Strength and stiffness of concrete under heating and cooling treatments

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ABSTRACT: Understanding the properties of concrete under high temperature is essential to enhance the fire resistance of reinforced concrete structures (RCS) and to provide accurate information for fire design of RCS. During and after a fire, different parts of a concrete structure experience different heating and cooling scenarios. Hence the remaining stiffness and strength properties in the concrete structure need to be assessed before a decision can be made as to whether the structure can be repaired or must be rebuilt. The purpose of this study is to investigate the strength and stiffness performance of concrete subjected to various temperature scenarios with the aid of residual compressive strength testing. In addition, the temperature distribution in the concrete cylinders, color changes, and cracks are investigated.

1 INTRODUCTION

Under a rapid heating condition, Portland cement concrete experiences a large volume change resulting from thermal dilatation of coarse and fine aggregates as well as from shrinkage of the cement paste. When the heating rate is high, spalling damage may occur in concrete due to high thermal stresses and pore pressure build-up. Therefore, it is very important to assess the residual mechanical properties after the concrete has been subjected to high temperature at different heating and cooling rates. The results will provide essential information to the concrete industry for improving the fire resistance of concrete.

Extensive experimental studies on this important topic were performed in the past. The important experimental parameters included maximum temperature, heating rate, types of aggregates used, various binding materials, and mechanical loads under high temperature conditions. The experimental studies have mainly concentrated on the strength of concrete. Poon et al. conducted experimental studies on the strength of normal and high strength concretes. However their studies did not consider the effects of heating and cooling rates which are very important factors on the degradation of concrete due to fire. From the strength point of view, the reduction of concrete strength is affected strongly by the heating rate as well as the cooling rate.

The purpose of this study is to investigate the residual compressive strength and stiffness of concrete subjected to various cooling regimes. The test variables are the maximum temperatures and cooling regimes under the constant heating rate of 2 °C/min and holding time of 4 hrs at the maximum temperatures. Additionally, the temperature distribution in the concrete cylinder, color changes in concrete, and crack patterns are also investigated.

2 SPECIMEN PREPARATION, HEATING EQUIPMENT, AND TEST VARIABLES

2.1 *Mix design and specimen preparation*

In this experimental study granite was used as the coarse and fine aggregate in all concrete specimens. In the initial mix design the water/cement (w/c) ratio was 0.50 (see Table 1).

Table 1. Moderate strength mix design of normal weight concretes (by weight)

Moderate strength mix design – kN/m^3				Strength (Mpa)
Cement	Water	Fine agg.	Coarse agg.	
3.49	1.75	8.32	10.12	30

However the adjusted w/c ratio was 0.71, which is very high (see Table 1). This was due to the dry aggregates and contents of very dry, fine aggregates in crushed granite sand. Different water/cement ratios were examined for workability and a w/c ratio of 0.71 was the lowest value with which the proper workability with slump value 38.10 mm could be obtained. The moisture content of the aggregates was measured using the test method B in ASTM D

2216. The absorption capacity of aggregates was 0.78 %. By subtracting the water absorption capacity of the aggregates from the initial water content a final w/c ratio of 0.67 by weight is obtained. The maximum size of coarse aggregate was 19.05 mm.

Concrete cylindrical specimens with dimensions of 101.6×203.2 mm were made for the experimental study. The size of the specimen was determined by the internal size of the furnace used in the study, which is 198.12 mm×289.56 mm×167.64 mm. The specimens were cured under the standard condition (Temperature 23 ˚C and RH 93 %) for 8 weeks in a fog room.

2.2 *Heating equipment and test variables*

An electrically heated furnace (Model RHF 15/8 of CARBOLITE) designed for maximum temperature up to 1600 ˚C was used. The temperature history inside the furnace was measured and recorded using type K-thermocouples. Model OM-CP-OCTTEMP produced by OMEGA was the data logger for the thermocouples.

The effect of three cooling conditions was examined in the study, and they were called slow cooling, natural cooling, and water cooling. Four maximum temperatures (target temperatures) were tested: 200, 400, 600, and 800 ˚C. The heating rate was 2 ˚C/min, and the holding time at the maximum temperature was 4 hrs.

