire-resistant paint (2.7 mm), which has been recognized for its 2 h fire resistance performance. A post-fire structural performance test showed that the ratio of the maximum load versus the nominal compressive strength was 1.17 on average, and the strength reduction was about 10% compared to the test specimen without fire damage. Therefore, it is judged that the CFT column (D-Column), which has secured the fire resistance performance by applying the fire-resistant paint, will not significantly degrade the structural performance even after the fire.

**Keywords** Concrete filled steel tube column · Fire resistance test · Structural performance test · Residual strength

## 1 Introduction

The causes of fires are various, including electricity, gas, and arson, and a considerable number of fires occur every year. Fires can cause buildings to collapse along with human and economic damage. In performance-based structural design for fire, the fire resistance design standards goal is life safety,

and it is to prevent the structure from collapsing due to fire in the structural design process. In addition, fires cause significant economic damage as well as human damage. If structural performance is maintained after the fire, it is possible to minimize economic damage by reusing structural members.

Since composite columns used in building structures are often applied to large-scale buildings, it is essential to secure fire resistance performance (Choi et al., 2021). With the recent number of fires in large buildings such as high-rise buildings and logistics warehouses, interest in the fire resistance performance of composite columns is increasing. Among the composite columns, concrete-filled steel tube columns (from now on referred to as CFT columns) in which concrete is filled inside the steel tube have recently been increasing due to the prevention of local buckling of the steel tube and improvement of workability (Choi et al., 2019, 2020; Park et al., 2007). The CFT column has improved fire resistance performance due to the heat storage effect of concrete inside the steel tube (Ahn & Lee, 2014; Park et al., 2000).

Cho et al. (2010) conducted a loading fire resistance test on the uncovered CFT column. Kwon (2010) worked on a loading fire resistance test on the uncovered CFT column

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and a study on the fire resistance performance of the CFT column through finite element analysis.

CFT columns require fire resistive covering material when steel plates are exposed. Fire-resistive covering materials include fire-resistant spray, paint, and fire-resistant boards. Kim and Lee (2010) performed a fire resistance test with fire-resistant spray and paint. In addition, Kim (2017) conducted a study on the fire resistance performance of the CFT column to which fire-resistant paint was applied by conducting loading and non-loading fire resistance test.

Several researchers conducted studies on the residual strength of building members after a fire. Liu et al. (2018) and Alhatmey et al. (2018) studied the post-fire performance of circular CFT columns. In addition, Hassani et al. (2021) studied the post-fire performance of CFT columns with different steel sections.

In this study, fire resistance tests were conducted on the concrete-filled steel tube column applied with fire-resistant paint. Structural performance tests were conducted on the test specimens after the fire resistance test to evaluate the structural performance and residual strength after the fire.

## 2 Design of Experiment

The experiments were conducted in two steps to evaluate the structural performance after the fire of the CFT columns. The first step was a test on the fire resistance performance of CFT columns, which were divided into loading and non-loading fire resistance tests. It produced the situation in which CFT columns with fire-resistant paint were directly exposed to fire. Two to four months after the fire resistance

tests, structural performance tests were conducted on the identical test specimens for the residual strength evaluation in the second step.

### 2.1 Plan for Test Specimens

The fire resistance and structural performance tests were conducted on the identical test specimens, and as shown in Table 1, six test specimens were planned. The variables are the fire-resistant paint's thickness, the fire resistance test's type, and whether the structural performance test is performed. The P1.5-NN-C is a test specimen that did not conduct a fire resistance test to compare the structural performance before and after the fire resistance test but only performed the structural performance test.

The shape of the test specimen is shown in Fig. 1. The test specimens are CFT columns in the size of  $D-300 \times 300 \times 6$  (width  $\times$  length  $\times$  thickness, mm) manufactured by complete joint penetration welding SM355A steel tube with a thickness of 6 mm and height of 3040 mm. Concrete was filled inside the steel tube, and the design standard strength of the concrete was 24 MPa. All six test specimens were manufactured in the same shape and size, and a fire resistance test was conducted with different fire-resistant paint thicknesses.

