Experimental investigation of high strength reinforced concrete columns damaged by fire and retrofitted with CFRP jackets

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Abstract. This paper presents the compressive behaviour of high strength circular reinforced concrete (RC) columns exposed to fire following the ISO834 standard fire curve for 1 or 2 hours, air cooled, and then retrofitted with 2 layers of transverse carbon fibre reinforced polymer (CFRP) wraps. Total 5 circular RC columns (ϕ300mm x 1000mm) were tested to investigate the effects of fire duration and the effectiveness of CFRP jackets. The investigated compressive behaviour of columns included load-displacement relationship, CFRP load-strain response and failure mode. Large temperature gradient was observed when columns were exposed to fire. For the unretrofitted columns, the test results showed that the ultimate capacity and stiffness of fire-damaged RC columns was decreased due to the deterioration of concrete at elevated temperatures. Longer fire duration leads to more severe damaged on columns. CFRP jackets could effectively restore the stiffness and improve the ultimate capacity of fire-damaged circular columns.

1. Introduction
Fire leads to serious threat to people’s lives and properties. Reinforced concrete (RC) structure has relatively good fire resistance, compared with steel and wood structures, because concrete has low thermal conductivity, high heat capacity. RC structures are rarely collapsed within relatively short fire duration, because concrete could be considered as an effective fire resistance shield for the RC structure. Therefore, repairing and strengthening of fire-damaged RC structures have become popular in the last few decades for both environmental and economic benefits. Carbon fiber reinforced polymer (CFRP) jackets, which is lightweight, high strength, corrosion resisted, and easy constructed, has been widely used to repair and strengthen the existing RC structures. Along with the increasing number of skyscrapers, high strength concrete (HSC) takes a big share of market in the RC structure design, especially for columns. Therefore, it is valued to study the performance of RC columns with HSC exposed to fire and retrofitted with CFRP jackets. The traditional repair method requires to remove all loose concrete before applying CFRP jackets. However, it is time consuming and high labor cost. In this study, a fast repair method is introduced. Loose fire-damaged concrete was left in and the cross-section area was slightly enlarged to restore the circular shape for fire-damaged columns. High strength mortar was used to fulfill the spalling area and the gap between the original cross section and new cross section. The experimental results showed retrofitted fire-damaged columns with loose concrete left in and CFRP jackets could effectively restore and improve the ultimate capacity of fire-damaged circular columns.
2. Experimental Programme

A total of 5 circular RC columns were tested, and the objective of this experimental program is to investigate the performance of RHSC columns under different fire durations and evaluate the effectiveness of CFRP jackets for repairing fire-damaged circular RC columns.

2.1. Specimen Details

All 5 circular RC columns were identical with a 300mm diameter and a length of 1000mm. Columns were reinforced by six 16mm diameter longitudinal reinforcing bars with equal spacing throughout the cross-section, and confined by 6mm diameter spirals with 40mm spacing throughout the length. The clear concrete cover was 25mm for all columns. Figure 1 shows the cross-sectional details for the columns.

![Figure 1. Cross-sectional details of specimen](image)

All columns were casted at the same day, and the average concrete strength at the test day was 71.5MPa. Two columns were instrumented with 4 type-K thermocouples as shown in Figure 2. Three type-K thermocouples were embedded at concrete core, 50mm and 100mm from column core at mid-height respectively, and one was tied at the mid-height of a steel bar to record temperature distribution through heating and cooling procedures. Detailed description for each column is listed in Table 1.

![Figure 2. Thermocouples locations](image)

| Number | Label    | Descriptions                                           |
|--------|----------|--------------------------------------------------------|
| 1      | RT-R     | Reference column, no fire-damage and retrofit          |
| 2      | FD1-R    | 1hr fire-damaged, non-retrofitted                      |
| 3      | FD1-CJ   | 1hr fire-damaged and retrofitted with 2 layers of CFRP jacket |
| 4      | FD2-R    | 2hr fire-damaged, non-retrofitted                      |
| 5      | FD2-CJ   | 2hr fire-damaged and retrofitted with 2 layers of CFRP jacket |

2.2. Experimental Program

2.2.1. Heating and Cooling Cycle. 4 columns were exposed to fire for 1 or 2 hours in the gas furnace, and the temperature in the furnace followed the ISO834 standard fire curve [1]. The temperature in the furnace was controlled by two type-K thermocouples attached on the furnace walls, and the average of these two temperature represented the temperature in the furnace and the surface temperature for columns. After 1 or 2 hours fire exposure, the furnace was turned off, and the door was opened to let columns cool down to the ambient temperature by air. The maximum temperature at each location are plotted in Figure 3, and occurrence time for maximum temperature at each location are summarized in Table 2. It is observed that large temperature gradient was developed in the column. There is a long delay at column core to reach the maximum temperature. The maximum temperature for 2 hours fire exposed columns is twice than 1 hour fire exposed columns. Steel bars reached the maximum temperature almost at the same time as furnace. Concrete spalled off during fire exposure, so the temperature in steel bar changed along with the temperature in the furnace. It is observed that the temperature at 100mm from column core for 2 hour fire-damaged column ascended sharply compared with 1 hour fire-damaged column due to larger spalling off area.
Table 2. Maximum temperature and occurrence time