In the case of natural cooling, the specimen was left in the furnace and the temperature change was recorded over time. For slow cooling, the cooling rate was controlled as shown in Fig. 1, slower than natural cooling. For water cooling, the specimen was taken out of the furnace and put in a tank of water with initial temperature of 20 ˚C. The furnace was heated using two groups of three heating elements on either side of the specimen. In order to provide a more uniform thermal condition inside the concrete specimen, the concrete specimen was placed inside a hollow tube made of mullite with a maximum operational temperature of 1700 ˚C.

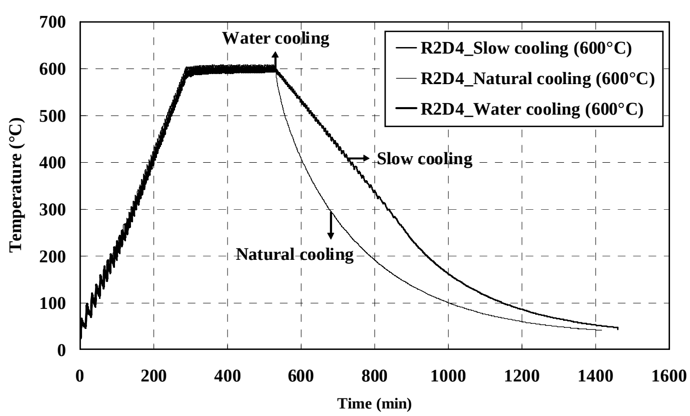


Figure 1. Temperature histories subjected to different cooling regimes

Figure 1 shows the temperature histories using a heating rate of 2 ˚C/min, a hold time of 4 hrs at the maximum temperature of 600 ˚C, and three different cooling conditions. The ambient temperature history was recorded between the mullite tube and the specimen with the help of a K-thermocouple.

3 TEMPERATURE DISTRUBUTION IN THE CONCRETE

Figure 2 shows the concrete cylinder used to investigate the temperature distribution at elevated temperature.

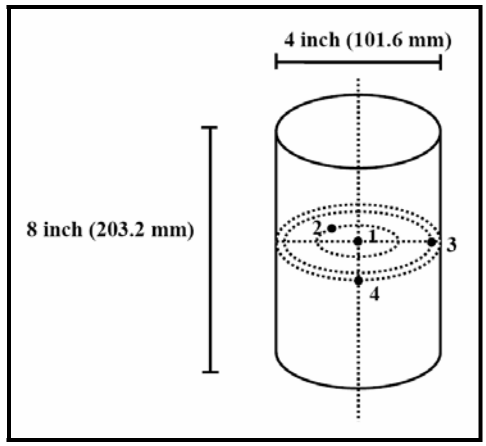


Figure 2. Locations of thermocouples and specimen geometry

One thermocouple was used to measure ambient temperature between the specimen and mullite hollow tube, and four additional K thermocouples were installed inside and on the surface of the specimen to measure the temperature distribution in the concrete cylinder. Figure 2 shows the specimen geometry and the locations of the thermocouples. The heating rate was 1 ˚C/min, the maximum temperature was 900˚C with a hold time of 2 hrs, and the cooling method was natural cooling.