### 2.2 Plan for Fire Resistance Test

Fire resistance tests were conducted to evaluate the fire resistance performance of the CFT columns and reproduce the situation exposed to the fire. In Korea, fire resistance performance for 2 h is required when a building has more than 4 floors and less than 12 floors. In this study,

**Table 1** Plan of test specimens

No.	Specimens	Size and material	Paint thickness	Fire resistance test	Structural performance test
1	P1.3-FN-C	Size : □-300x300x6 Steel : SM355A ( $F_y = 355$ MPa) Concrete : $f_{ck} = 24$ MPa	1.3	Non-loading test	O
2	P1.5-FN-C		1.5	Non-loading test	O
3	P1.5-NN-C		1.5	-	O
4	P1.7-FN-C		1.7	Non-loading test	O
5	P1.7-FN-N		1.7	Non-loading test	X
6	P1.7-FL-C		1.7	Loading test	O

P1.3-FN-C

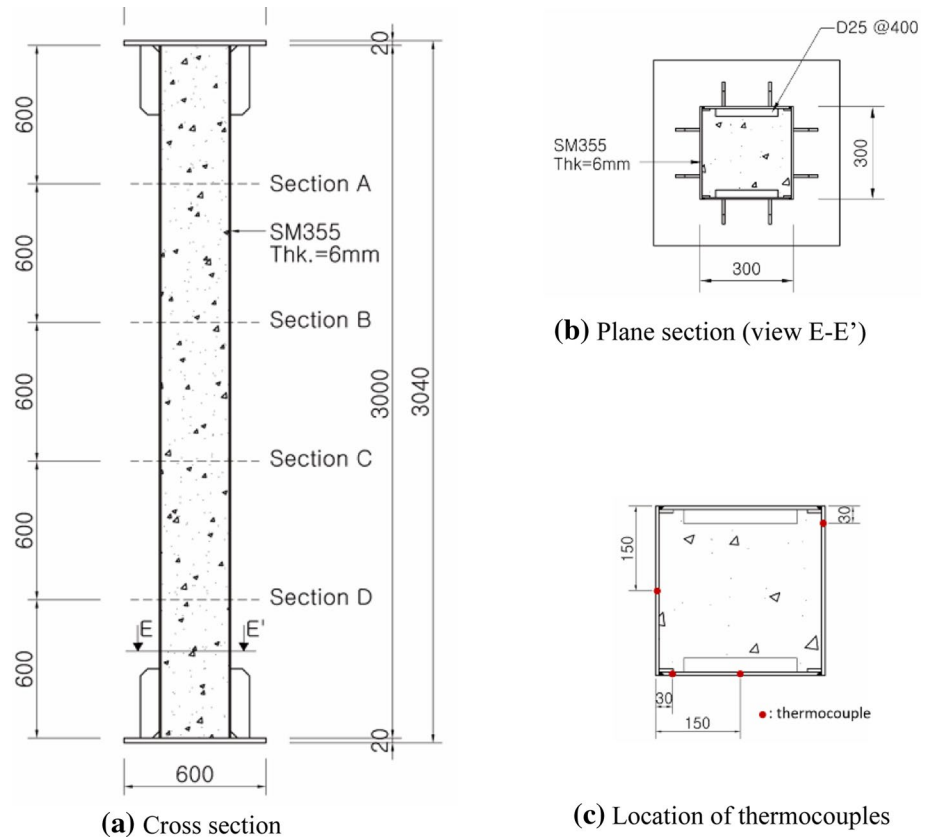
C : Compression structural performance test  
 N : No compression structural performance test  
 FN : Fire resistance test \_ Non-loading test  
 FL : Fire resistance test \_ Loading test  
 NN : No fire resistance test

P1.3 : Paint thickness 1.3mm

P1.5 : Paint thickness 1.5mm

P1.7 : Paint thickness 1.7mm

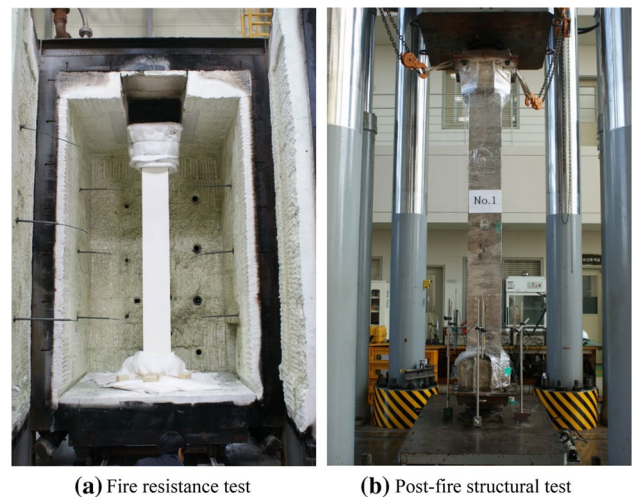
**Fig. 1** Plan of specimens and location of thermocouples



fire resistance tests were conducted based on the fire resistance performance of the CFT column for 2 h, and fire-resistant paint was applied as fire-resistive covering material. The thickness of the fire-resistant paint, which was recognized as the fire resistance structure for 2 h, was generally 2.7 mm. However, in this study, the thickness of fire-resistant paint was applied from 1.3 to 1.7 mm through the performance test performed in advance.

