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Review

Influence of Natural Fire Development on Concrete Compressive Strength

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Abstract: With increasing acceptance of performance-based design principles in the field of fire safety, it is imperative to accurately define the behaviour of materials during fire exposure. Real-world fire events, otherwise referred to as natural fires, are defined by four characteristics: heating rate, maximum temperature, exposure duration, and cooling rate. Each of these four characteristics influences concrete's behaviour in a different manner. In this paper, the available experimental work for concrete, tested at elevated temperatures, is examined to identify the influence of the four natural fire characteristics on concrete compressive strength. This review focuses on normal strength concrete tests only, omitting parameters such as unique additives and confinement. The intent is to provide a fundamental understanding of normal strength concrete. The findings show that maximum temperature and cooling rates have a significant influence on concrete strength. Exposure duration has a moderate impact, particularly at shorter durations. Variable rates of heating have minimal influence on strength. Detailed conclusions are provided along with review limitations, practical considerations for designers, and future research needs.

Keywords: natural fire; concrete strength; exposure duration; maximum temperature; heating rate; cooling rate



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1. Introduction

In contrast with timber and steel construction, one of the primary advantages of using concrete as a building material is that it can withstand fire events without burning, melting, or needing additional protective materials. Concrete, however, is not completely unaffected by fire exposure. Studies on normal strength concrete (NSC) have shown that an exposure temperature of 600 °C can reduce concrete compressive strength by up to 55% [1].

There are a number of material properties that are known to affect concrete strength at elevated temperatures, such as ambient strength, aggregate type, water-cement ratio, additives, and prestress level. The influence of these properties is well investigated in the existing experimental work and detailed in numerous textbooks and literature-review publications [2,3]. Concrete strength is also affected by fire characteristics, such as: rate of heating, maximum temperature level, exposure duration, and rate of cooling [4]. The influence of these four fire characteristics is less thoroughly addressed in the existing literature.

It is imperative for designers to understand the behaviour of fires and its influence on concrete compressive strength. Existing performance-based models have shown that understanding these four fire characteristics is a necessary step to accurately modeling reinforced concrete beam and column behaviour [5]. By implementing the findings of this review into existing models, designers can produce performance-based solutions with increased confidence in safety, reliability, and efficiency.

2. Natural Fire Definition

Fire events are typically represented by temperature–time relationships, as shown in Figure 1. The term natural fire is used to define a fire event as it would occur in the real world. No two natural fires will ever be identical, as these fires are influenced by a wide range of compartment and environmental properties. Three examples of potential fire profiles are shown in Figure 1. Fire events can have high temperature over short duration, low temperature over long duration, or anywhere in between.

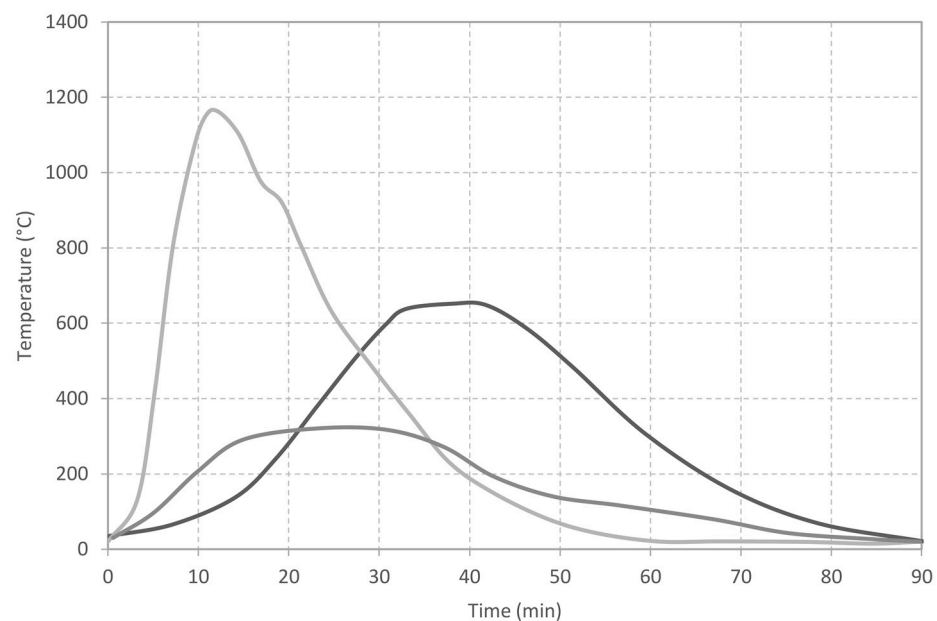


Figure 1. Examples of natural fire temperature–time curves.