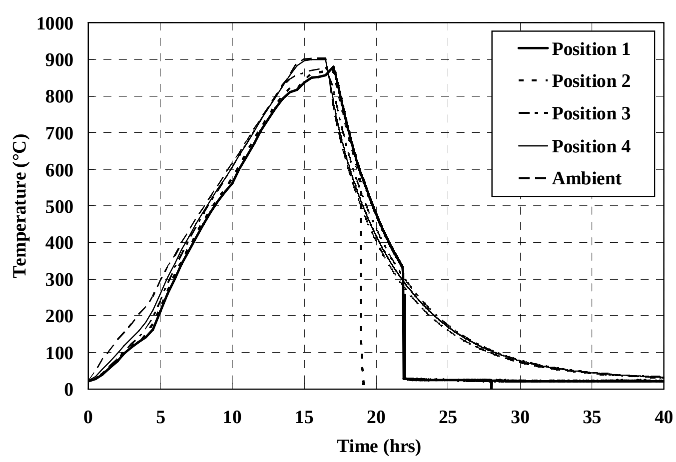


Figure 3. Transient temperature at each position in the concrete cylinder

Figure 3 shows temperature histories over time measured at each thermocouple location. One can see that in the heating phase the surface temperature is higher than the internal temperature and in the cooling phase the surface temperature is lower than the internal temperatures, which are expected experimental results. The two vertical drops in the cooling phase in Figure 3, near 540 ˚C at the center and 330 ˚C at mid-radius, are due to the damage of the two thermocouples in position 1 and position 2.

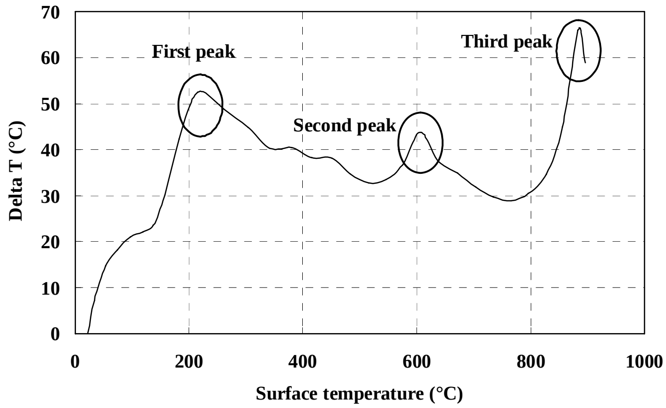
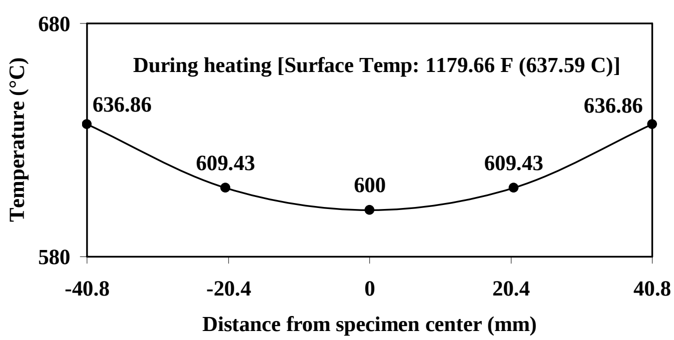


Figure 4. ΔT vs. Surface temperature (during heating)

Figure 4 shows the temperature difference between the surface and center of the specimen at different temperature ranges. One can see that the temperature difference varies during the entire testing period, which means that a steady state condition in the concrete was never reached. This is mainly due to the phase transformations taking place at different temperatures in the concrete. The three peaks shown in the figure are related to the micro-structural changes due to complex physicochemical transformations in the concrete under different high temperatures. The peaks shown in Fig. 4 may be explained by the test results of DTA (Differential Thermal Analysis) conducted by Lankard. The first peak occurs at about 200 ˚C due to evaporation of free water and dehydration of calcium silicate hydrate (C-S-H). The second peak appears between 550 ˚C and 650 ˚C and is related to the decomposition of calcium hydroxide (CH) and calcium silicate hydrate (C-S-H). The third peak occurring at 850˚C is likely due to the decomposition of calcium carbonate ($CaCO_3$) which was observed in the DTA experiments by Lankard.



