Research Article

Effects of High Temperatures on the Physical and Mechanical Properties of Carbonated Ordinary Concrete

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Fires are always known for seriously deteriorating concrete in structures, especially for those with certain carbonation due to long-time service. In this paper, 75 prism specimens were prepared and divided into four groups (three carbonated groups and one uncarbonated group). Specimens were tested under different temperatures (20, 300, 400, 500, 600, and 700°C), exposure times (3, 4, and 6 hours), and cooling methods (water and natural cooling). Surface characteristics, weight loss rate, and residual mechanical properties (strength, initial elastic modulus, peak, and ultimate compressive strains) of carbonated concrete specimens after elevated temperatures were investigated and compared with that of the uncarbonated ones. Results show that the weight loss rates of the carbonated concrete specimens are slightly lower than that of the uncarbonated ones and that the cracks are increased with raising of temperatures. Surface colors of carbonated concrete are significantly changed, but they are not sensitive to cooling methods. Surface cracks can be evidently observed on carbonated specimens when temperature reaches 400°C. Residual compressive strength and initial elastic modulus of carbonated concrete after natural cooling are generally larger than those cooled by water. The peak and ultimate compressive strains of both carbonated and uncarbonated concrete specimens increase after heating, but the values of the latter are greater than that of the former. Finally, the constitutive equation to predict the compressive behaviors of carbonated concrete after high temperatures was established and validated by tests.

1. Introduction

In recent years, concrete has been increasingly used, and frequent building fires have caused huge damages to concrete structures [1, 2]. Concrete is generally an inert material, and short-time elevated temperatures slightly affect the performances of it when initially exposed to fire [3, 4]. However, relatively long-time exposure would inevitably change the chemical composition and physical and mechanical properties severely. Thermal-induced cracks usually induce material degeneration and further threaten the safety and serviceability of whole structures [5, 6]. Therefore, extensive experiments have been conducted on the fire responses of concrete after high temperatures [7–12].

Although the elevated temperatures of a natural or accidental fire can reach up to 1000°C [13], almost all the compressive strengths of concrete would be lost beyond 800°C [14]. Accordingly, temperatures in most existing experiments were basically controlled between room temperature (approx. 20°C) and 800°C. Ma et al. [14] and Wu [15] reported that the weight loss of concrete subjected to temperatures below 300°C was mainly caused by the moisture evaporation. Similarly, Mendes et al. [16] confirmed that 400°C was the critical temperature for performances degradation of concrete. Yang et al. [17] performed a series of high temperature tests (400, 500, 550, and 600°C) on cylindrical concrete specimens to predict the residual mechanical strengths by using the ultrasonic pulse velocity. It can be clearly seen that the performances of concrete subjected to 300–700°C are most worthy of study.

Concrete performances after high temperatures for different exposure times have also been extensively evaluated [7, 8]. Since the existing concrete experiments on fire resistances were always conducted within 3 h [9, 18], Shi et al.
[19] examined the influences of long exposure times (6 h, 24 h, 36 h, 48 h, and 72 h) during experiments and reported that the increase of exposure times would cause little degradation of residual compressive strengths. However, the adopted exposure times were considered a failure to match most of the actual fire scenarios [20]. Zhao et al. [21] conducted experiments on performances deterioration of high-strength concrete (HSC, C60) under different heating temperatures and exposure times. Results show that the change of burning loss was not evident when the duration time was within 0.5–2 h, but began to increase when exceeded 3 h. In summary, fire resistances of concrete within duration of 3–6 h, where the degradation is adequately completed, deserve in-depth investigation.

When extinguishing an actual building fire, the adopted cooling methods will greatly affect concrete resistances [22–28], which should also be given full consideration. Luo et al. [23] described the fire resistances of high-performance concrete (HPC) subjected to high temperatures (800 and 1100°C) with both gradual and rapid cooling regimes and concluded that water cooling causes more serious degradation for concrete strengths (thermal shock). Bingöl and Gül [24] observed similar phenomena from experiments conducted on fiber concrete. According to Aslani [26], water cooling produced greater loss than slow cooling for HPC. Ahmed et al. [27] drew similar conclusion that the effects of cooling regimes were similar on normal concrete and rubber fiber concrete.

In recent years, many existing concrete structures around the world were damaged due to concrete carbonation and thus have been greatly concerned [1, 29–34]. Jerga [35] experimentally observed that concrete density, strengths, initial elastic modulus, and shrinkage after carbonation would increase. Cui et al. [36] investigated the relationship between the depth of concrete carbonation and CO2 concentration. However, the performances of carbonated concrete after high temperatures have never been reported. This is one of the most critical problems involving both durability and fire resistances of concrete [20].