The fire resistance tests were conducted by KS F 2257-1(2019) and KS F 2257-7(2014) and were divided into a non-loading test and a loading test. In the fire resistance tests, after installing the test specimens, as shown in Fig. 2a, the test specimens were heated for 2 h according to the standard fire temperature–time curve in Fig. 3. In order to measure the temperature of the steel tube, 16 thermocouples were installed on cross-sections of A, B, C, and D in Fig. 1a, four as shown in Fig. 1c.

The loading test was a fire resistance test in which a constant load was applied with UTM inside the heating furnace, and the shortening amount of the test specimen was measured as the displacement of UTM. As for the loading, 800 kN, which is 30% of the value  $R_a$  obtained by dividing the nominal compressive strength ( $R_n$ ) by the safety factor ( $\Omega$ ), was applied as shown in Eq. 1. Nominal compressive strength was calculated according to 0709.2.2 of KBC 2016.



**Fig. 2** Set-up of specimens

$$R_a \leq \frac{R_n}{\Omega} \quad (1)$$

$R_a$ : required strength  
 $R_n$ : nominal strength  
 $\Omega$ : safety factor

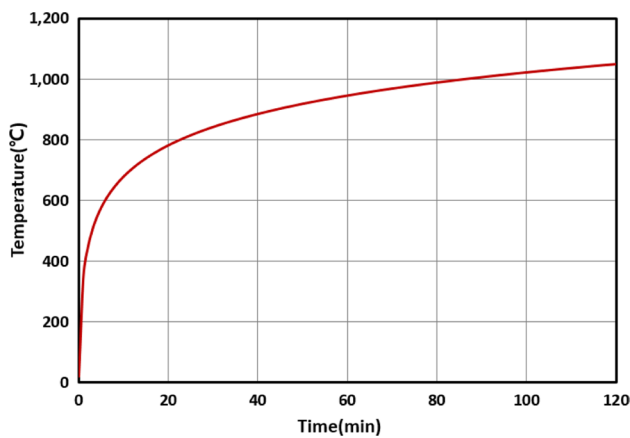


Fig.3 Standard fire temperature–time curve

### 2.3 Plan for Post-Fire Structural Performance Test

In order to evaluate the structural performance of the CFT columns after a fire, structural performance tests were conducted on the identical test specimens 2 to 4 months after the fire resistance test. Since the P1.7-FN-N test specimen has the same conditions as the P1.7-FN-C test specimen, the structural performance test was not conducted, and concrete test pieces and steel tensile test pieces were collected to confirm the material characteristics after a fire.

In the structural performance tests of the CFT columns, as shown in Fig. 2b, test specimens were installed using pinpoint jigs at the top & bottom and compressed with 10 MN UTM. In addition, displacement was measured by installing displacement meters on the four sides of the specimens.

## 3 Results of Fire Resistance Test

After the 2-h fire resistance test by the standard fire temperature–time curve was completed, the shapes of the test specimens were shown in Fig. 4, and fire resistance performance was ensured as the fire-resistant paint expanded. The results of the fire resistance tests are summarized in Table 2. The results of the non-loading tests were summarized as the maximum temperature and average temperature of the test specimens until the end of the fire resistance test, and the results of the loading test were summarized as the maximum deformation and maximum deformation rate.

### 3.1 Results of Non-loading Fire Resistance Test

As a result of the non-loading fire resistance test, when the fire-resistant paint was covered, the maximum temperature was 417 to 634 °C, and the average cross-sectional temperature was 398 °C to 541 °C. Considering that the heating



(a) Non-loading test (b) Loading test

Fig. 4 Specimens after fire resistance test

temperature at 120 min is 1049 °C, the maximum temperature is 60% or less, and the average temperature is 50% or less.