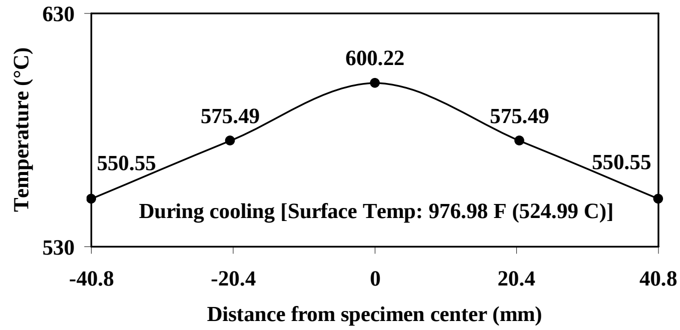


Figure 5. Temperature profiles during heating and cooling

Figure 5 shows the temperature profiles measured at positions 1, 2, and 3, when the temperature at the center of specimen reaches 600 ˚C. The surface temperature corresponding to each temperature profile is indicated in the figure. One can clearly see that while undergoing natural cooling, the temperature difference between the center and the surface of the concrete is larger than that during the heating period. In general, a larger temperature difference represents a higher temperature gradient, which leads to a higher damage. Therefore, this test data means that the damage of the concrete is strongly affected by cooling method.

4 RESIDUAL COMPRESSION TEST

Axial strains were measured using two extensometers (MTS model 632.94E-20). The average of the two strain readings was taken as the axial strain. Displacement control with a rate of 0.0001 inch/sec was used for the compression test.

Figure 6, 7, and 8 plot stress versus strain for the specimens that were subjected to slow, natural, and water cooling, respectively. As can be seen from the figures both strength and stiffness properties of concrete decrease significantly with increasing maximum temperature.

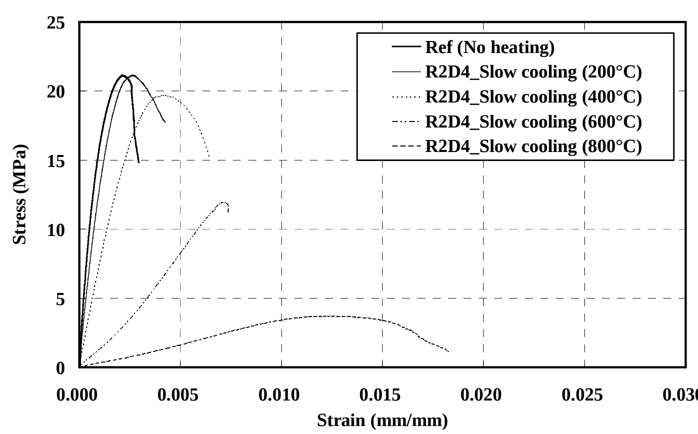


Figure 6. Axial stress-strain response (Slow cooling)

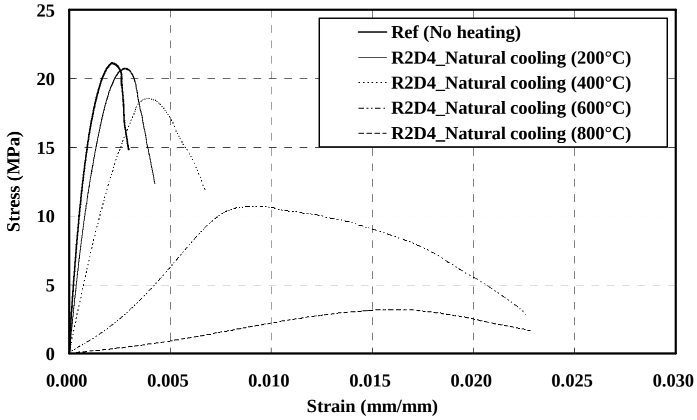


Figure 7. Axial stress-strain response (Natural cooling)

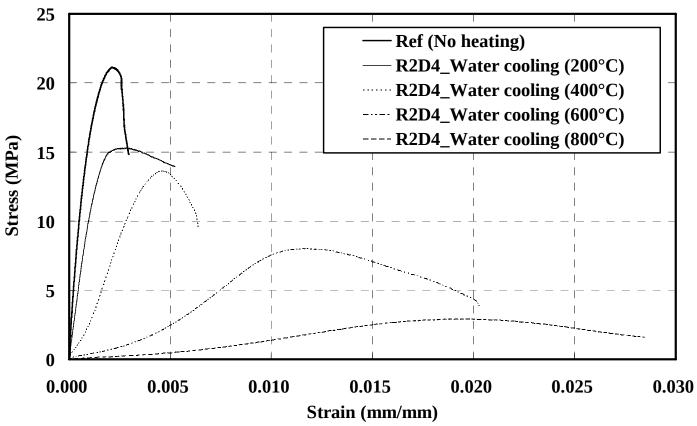


Figure 8.Axial stress-strain response (Water cooling)