In this paper, seventy-five carbonated concrete prism specimens were subjected to different high temperatures, exposure times, and cooling methods. Related physical and mechanical properties were obtained by means of observation, weight weighing, and uniaxial compression tests. The objectives of this paper are to experimentally investigate the performances of carbonated concrete after elevated temperatures and to compare with the uncarbonated concrete.

2. Experiment Procedures

2.1. Materials. All the specimens were prepared from the same batch of readily mixed concrete, and the mix proportion is shown in Table 1. It mainly includes Portland cement (PC) 42.5N [37], 5–10 mm and 10–20 mm naturally fragmentized limestone (coarse aggregate), 0–4 mm river sand (fine aggregate), and fly ash. Class F fly ash was selected according to Chinese standard GB/T1596-2005 [38]. The chemical composition of cement and fly ash is shown in Table 2.

| Table 1: Concrete mixture proportions (kg/m³). |
|---|---|---|---|---|---|
| Cement | Water | Fine aggregate | Coarse aggregate | Fly ash | W/C ratio |
| 280 | 190 | 880 | 880 | 100 | 0.67 |

| Table 2: Chemical composition of cement and fly ash (%). |
|---|---|---|---|---|
| Oxide | SiO₂ | Al₂O₃ | Fe₂O₃ | MgO CaO Na₂O K₂O SO₃ |
| Cement | 20.85 | 5.75 | 3.15 | 2.49 | 61.56 | 0.37 | 1.1 | 3.84 |
| Fly ash | 57.11 | 35.70 | 6.46 | 0.96 | 3.85 | 0.49 | 1.26 | 0.59 |

2.2. Specimens Preparation and Grouping. Seventy-five prism specimens with dimensions of 100 mm × 100 mm × 300 mm were cast in polyvinyl chloride moulds to test the axial compressive stress-strain curves. Detailed grouping of these specimens is shown in Table 3, describing the subsequent carbonation and heating schemes. Three 150 mm cube specimens were also prepared to test the cube compressive strengths at 28 d age without carbonation and high-temperature processing. The average cube compressive strengths at room temperature are tested at 32.2 MPa.

All the specimens were divided into four groups (i.e., Groups 1–4 in Table 3). “C” represents carbonation; “T” is the target temperature (T20 = 20°C, T300 = 300°C, T400 = 400°C, T500 = 500°C, T600 = 600°C, and T700 = 700°C); “E” is the high temperature exposure time (E3 = 3 h, E4 = 4 h and E6 = 6 h); “W” is the water cooling method; and “A” is the air (natural) cooling method. For example, “C-T300E3W” means that the carbonated concrete specimens were exposed to 300°C for 3 h and finally cooled by water. “T300E3W” means that uncarbonated concrete specimens were exposed to 300°C for 3 h and finally cooled by water.

The first two groups of specimens were designed to investigate the impacts of carbonation on the residual compressive strengths of ordinary concrete subjected to high temperatures after water cooling. The third group of samples was used to compare with those in the first group to study the effects of cooling regimes on residual compressive strengths of carbonated concrete. The fourth group of specimens coupled with the first group of specimens was utilized to explain the influences of exposure times on carbonated concrete after high temperatures.

2.3. Accelerated Carbonation Test of Concrete. Accelerated carbonation experiments were conducted according to the Standard for Test Methods of Long-term Performance and Durability of Ordinary Concrete (GB/T50082-2009) [39]. The adopted temperatures, relative humidity, and CO₂ concentration were, respectively, controlled at (20 ± 2)°C, (70 ± 5)%, and (20 ± 3)% [39] (Table 4). The carbonation tests were stopped after 120 days.

The phenolphthalein alcohol reagent method was used to determine the carbonation depth of the test pieces [39] at regular intervals of 3 d, 7 d, 14 d, 28 d, 56 d, 90 d, and 120 d. The surfaces at the cut specimens cross section were brushed, and concentration of 1% phenolphthalein alcohol reagent
was sprayed. The uncarbonated concrete surface appears purple, while the colors of carbonated concrete surfaces remain unchanged, as shown in Figure 1. The average carbonation depth is about 22 mm, and the carbonated occupation ratio is about 68% of the whole area.