In addition, the fire performance standard of the non-loading test at KS F 2257–7(2014) should not exceed the average cross-sectional temperature of 538°C and the maximum temperature of 649 °C of steel. The P1.3-FN-C and P1.5-FN-C test specimens showed that the average cross-sectional temperature exceeded 538°C, but the P1.7-FN-C and P1.7-FN-N test specimens were found to satisfy the fire performance criteria. It can be confirmed that the test specimens with fire-resistant paint thickness of 1.7 mm have a maximum temperature and an average cross-sectional temperature lower than the test specimens with fire-resistant paint thickness of 1.3 mm and 1.5 mm. Thus the temperature is reduced in proportion to the thickness of the fire-resistant paint.

Figure 5 shows the temperature–time curves of the test specimens conducted for the non-loading test as the maximum temperature and the average temperature of each

Table 2 Fire resistance test results

No	Specimens	Non-loading test		Loading test	
		Average temperature	Maximum temperature	Maximum deformation	Maximum deformation rate
1	P1.3-FN-C	541.1 °C	634.0 °C	–	–
2	P1.5-FN-C	541.0 °C	617.4 °C	–	–
3	P1.5-FN-N	–	–	–	–
4	P1.7-FN-C	410.5 °C	437.1 °C	–	–
5	P1.7-FN-N	398.2 °C	416.8 °C	–	–
6	P1.7-FL-C	–	–	12.56 mm	0.216 mm/min

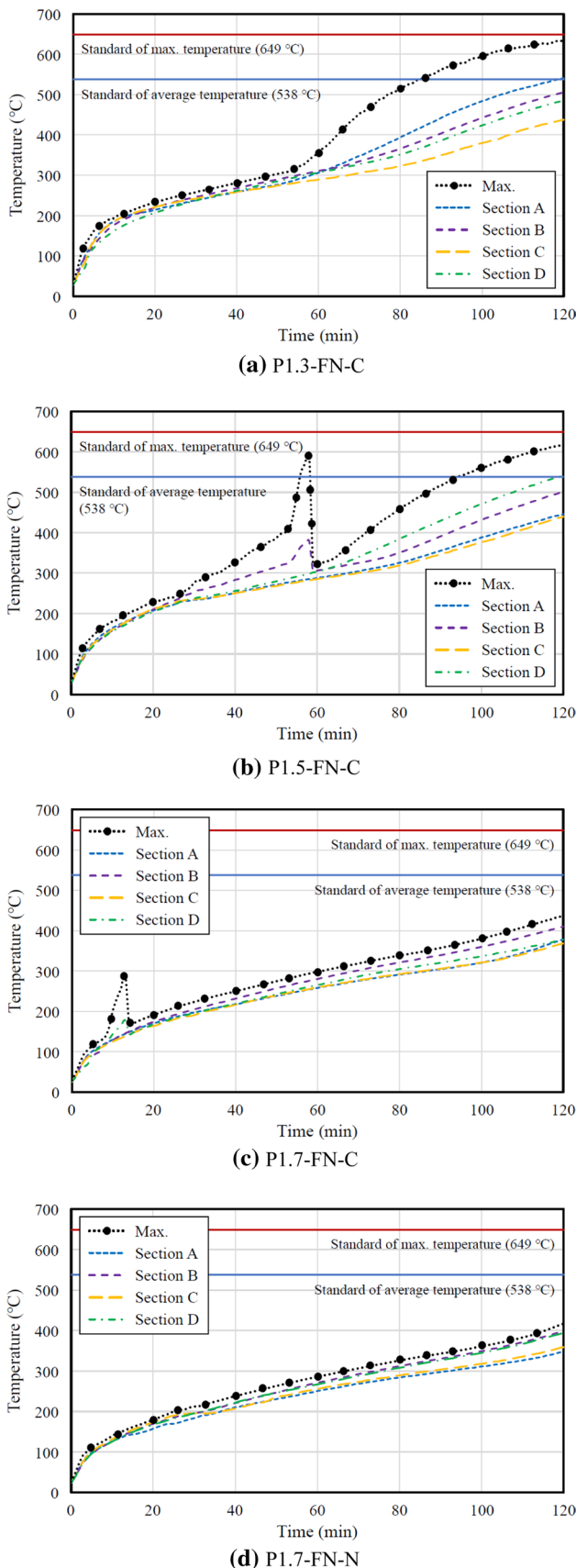


Fig. 5 Temperature–time curve

cross-section. In Fig. 5, the fire resistance performance standard, the average cross-sectional temperature (538 °C), and the maximum temperature (649 °C) are shown as solid lines.