To define a natural fire, four fire characteristics can be calculated: heating rate, maximum temperature, overall exposure duration, and cooling rate [4]. During the growth of a fire, variable rates of heating can occur, ranging from slow heating to almost instantaneous flashover. The rate of heating is greatly dependent on available oxygen and the presence of highly combustible materials. At the peak of a fire event, the value of the maximum temperature as well as its exposure duration vary based on reliability of fuel and oxygen supply. Once a fire begins to decay, variable rates of cooling can be present, ranging from slow air cooling in a smoldering compartment to rapid water cooling achieved by firefighting efforts. Each of these four fire characteristics plays a notable and different role in the deterioration of concrete strength. It is the influence of these characteristics on concrete strength that is evaluated in this paper.

3. Available Experimental Work

The available experimental work features a wide range of testing parameters. To narrow the scope of this review, several concrete and testing parameters are controlled. Only tests with the following attributes have been reviewed in this paper: (a) unstressed tests, (b) unconfined tests, (c) unsealed tests, (d) ordinary Portland cement (no additives such as fly ash, silica, fibers, etc.), and (e) NSC with ambient strength less than or equal to 50 MPa. The intent of the control parameters is to focus the evaluation on basic NSC. Doing so highlights the influence of fire characteristics on behaviour and allows future researchers to identify when newly introduced parameters present unusual responses. Additionally, the majority of existing work is based on NSC, allowing for a wide range of sample data points.

In addition to the controlled parameters, there are other parameters that are known to affect compressive strength during fire exposure. These parameters include water–cement

ratio, aggregate-cementitious material ratio, aggregate type, size and content, geometric dimensions, and testing procedure [3]. Because there is so much variation in the existing experimental work, it is difficult to control all these parameters. Furthermore, the variation of these parameters is acceptable within the definition of NSC. To provide meaningful review, specific sections of this paper control the various parameters when possible, and when not possible, a selection of similar tests are averaged and presented for evaluation.

Table 1 shows a summary of the experimental tests investigated during this review. Full details are available in the referenced work. The required fire characteristics of each test, shown in Figure 2, are identified in the table. The concrete ambient strength (" $f'_{c,20}$ ") and reported "aggregate" type are also recorded for additional context. "Testing time" refers to when the compressive strength of the sample was taken. A "residual" testing time indicates that testing occurred after the specimen cooled back to ambient temperature. A "hot" testing time indicates testing occurred while the sample was still at the maximum applied temperature.

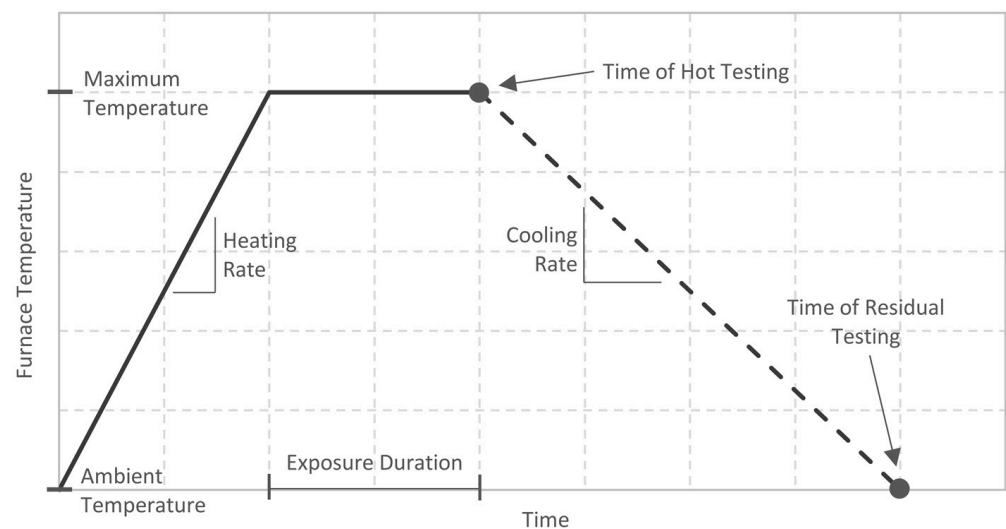


Figure 2. Furnace heating profile during specimen testing.