2.4. Thermal Treatment and Cooling of Carbonated Concrete.

The working dimensions of the high temperature furnace are $1500 \times 500 \times 500$ mm$^3$. The acceptable temperatures in the furnace range from room temperature to 1000°C with an accuracy of $\pm 1°C$. Concrete specimens after carbonation were placed into the furnace, as shown in Figure 2. The selected target temperature series were 20°C, 300°C, 400°C, 500°C, 600°C, and 700°C. The temperature in the furnace increased rapidly at rate of 4°C/min up to the target temperature, followed by the temperature stabilization phase. After experiencing different exposure times, the specimens were treated with different cooling methods in light of the test scheme.

2.5. Loading and Measurement Process.

The diagrammatic sketch of the used loading machine and displacement devices is shown in Figure 3, illustrating the configuration of a $100 \times 100 \times 300$ mm$^3$ specimen being pressed by load $P$ through the loading Jack. The measurement range of the machine is within 1000 kN.

Specimens were placed on the underlying plate of the machine and then to align the axis of the specimen with that of the plate. Load was evenly applied once the contact between machine upper plate and specimen upper surface was steadily established.

Compressive strengths of these prism specimens were obtained based on the requirement of GB/T 50081-2002 [40]. Loading speed was controlled at 3.0 kN/s. The whole loading process was strictly controlled according to GB/T 50081-2002 [40].

<table>
<thead>
<tr>
<th>Group</th>
<th>Notation</th>
<th>Carbonation for 120 days</th>
<th>Number of specimens</th>
<th>Exposure temperature (°C)</th>
<th>Exposure time (h)</th>
<th>Cooling method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>C-T20</td>
<td>√</td>
<td>3</td>
<td>20</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>C-T300E3W</td>
<td>√</td>
<td>3</td>
<td>300</td>
<td>3</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T400E3W</td>
<td>√</td>
<td>3</td>
<td>400</td>
<td>3</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T500E3W</td>
<td>√</td>
<td>3</td>
<td>500</td>
<td>3</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T600E3W</td>
<td>√</td>
<td>3</td>
<td>600</td>
<td>3</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T700E3W</td>
<td>√</td>
<td>3</td>
<td>700</td>
<td>3</td>
<td>Water</td>
</tr>
<tr>
<td>Group 2</td>
<td>T20</td>
<td>No carbonation</td>
<td>3</td>
<td>20</td>
<td>—</td>
<td>—</td>
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<tr>
<td></td>
<td>T300E3W</td>
<td>No carbonation</td>
<td>3</td>
<td>300</td>
<td>3</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>T400E3W</td>
<td>No carbonation</td>
<td>3</td>
<td>400</td>
<td>3</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>T500E3W</td>
<td>No carbonation</td>
<td>3</td>
<td>500</td>
<td>3</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>T600E3W</td>
<td>No carbonation</td>
<td>3</td>
<td>600</td>
<td>3</td>
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<td>T700E3W</td>
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<td>Group 3</td>
<td>C-T300E3A</td>
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<td>3</td>
<td>300</td>
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<td>Air</td>
</tr>
<tr>
<td></td>
<td>C-T400E3A</td>
<td>√</td>
<td>3</td>
<td>400</td>
<td>3</td>
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<td>C-T500E3A</td>
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<td>3</td>
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<tr>
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<td>700</td>
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<td>Air</td>
</tr>
<tr>
<td>Group 4</td>
<td>C-T400E4W</td>
<td>√</td>
<td>3</td>
<td>400</td>
<td>4</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T400E6W</td>
<td>√</td>
<td>3</td>
<td>400</td>
<td>6</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T500E4W</td>
<td>√</td>
<td>3</td>
<td>500</td>
<td>4</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T500E6W</td>
<td>√</td>
<td>3</td>
<td>500</td>
<td>6</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T600E4W</td>
<td>√</td>
<td>3</td>
<td>600</td>
<td>4</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T600E6W</td>
<td>√</td>
<td>3</td>
<td>600</td>
<td>6</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T700E4W</td>
<td>√</td>
<td>3</td>
<td>700</td>
<td>4</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td>C-T700E6W</td>
<td>√</td>
<td>3</td>
<td>700</td>
<td>6</td>
<td>Water</td>
</tr>
</tbody>
</table>

Note. Degree centigrade (°C) is adopted throughout the paper.