In Fig. 5a, in the case of the P1.3-FN-C test specimen, the maximum temperature after 54 min was slightly higher than the average cross-sectional temperature. In Fig. 5b, in the case of the P1.5-FN-C test specimen, the maximum temperature increased rapidly at 54 min and then decreased to a degree similar to the average cross-sectional temperature at 59 min and then slightly higher than the average cross-sectional temperature after 60 min. It is estimated that the rapid temperature rise is due to the instability of some thermocouples.

In Fig. 5c, for the P1.7-FN-C test specimen, the maximum temperature increased rapidly in 9 min and decreased to a degree similar to the average cross-sectional temperature in 14 min. In Fig. 5d, in the case of the P1.7-FN-N test specimen, the maximum temperature was similar to the average cross-sectional temperature. Therefore, it is determined that the temperature rise of the test specimens covered with fire-resistant paint thickness of 1.7 mm is stable.

It is determined that the steel temperature can be predicted if fire resistance performance is ensured by covering the fire-resistant paint on the CFT column through the non-loading fire resistance test results. In addition, it is judged that the temperature estimation can be used to evaluate the structural performance of the CFT column after a fire.

### 3.2 Results of Loading Fire Resistance Test

In KS F 2257–1(2019), the fire performance criteria of the loading fire resistance test are evaluated as unsuitable when both deformation ( $C$ ) and deformation rate ( $dC/dt$ ) are exceeded, and the deformation and deformation rate is calculated as in Eqs. (2 and 3).

$$C = \frac{h}{100} \tag{2}$$

$$\frac{dC}{dt} = \frac{3h}{1000} \tag{3}$$

$h$ : height of test specimen.

Figure 6 shows the deformation-time curve and deformation rate-time curve of the test specimen for the loading fire resistance test and the performance criteria of the deformation and deformation rate as a solid line. As a result of the loading test, the maximum deformation was 12.56 mm, and the maximum deformation rate was 0.216 mm/min, which satisfied the fire performance standard.

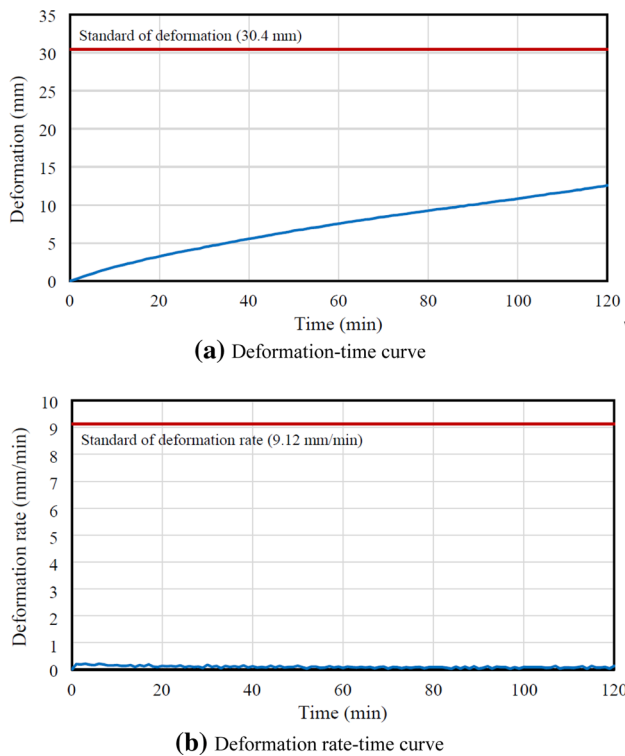


Fig. 6 Deformation and Deformation rate-time curve

## 4 Results of Post-Fire Structural Performance Test

### 4.1 Results of Material Test

Material tests were conducted to analyze the material characteristics of the test specimens. The test results are shown in Tables 3 and 4, Table 3 is the concrete compressive strength test result, and Table 4 is the steel tensile test result.

In Tables 3 and 4, the original is the test result of the material used in manufacturing the test specimens and represents the material characteristics before the fire resistance test. Post-fire is the test result of concrete test pieces and steel tensile test pieces collected from the P1.7-FN-N test specimen after the fire resistance test. It represents the material characteristics after four months of the fire resistance test.

Table 3 Material test results of concrete

Concrete test piece	$f_{ck}$	Compressive strength test results
Original	24 MPa	26.58 MPa
Post-fire		21.49 MPa

Table 4 Material test results of steel

Steel tensile test piece	Thickness	Yield strength	Tensile strength	Elongation (%)
Original	6 mm	443.4 MPa	529.1 MPa	21.52
Post-fire		441.9 MPa	517.5 MPa	23.67

The original in Table 3 shows the average compressive strength for the two concrete test pieces, which was 26.58 MPa. Post-fire in Table 3 shows the average compressive strength for four steel tensile test pieces, which was 21.49 MPa. It is judged that the compressive strength of concrete has decreased by about 20% due to the fire.