The elastic response behavior is fairly nonlinear for those specimens subjected to slow or natural cooling and maximum temperatures of 600 ˚C and beyond. The initial slope of the axial stress-strain response is lower than the slope in the middle part, indicating that the concrete is hardened or rather reconsolidating during mechanical loading. For the case of water cooling, the same trend is shown in the stress-strain curves for the range of maximum temperatures of 400 ˚C and beyond. This hardening behavior may be caused by the closure of thermally induced cracks formed during high temperature heating and cooling treatments. In the initial stage of loading the cracks are open as the load gradually increases, resulting in a lower slope at the beginning. After the cracks are closed, the slopes of the curves gradually increase and follow the general trend of a compression strength test.

Table 4 summarizes the ultimate strength values from the residual compression tests. Figure 9 depicts the residual strength normalized by the reference strength (25 ˚C) for the three cooling methods as a function of the target temperature. The residual strength rapidly drops beyond 400 ˚C, whereby the specimens subjected to water cooling decrease more rapidly than the specimens subjected to other cooling methods. The strength of specimens exposed to a target temperature of 600 ˚C is less than 57 % of the reference strength at room temperature. For 800 ˚C, the strength is less than 18 % of the reference strength.

Table 4. Ultimate strength results of residual compression test

Temp. (˚C)	Ultimate strength (Mpa)		
	Slow	Natural	Water
25	21.16	21.16	21.16
200	21.15	20.77	15.32
400	19.69	18.59	13.64
600	11.96	10.72	8.02
800	3.71	3.19	2.93

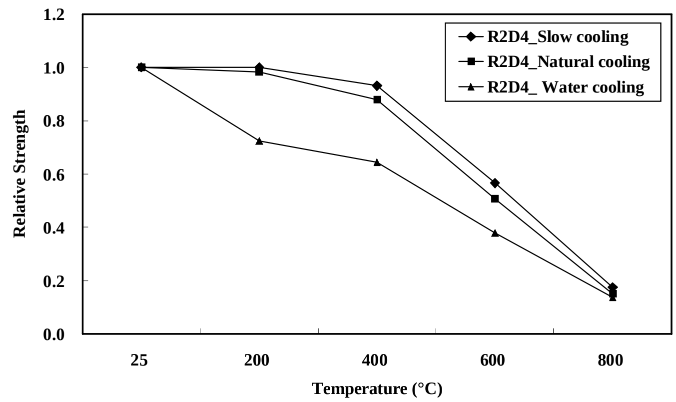


Figure 9. Relative residual strength vs. maximum temperature

Table 5 summarizes the test results for the initial tangent modulus from the residual compression test. Figure 10 shows the values of the initial tangent modulus with different temperatures, normalized by the reference specimen tested at room temperature.

Table 5. Initial tangent modulus from residual compression test

Temp. (˚C)	Initial tangent modulus (Mpa)		
	Slow	Natural	Water
25	2.48E+04	2.48E+04	2.48E+04
200	1.61E+04	1.44E+04	1.12E+04
400	7.86E+03	7.17E+03	2.43E+03
600	1.32E+03	9.51E+02	3.97E+02
800	3.03E+02	1.68E+02	8.96E+01