For experimental work with a variable "heating rate", the average rate is provided in the table. A heating rate of "instant" indicates that the specimen was placed in a preheated furnace. A rate of "standard" indicates that the standard fire curve was applied for the heating profile. The term "measured" is used for tests where the heating rate was controlled based on measuring the internal temperature of the specimen and maintaining some maximum difference from the furnace temperature. Maximum temperature ("max temp") is recorded as the maximum temperature of the furnace. Exposure "duration" is recorded in hours from the time when heating ends to the time when hot testing or residual cooling begins. An exposure duration of "uniform" indicates the specimen's internal temperature was measured and that heat was applied for a continuous duration until the specimen's internal temperatures uniformly reached the furnace temperature. "Cooling rate" is stated as either "slow" or "rapid". Comprehensive definitions of the two cooling rates are provided in Section 4.4. It should be noted that similar to maximum temperature, the heating and cooling rates refer to the temperature change in the furnace, not the specimen itself. Although the furnace temperature is not necessarily an ideal way to represent these values, it is easier to record and is widely reported in the literature as such.

Table 1. List of Evaluated Experimental Work with Test Parameters.

Label	Ref.	$f'_{c,20}$ (MPa)	Agg Type	Duration (hr)	Testing Time	Heating Rate (°C/min)	Cooling Rate
T-6A	[6]	27	Siliceous	uniform	residual	2.4	slow
T-6B	[6]	27	Siliceous	uniform	residual	2.4	rapid
T-7A	[7]	27	Calcareous	uniform	hot	measured	—
T-7B	[7]	27	Calcareous	uniform	residual	measured	slow
T-7C	[7]	27	Siliceous	uniform	hot	measured	—
T-7D	[7]	27	Siliceous	uniform	residual	measured	slow
T-8	[8]	50	Quartzite	2.00	hot	5.0	—
T-9A	[9]	35	Calcareous	3.00	residual	16.0	slow
T-9B	[9]	35	Calcareous	3.00	residual	16.0	rapid
T-10	[10]	45	Siliceous	15.00	residual	1.0	rapid
T-11	[11]	47	Limestone	varied	residual	0.3	slow
T-12	[12]	31	Limestone	0.17	hot	7.5	—
T-13A	[13]	27	Siliceous	2.00	residual	2.8	slow
T-13B	[13]	40	Siliceous	2.00	residual	2.8	slow
T-14	[14]	33	Siliceous	2.00	hot	2.0	—
T-15	[15]	34	Granite	1.00	hot	2.0	—
T-16A	[16]	21	Siliceous	2.00	hot	1.0	—
T-16B	[16]	42	Siliceous	2.00	hot	1.0	—
T-17A	[17]	25	Sandstone	1.00	residual	1.5	slow
T-17B	[17]	23	Gravel	1.50	hot	7.5	—
T-18	[18]	50	Limestone	2.00	hot	2.0	—
T-19A	[19]	20	Granite	4.00	residual	2.0	slow
T-19B	[19]	20	Granite	4.00	residual	2.0	rapid
T-20	[20]	44	Basalt	none	residual	standard	slow
T-21A	[21]	24	Gravel	1.00	hot	measured	—
T-21B	[21]	24	Gravel	1.00	residual	measured	slow
T-22	[22]	32	Basalt	varied	residual	5.0	slow
T-23	[23]	39	Siliceous	1.00	residual	1.0	slow
T-24A	[24]	43	Gravel	uniform	residual	standard	slow
T-24B	[24]	43	Gravel	uniform	residual	standard	rapid
T-25A	[25]	47	Gravel	1.50	residual	instant	slow
T-25B	[25]	46	Dolomite	1.50	residual	instant	slow
T-26	[26]	37	Limestone	1.00	residual	1.0	slow
T-27A	[27]	50	Limestone	uniform	hot	5.0	—
T-27B	[27]	50	Limestone	uniform	residual	5.0	slow
T-28	[28]	38	Granite	1.00	residual	2.5	slow
T-29	[29]	35	Gravel	0.25	hot	2.7	—
T-30	[30]	49	Limestone	2.00	residual	2.5	slow
T-31	[31]	28	Siliceous	0.50	residual	8.0	rapid
T-32	[32]	28	Siliceous	0.50	residual	2.0	slow
T-33	[33]	40	Siliceous	0.50	hot	5.0	—

4. Influence of Fire Characteristics

In this section, the influence of each fire characteristic is evaluated. Contrary to the chronological order of a natural fire event, the influence of maximum temperature is discussed first as it is the most well-documented characteristic in the literature. It is intended that by recognizing the effects of maximum temperature first, the less-documented fire characteristics can be subsequently evaluated with greater clarity.