The original and post-fire in Table 4 are the average values of each of the three tensile test pieces. After four months of fire resistance test, the mechanical properties of steel were not significantly different from before the fire resistance test. If the steel material temperature is about 400 °C in the event of a fire, it is judged that the strength reduction after the fire is not significant.

Figure 7 shows the stress–strain curve of the steel tensile test pieces. It was found that the stress–strain curve patterns of the original pieces and post-fire pieces were similar and formed the same yield point.

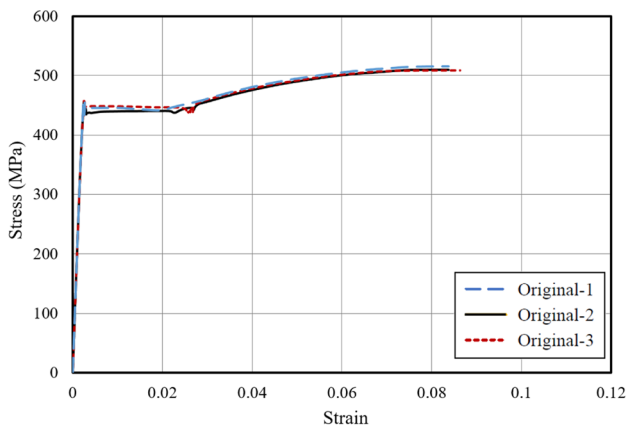
### 4.2 Results of Structural Performance Test after Fire Resistance Test

During the fire resistance test, the average cross-sectional temperature of the CFT columns reached 398 °C to 541 °C, and the structural performance test was conducted about two to four months after the fire resistance test. The results of the structural performance test after the fire are shown in Figs. 8 and 9 and Table 5.

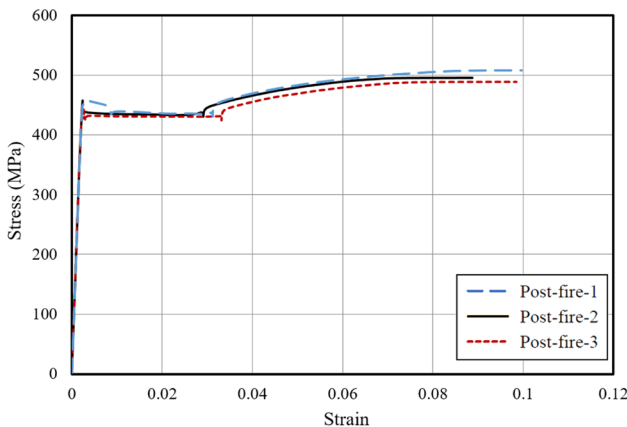
Figure 8 is the load–displacement curve of the test specimens conducted for the structural performance test. All test specimens show sufficient deformation capabilities, and the test specimens damaged by a fire show similar load–displacement curves compared to the P1.5-NN-C test specimen not conducted fire resistance test.

Figure 9 shows the destructive patterns of the test specimens. All test specimens were destroyed after local buckling, showing similar patterns of destruction. In the case of P1.3-FN-C and P1.7-FN-C test specimens, local buckling had occurred only in the upper part of the column.

In Table 5,  $P_{test}$  is the maximum load,  $\delta_{test}$  is the displacement at maximum load.  $P_{n1}$  is the nominal compressive strength based on the specified design strength.  $P_{n2}$  is the compressive strength based on the material test results of the original test pieces.  $P_{n3}$  is the compressive strength based on the material test results of post-fire test pieces. Nominal



(a) Original



(b) Post-fire

Fig. 7 Stress–strain curve of tensile specimens

compressive strength and compressive strength were calculated according to 0709.2.2 of KBC 2016.

In Table 5, the ratio of the maximum load to the nominal compressive strength ( $P_{test}/P_{n1}$ ) was 1.14 to 1.29, and the average was 1.19. Excluding the P1.5-NN-C test specimen not conducted fire resistance test, it was 1.14–1.21 and an average of 1.17. Thus, if fire-resistant paint is applied to ensure fire resistance performance, the average

temperature of the steel material will be about 500 °C during a fire so that the structural performance of the CFT column will not be significantly degraded even after a fire.