**Table 4: Experimental climate environment conditions.**

<table>
<thead>
<tr>
<th>Group number of specimens</th>
<th>T (°C)</th>
<th>Relative humidity (%)</th>
<th>CO$_2$ concentration (%)</th>
<th>Time (d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1, Groups 3–4</td>
<td>20±2</td>
<td>70±5</td>
<td>20±3</td>
<td>120</td>
</tr>
</tbody>
</table>

![Figure 1: Specimen carbonation depth testing.](image-url)
Initial elastic modulus can be determined by the initial tangent slopes of the obtained stress-strain curves. Strain was calculated by the ratio of the displacement measured by LVDT (Figure 3) to the initial reference gauge distance (200 mm).

3. Test Results and Discussion

3.1. Physical Properties. Physical properties degeneration of carbonated concrete after elevated temperatures can be macroscopically observed by comparing the surfaces changes and weight loss rate of the carbonated concrete specimens before and after heat handling.

3.1.1. Surface Characteristics. Surface characteristics of carbonated concrete specimens after elevated temperatures (water or natural cooling) are illustrated in Figure 4. The appearance morphology of carbonated concrete changes gradually with the increase of temperatures when the water cooling method is used. It can be clearly seen that the surface cracks of carbonated concrete after both water and natural cooling become evident at 400°C, which is similar to that observed in experiments from [41]. As for uncarbonated concrete, no visible surface cracks could be observed after high temperatures from 100°C to 600°C, a similar phenomenon as described in [9], implying that the carbonation process reduces the resistance of ordinary concrete to surface cracks at high temperatures. In addition, more serious surface cracks occur in specimens after water cooling than natural cooling, which is produced by the residual stresses between outer and inner core of the concrete induced by fast cooling [27]. Microcracks subsequently occur in the matrix, aggregate, and the interface of them [26, 28]. Similar phenomena can be observed from [24]. Surface colors of carbonated concrete specimens after high temperatures can be evidently changed with the increase of temperatures. The surface colors of carbonated concrete specimens subjected to different temperatures after water cooling and natural cooling are almost the same.

3.1.2. Weight Loss Rate. Mass loss is linked to loss of moisture present in various forms in concrete, namely, free, capillary, adsorbed, and chemically combined water [42]. During the heating process of the carbonated concrete specimens, the residual-free water in the interior has evaporated when temperatures are controlled below 300°C [4]. The bound water in the cement gel rises as the temperatures continuously increase, leading to the decrease of the quality of the concrete specimens and the increase of the burning loss rate.

The weight loss rate of specimens, \( \delta_w \), is determined through the following equation:

\[
\delta_w = \frac{m_s - m_h}{m_s} \times 100\%.
\]  

where \( m_h \) represents the weight of specimens after carbonation and then exposed to high temperatures and \( m_s \) the weight of specimens after carbonation without high temperature duration. Figure 5 shows the weight loss rate with different exposure temperatures. All samples experienced a slight monotonous decrease in weight with the increase of temperatures [43]. When the temperature does not exceed 400°C, the weight loss rate of carbonated concrete (Group 1 in Table 1, black color in Figure 5) increases with the same trend as uncarbonated concrete (Group 2 in Table 1, blue color in Figure 5) due to the removal of water [41]. When the temperature is higher than 400°C, the weight loss rate of the former is greater than that of the latter. This is because that the uncarbonated Ca(OH)₂ begins to dehydrate to CaO [2], causing the microcracks in the interface between the cement gel and the coarse aggregate to expand. It can also be seen that the weight loss rate of carbonated concrete with water cooling method (Group 3 in Table 1, red color in Figure 5) is higher than that with water cooling method (Group 1 in Table 1, black color in Figure 5). However, the difference decreases gradually with the increase of temperatures. The reason lies in that water-cooled concrete may absorb more water than natural-cooled concrete during cooling process, resulting in the less quality loss of water-cooled concrete than natural-cooled concrete when subsequently exposed to relatively lower thermal process (below 300°C). However, as the increase of temperatures and exposure times, the absorbed water of water-cooled concrete was released, and the weight loss of both water- and natural-cooled concrete tended to be consistent gradually. Eventually
the weight loss rates of cooled concrete using the two kinds of methods keep decreasing at the same pace.

Figure 6 shows the weight loss rate of carbonated concrete exposed to different temperatures with different exposure times (3 h, 4 h, and 6 h). It can be clearly seen that the change span of the weight loss rate of the carbonated concrete is small, indicating that the effect of the high temperature duration on the weight loss rate of carbonated concrete basically stabilizes after the constant temperature time exceeds 3 h.