| Location         | Fire exposed for 1 hr | Fire exposed for 2 hrs |
|------------------|-----------------------|------------------------|
|                  | Max. temperature (°C) | Occurrence time (hr)   | Max. temperature (°C) | Occurrence time (hr) |
| Column core (cc) | 168.7                 | 3.22                   | 323.7                 | 4                     |
| 50mm from cc     | 178.4                 | 2.4                    | 335.1                 | 2.92                  |
| 100mm from cc    | 252.1                 | 1.49                   | 503.4                 | 2.17                  |
| Steel            | 380.3                 | 1.18                   | 585                   | 2.04                  |
| Furnace          | 797.1                 | 1.08                   | 950                   | 2.01                  |

Figure 3. Maximum temperature in columns at different locations

2.2.2. **Visual Inspections and Repair Process.** After fire exposure, visual inspections were performed to each column. It was observed that concrete cover was severely damaged and cracks were developed. Severely damaged concrete was peeled off, when fire-exposed columns were transported outside the furnace. All columns had spalling area at random locations. The columns with 2 hours fire duration show larger spalling area compared with 1 hour fire-damaged columns. It is observed that there is no concrete cover for part of 2 hour fire-damaged column as shown in Figure 5(a). The concrete cover peeled off, which means the bond between steel bars and concrete becomes very weak after fire exposure.
Figure 4. Side and cross-sectional views of 1 hour fire-exposed column

Figure 5. Side and cross-sectional views of 1 hour fire-exposed column
After heating and cooling procedures, loose concrete was left in fire-damaged columns, and oiled plastic tubes with 370mm diameter were used as mold to slightly enlarged cross-section to restore the circular shape for fire-damaged columns. High strength mortar was applied to fill out the spalling area and the gap between the original cross-section and the new cross-section. After mortar cured, two layers of uni-directional CFRP transverse wrappings were applied to fire-damaged columns. The new cross-section for retrofitted columns is shown in Figure 6. All repaired columns were cured at room temperature for one month before compressive tests.

![Cross-sectional details of retrofitted column](image)

**Figure 6. Cross-sectional details of retrofitted column**

### 2.2.3. Compressive Tests

After being repaired, all columns were tested to failure under axial compression using a testing machine with 10000kN capacity, and the loading rate was set as 1kN/s. Two steel plates with 300mm diameter were placed at top and bottom to make all columns have the same loading area.

### 2.2.4. Test Results and Discussions

Test results of the ultimate capacity for each columns are shown in Table 3, and load-displacement responses for reference column RT-R and fire-damaged columns FD1-R and FD2-R are shown in Figure 7. The ultimate capacity of column is decreased as fire duration increased, and fire-damaged columns behave more ductile compared with no fire-damaged column due to a decrease in compressive strength and increase in ductility of concrete at elevated temperature.[2] Load-displacement relationships for reference column RT-R and retrofitted columns FD1-CJ and FD2-CJ are presented in Figure 8. The failure mode for CFRP jacketing fire-damaged columns is CFRP ruptured and concrete crushed.

| Label      | Ultimate capacity (kN) | Failure mode                           |
|------------|------------------------|----------------------------------------|
| RT-R       | 4438                   | Concrete crushing                      |
| FD1-R      | 3498                   | Concrete crushing                      |
| FD1-CJ     | 7123.6                 | Concrete crushing and CFRP rupture     |
| FD2-R      | 3031.7                 | Concrete crushing                      |
| FD2-CJ     | 6388.7                 | Concrete crushing and CFRP rupture     |

It was observed that the residual ultimate capacity was decreased with the fire duration increased. RHSC columns lost 21.18% and 31.69% of the axial strength after 1 hour and 2 hours fire exposed, respectively. CFRP jacketing system with high strength mortar could effectively restore and improve the ultimate capacity of RHSC columns. It could restore and improve 60.5% and 43.95% of the ultimate capacity under room temperature for 1 hour and 2 hours fire-damaged RHSC columns, respectively. This retrofitted can also effectively restore the stiffness for fire-damaged columns as shown in Figure 8.
3. Conclusions
Concrete is an effective shield for RHSC column due to its thermal properties. Large temperature gradient could be developed between concrete core and concrete cover, when RHSC column is under fire. Spalling off should be considered for HSC in fire resistance and safety design. Concrete cover gets loose and severely damaged due to the deterioration of concrete at elevated temperature. Fire-damaged RHSC columns could be repaired successfully. Enlarged cross-section with high strength mortar and applied CFRP jackets can effectively restore and improve the ultimate capacity of fire-damaged circular RSHC column. This retrofitted method could also effectively help on stiffness recovery.

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