In addition, in Table 5, the ratio of the maximum load to the compressive strength based on the material test results of the original test pieces ( $P_{test}/P_{n2}$ ) was 0.97–1.10, and the average was 1.02. Excluding the P1.5-NN-C test specimen not conducted fire resistance test, it was 0.97–1.03, and an average of 1.00. Therefore, it was found that the compressive strength ( $P_{n2}$ ) showed almost the same strength as the structural performance after the fire resistance tests.

In Tables 3 and 4, the compressive strength of concrete test pieces after the fire resistance test decreased by about 20%, but the yield strength of steel tensile test pieces hardly decreased. As a result, the compressive strength based on the material test results of post-fire test pieces ( $P_{n3}$ ) decreased by 7% compared to the compressive strength based on the material test results of original test pieces ( $P_{n2}$ ).

In Table 5, the ratio of the maximum load to the compressive strength based on the material test results of post-fire test pieces ( $P_{test}/P_{n3}$ ) was 1.04 to 1.18, and the average was 1.09. Excluding the P1.5-NN-C test specimen not conducted fire resistance test, it was 1.04 to 1.11, and an average of 1.07.

Therefore, it is judged that the compressive strength based on the material test results of post-fire test pieces ( $P_{n3}$ ) reflects the structural performance well after the fire resistance tests. However, in most cases, the strength of the material cannot be confirmed for structures after a fire. Therefore, if the safety of the structural member is evaluated by nominal compressive strength based on the specified design strength, it is judged that the strength of the structural member after the fire can be conservatively evaluated.

The test specimens conducted the structural performance tests after the fire resistance tests showed a 6–12% reduction in strength compared to the test specimen conducted the structural performance test without a fire

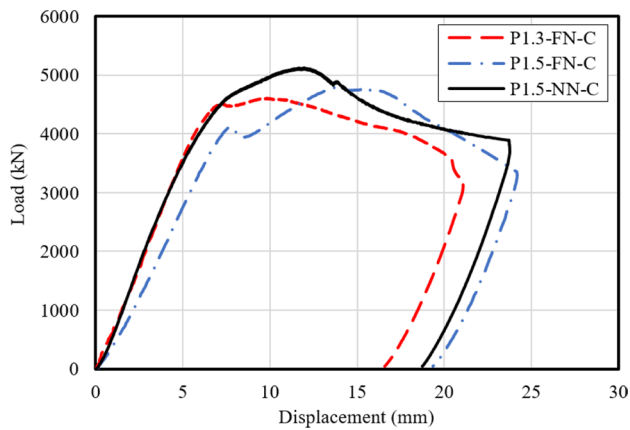
Table 5 Structural performance test results

No	Specimens	$P_{test}$ (kN)	$\delta_{test}$ (mm)	$P_{n1}$ (kN)	$P_{test}/P_{n1}$	$P_{n2}$ (kN)	$P_{test}/P_{n2}$	$P_{n3}$ (kN)	$P_{test}/P_{n3}$
1	P1.3-FN-C	4611.7	9.84	3957.2	1.17	4667.2	0.99	4338.8	1.06
2	P1.5-FN-C	4794.8	13.95	—	1.21	—	1.03	—	1.11
3	P1.5-NN-C	5117.8	11.92	—	1.29	—	1.10	—	1.18
4	P1.7-FN-C	4516.8	13.88	—	1.14	—	0.97	—	1.04
5	P1.7-FN-N	—	—	—	—	—	—	—	—
6	P1.7-FL-C	4601.2	10.81	—	1.16	—	0.99	—	1.06

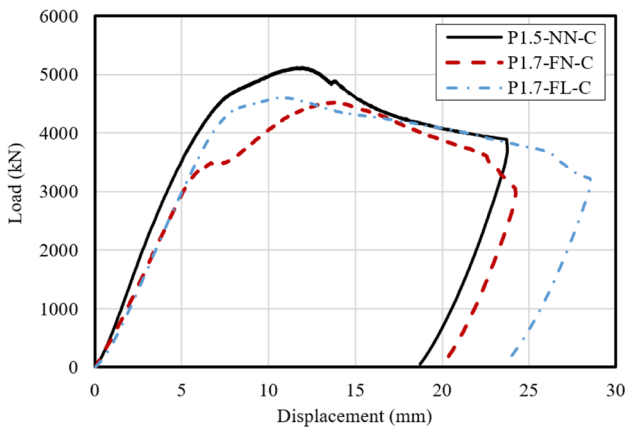
$P_{n1}$ : nominal compressive strength based on the specified design strength