3.2. Mechanical Properties

3.2.1. Failure Modes of Carbonated Concrete after High Temperatures. The integrity of most specimens during the loading is good, but the forms of fracture and destruction of the specimens are different with the increase of load, as
shown in Figure 7. Longitudinal splitting failure always occurs in these specimens subjected to temperatures. When the stress does not reach the peak stress, continuous load increase does not inspire evident cracks on the specimen surface. With the increase of load, fine cracks gradually appear and continue to develop. When the stress exceeds the peak stress, the cracks concentrate in a short period, and subsequently a sharp bearing capacity decline occurs.

3.2.2. Residual Compressive Strength. The compressive strengths of the prism concrete specimens, \( f_c(T) \), were calculated based on the following equation:

\[
f_c(T) = \alpha \frac{F}{A}
\]

where \( F \) and \( A \) are, respectively, the failure load and bearing area of specimens and \( \alpha \) is the conversion strength coefficient from the nonstandard specimens \((100 \times 100 \times 300 \text{ mm}^3)\) to standard ones \((150 \times 150 \times 300 \text{ mm}^3)\) according to GB/T 50081–2002 [40].

Figure 8 illustrates a comparison between the available residual compressive data of ordinary concrete at elevated temperatures based on the fruits of these previous studies [8, 9, 18–20] and that of this study (Group 2 in Table 3). It can be clearly noted that the overall downward trend is consistent, although the relative compressive strengths from the literature are generally higher than that of Group 2 (Table 3) in this study.

Figure 9 shows the effects of cooling methods on relative compressive strengths. It can be clearly observed from this figure that the residual compressive strengths of Group 1 decrease with the raising of the target temperatures, especially when the critical temperature 500°C is exceeded. The residual compressive strengths of the carbonated specimens (Group 1) are slightly higher than that of the uncarbonated ones (group 2) when temperatures increase from 300°C to 700°C, which is caused by the CaCO₃ produced by concrete carbonation filling in the internal micropore of concrete. In particular, the residual compressive strengths of carbonated specimens are only approx. 15% once the temperature reaches 700°C, when the bearing capacity is almost lost.

Before 400°C, residual compressive strengths of group 1 after carbonation are almost the same as that of group 3. When the target temperature is over 400°C, rapid cooling increases the width of the microcracks. The residual compressive strengths cooled by water are lower than that cooled naturally, and the higher the temperatures, the greater the differences between these two cases, which indicates that air cooling can more effectively mitigate the reduction degree of the compressive strengths.

Using the linear fitting method, the residual compressive strengths calculation model of carbonated specimens under water cooling is established as follows:

\[
f_c(T) = \begin{cases} 
1.01074 \times 10^{-4}T + 8.96687 \times 10^{-4}, & 20°C \leq T \leq 500°C, \\
1.58433 \times 10^{-4}T + 0.00206T, & 500°C \leq T \leq 700°C.
\end{cases}
\]

Figure 10 shows the ratio of residual compressive strengths to compressive strengths at room temperature \( f_c(T)/f_c \) of carbonated concrete specimens. Increasing exposure times while keeping constant temperatures would not greatly change the residual compressive strengths of the carbonated concrete, which indicates that the internal reaction of the concrete has basically completed after the exposure time exceeds 3 h.

3.2.3. Relative Initial Elastic Modulus of Concrete. The initial elastic modulus of concrete is related to the density of the internal structure. The denser the internal structure, the greater the initial elastic modulus of concrete. In the heating process, microcracks between the interface of cement matrix and the coarse/fine aggregate continuously emerge and expand. The compactness declines inside the concrete structure, resulting in the decrease of the initial elastic modulus. Figure 11 shows the ratio change of the initial elastic modulus \( E_c(T) \) at \( T \) temperature to initial elastic modulus at room temperature \( E_c \). Both \( E_c(T) \) and \( E_c \) are defined as initial elastic modulus. It can be seen from the figure that the relative initial elastic modulus of Group 1 concrete is slightly higher than that of Group 2 in all temperature ranges, and both of them decrease with the increase of temperatures.

Using polynomial nonlinear fitting method, the calculation model of relative initial elastic modulus of carbonated concrete after water cooling is established as

\[
\frac{E_c(T)}{E_c} = 1.0439 - 2.37 \times 10^{-3}T + 1.425 \times 10^{-6}T^2.
\]

3.2.4. Peak Compressive Strain and Ultimate Compressive Strain. Figure 12 shows the peak strain and ultimate strain ratio of concrete after high temperatures to that at room temperature, respectively. As can be seen from the figure, the peak strains and ultimate strains of uncarbonated concrete at each temperature are larger than that of carbonated concrete, indicating that the ductility of carbonated concrete after water cooling is lower than that of uncarbonated concrete.