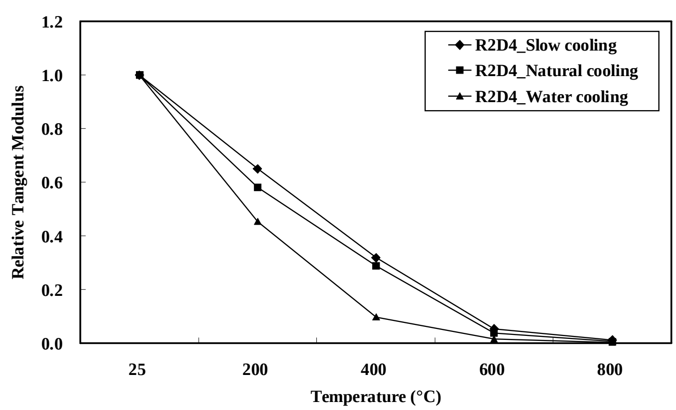


Figure 10. Relative initial tangent modulus vs. maximum temperature

The basic trend of the variation of initial tangent modulus shown in Figure 10 is similar to that of the residual strength shown in Figure 9. However, the initial tangent modulus of concrete is more sensitive to elevated temperature than the compression strength. As shown in Figure 10, the drop in stiffness at lower temperatures is larger than the drop in strength shown in Figure 9. In contrast, the drop in strength in the higher temperature range is larger than the drop in stiffness shown in Figure 10. Specifically, the initial tangent moduli of the specimens exposed to a maximum temperature of 600 ˚C are less than 5.3 % of the initial reference tangent modulus. For 800 ˚C, the moduli are less than 1.3 % of the reference values. In addition to the deterioration of stiffness and strength due to maximum temperature, we also observe that both heating as well as cooling rates have significant effects on the mechanical properties. In terms of cooling rate, faster cooling contributes to a more severe decrease of both the strength and stiffness properties of concrete.

5 COLOR CHANGES AND CRACKS

Color changes and crack patterns in concrete are important features for assessing the fire damage of the concrete. They can be used to evaluate the degree of damage in concrete and to estimate the maximum temperature experienced by the concrete specimen.

In the concrete exposed to high temperature, the exposed maximum temperature can be predicted from some changes in color. The chemical composition of granite used as the aggregates in the study is composed mostly of silica (SiO_2) and alumina (Al_2O_3). Pink or red spots were observed on every specimen exposed to a maximum temperature of 400 ˚C and above. The colors of the specimens subjected to slow and natural cooling were generally pink or red, while the colors of the specimens subjected to water cooling were dark pink or dark red due to water absorption during cooling in water.

The distributed cracks were visually observed on every surface of the specimens exposed to a maximum temperature of 600 ˚C and above. Particularly, it is likely that the distributed cracks between the aggregate and cement paste matrix are deeply related to the differences in the thermal deformations between the two. The crack widths of the specimens exposed to a maximum temperature of 800 ˚C were visibly wider than those of the specimens exposed to a maximum temperature of 600 ˚C. Also, in the specimens exposed to maximum temperature 800 ˚C, the crack widths of the specimens heated quickly (15 ˚C/min) were significantly larger than those of the specimens heated slowly (2 ˚C/min).

6 CONCLUSIONS

1. A systematic experimental study was conducted for concrete specimens under different maximum temperatures and cooling rates. Residual strength and stiffness, color changes, and crack patterns were observed and reported. Also temperature profiles inside the specimens were measured and recorded.
2. Temperature differences between the center and surface of concrete cylinders at steady state did exhibit distinguishing features. There are three distinct peaks in the temperature difference within the range of room temperature and 850 ˚C. The three peaks are related to the micro-structural changes due to complex physicochemical transformations in the concrete under high temperature. The first peak at about 200 ˚C is associated with the dehydration of calcium silicate hydrate (C-S-H) and the evaporation of free water. The second peak at 550 ˚C is related to the decomposition of calcium hydroxide (CH) and calcium silicate hydrate (C-S-H). The third peak at 850 ˚C may be due to the decomposition of calcium carbonate.
3. Complete stress-strain curves were obtained for evaluating residual mechanical properties of the concrete. Strength and stiffness properties decrease significantly as the maximum temperature and the cooling rate increase. The present test data demonstrate that the cooling rate is an important parameter for the degradation of mechanical properties of concrete subjected to high temperatures.
4. The crack width increases with increasing temperature and heating rate which means both contribute and enlarge the damage of concrete.

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