$P_{n2}$ : compressive strength based on the material test results of original

$P_{n3}$ : compressive strength based on the material test results of post-fire



(a) P1.3-FN-C, P1.5-FN-C, P1.5-NN-C



(b) P1.5-NN-C, P1.7-FN-C, P1.7-FL-C

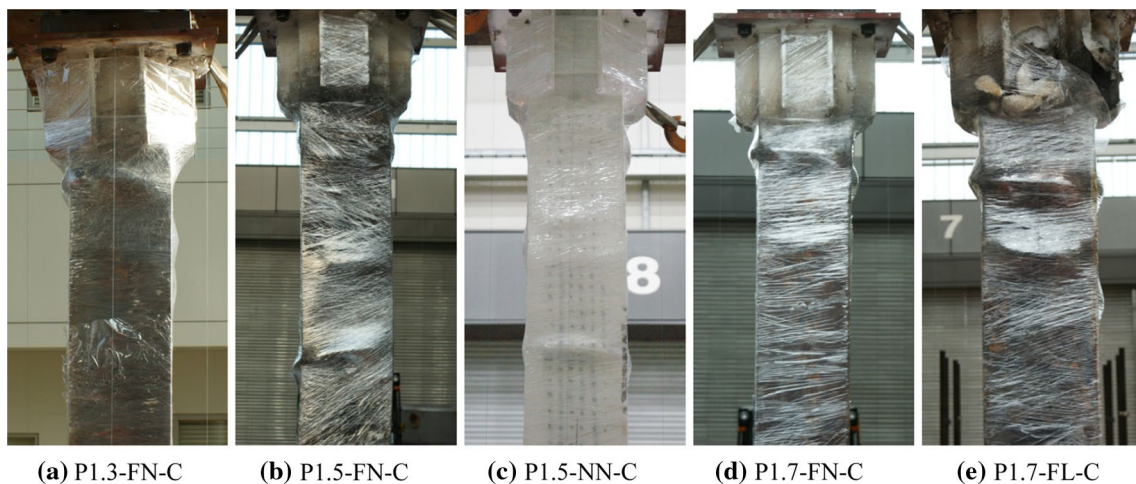
Fig. 8 Load–displacement curve

resistance test. Therefore, if the average temperature of the member rises to about 500 °C during a fire, the strength reduction after the fire is not considered significant, and strength recovery over time can be expected.

### 5 Conclusions

Fire resistance tests were conducted on the concrete-filled steel tube column covered with fire-resistant paint for 2 h, and structural performance tests were conducted after the fire resistance test. The test results were summarized, and the following conclusions were drawn.

- 1) As a result of the non-loading fire resistance tests of the CFT columns, the maximum temperature was 417 °C to 634 °C, and the average cross-sectional temperature was 398 °C to 541 °C. When fire resistance performance is ensured by covering with fire-resistant paint on the CFT column, it is judged that it can be used to evaluate the structural performance after a fire through temperature estimation of steel.
- 2) As a result of the structural performance tests after the fire resistance tests, the ratio of the maximum load to the nominal compressive strength based on the specified design strength ( $P_{test}/P_{n1}$ ) was an average of 1.17. The ratio of the maximum load to the compressive strength based on the material test results of original test pieces ( $P_{test}/P_{n2}$ ) was an average of 1.00. The ratio of the maximum load to the compressive strength based on the material test results of post-fire test pieces ( $P_{test}/P_{n3}$ ) was an average of 1.07 on average. Therefore, it is judged that the strength of the structural members can be evaluated without reflecting the material characteristics of concrete and steel affected by the fire.



(a) P1.3-FN-C (b) P1.5-FN-C (c) P1.5-NN-C (d) P1.7-FN-C (e) P1.7-FL-C

Fig. 9 Local buckling of specimens



- 3) The test specimens that conducted the structural performance tests after the fire resistance tests showed a reduction in strength by about 10% (6%–12%) compared to the test specimen that conducted the structural performance tests without fire resistance tests. Therefore, if it is confirmed that the average temperature of the steel material is around 500 °C in the event of a fire, it is judged that the stability of the structural member can be evaluated with the nominal compressive strength based on the specified design strength.
- 4) The material test results after the fire resistance test showed that the compressive strength of concrete test pieces decreased by about 20% compared to the material test results without the fire resistance test. However, there was no significant difference in the material characteristics of steel tensile test pieces. In addition, the yield points in the stress–strain curve of the steel tensile test pieces were similarly formed.

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