Using polynomial nonlinear fitting method, the calculation model of peak strain and ultimate strain of concrete after water cooling is established as

\[
\frac{\varepsilon_p(T)}{\varepsilon_0} = 1 - 8.9356 \times 10^{-5}T + 1.3095 \times 10^{-5}T^2,
\]

\[
\frac{\varepsilon_u(T)}{\varepsilon_0} = 1 + 9.2214 \times 10^{-4}T + 8.8025 \times 10^{-6}T^2.
\]

Figure 12(b) shows that the peak strain and ultimate strain of concrete with carbonization and uncarbonation almost coincide and increase with the same speed. Therefore, the same relation can be used to express the relationship between peak strains and ultimate strains and temperatures after concrete is cooled.
Figure 7: Failure forms of specimens after exposure to different high temperatures for 3 hours.

Figure 8: Effects of exposure temperatures on relative compressive strength.

Figure 9: Effects of cooling methods on relative compressive strength.

Figure 10: Variation of relative compressive strength with exposure time (water cooling).

Figure 11: Variation of relative initial elastic modulus with exposure temperature (water cooling).
4. Constitutive Model of Carbonated Concrete after High Temperature

Finite element simulation has become an increasingly used method to evaluate the healthy condition of structures. For accurate analysis of fire damaged carbonated concrete structures, the mechanical properties of fire damaged carbonated concrete materials are of great significance, of which the most important one is the stress-strain relationship.

Since the exposure time slightly affects the behaviors of carbonated concrete at elevated temperatures, reasonable constitutive model only needs to involve the temperatures. The compressive constitutive model of carbonated concrete after high temperatures with different times is proposed as follows:

\[ \sigma = \eta(T) \cdot \left[ K_T(T) \cdot E_c \right] \cdot \exp \left[ -\frac{\varepsilon^{1.75}}{\alpha(T)} \right], \]  

(6)

where \( E_c = 30134 \text{ N} \cdot \text{mm}^{-2} \); \( \eta(T) \) and \( \alpha(T) \) are, respectively, the defined correction factor and scale parameter; and \( K_T(T) \) is the temperature softening parameter and can be expressed as

\[ K_T = \frac{E_c(T)}{E_c}. \]  

(7)

The parameters in the proposed constitutive model are determined using least square method according to the test results and given in Table 5 and equation (8):

\[ \eta(T) = 1.131 - 4.906 \times 10^{-4}T - 8.739 \times 10^{-7}T^2, \]

\[ \alpha(T) = 6.112 \times 10^{-5} - 6.316 \times 10^{-5}T + 2.719 \times 10^{-9}T^2. \]  

(8)

Figure 13 illustrates the comparison between the established constitutive model and the measured stress-strain data. It can be clearly observed that the proposed
model can be used for empirical prediction of the behaviors of carbonated concrete after high temperatures process.

5. Conclusions

Effects of different heating temperatures, exposure times, and cooling methods on the performances of the prism carbonated concrete specimens were examined and compared with that of uncarbonated ones. The following conclusions were drawn:

(1) Surface cracks of carbonated concrete become evident at 400°C, which are similar with that of the uncarbonated concrete, as observed in [41]. The surface colors of carbonated concrete after both water and natural cooling are indistinguishable.

(2) Weight loss rate of the carbonated concrete specimens after natural cooling is higher than that after water cooling. Different declining degrees between all of them also occur with the raising of exposure temperatures, especially when temperatures reach 700°C.

(3) Heating time over 3h has little effect on the weight loss rate, residual compressive strengths, and initial elastic modulus of carbonated concrete. The same conclusions were also drawn from the uncarbonated concrete [9].

(4) Residual compressive strengths and initial elastic modulus of carbonated concrete decrease with the gradual raising of exposure temperatures and overall perform better than the uncarbonated concrete. The abovementioned properties of carbonated concrete after natural cooling are generally larger than that under water cooling, an opposite phenomenon compared with that of uncarbonated experiments of [23–27].

(5) Peak strains and ultimate strains of both carbonated and uncarbonated concrete increase with elevated temperatures, but the value of the former keeps lower than that of the latter, implying that the carbonation process will weaken the ultimate deformation capability.

(6) A constitutive model was empirically proposed to well predict the performances of carbonated concrete subjected to high temperatures.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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