4.1. Influence of Maximum Temperature

Figure 3 presents the averaged relative strength of hot and residual tests for a range of maximum temperature exposures. The averaged values consist of findings from 37 different studies. To provide an understanding of the variation in existing data, upper and lower limits of the evaluated test data are given (dotted line). Eurocode prescribed strength reductions for siliceous aggregate (dashed line) are also given [34,35].

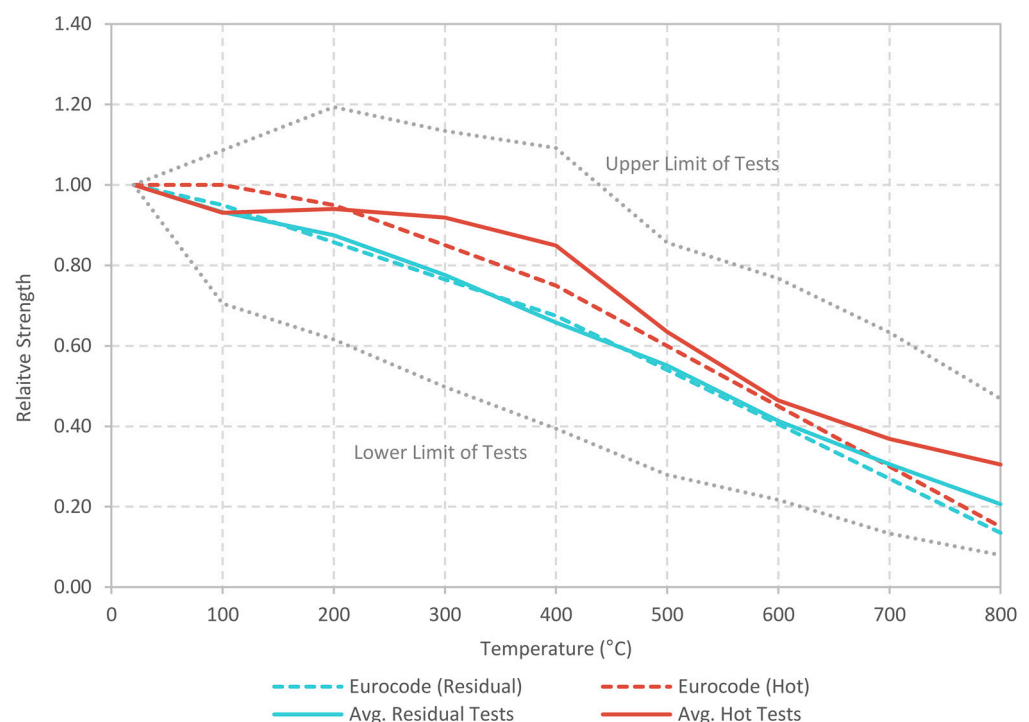


Figure 3. Relative strength of concrete for hot and residual conditions. [Includes Tests from T-7A to T-10, T-12 to T-20, T-21A to T-28, T-30 to T-34].

The averaged experimental work shows that increasing maximum temperature has a significant influence on concrete strength. Concrete tested after cooling exhibits lesser strength at every temperature compared with hot tested concrete. This relationship is largely due to the influence of cooling, which is examined in Section 4.4. To address the influence of maximum temperature specifically, discussion focuses on the response of the hot tested profile.

Concrete strength exhibits three trends when exposed to elevated temperature. At lower temperatures below 350 °C, strength loss is relatively minor. Some of the experimental work, such as by Diederichs et al. [14] and Fu et al. [15], even observed moderate strength gains in the low temperature ranges. The extent of these gains can be seen in the steep rise of the dotted upper limit line. Castillo and Duranni [12] proposed that this strength gain results from stiffening of the cement gel due to the evaporation of concrete moisture. As such, changing concrete properties, such as porosity and moisture content, can have a notable impact on delaying strength loss at low maximum temperatures.

In the mid-range temperatures, 350–600 °C, strength drops sharply. By 600 °C, relative strength levels of 45% and 41% can be expected for hot and residual test averages, respectively. In this temperature range, the concrete becomes substantially dehydrated, such that the full influence of micro-cracking, cement and aggregate decomposition, and thermal expansion stresses is realized [36].

Above 600 °C, severe degradation can be expected, with as much as 90% strength loss by 800 °C. This reduction illustrates the substantial influence that maximum temperature has on the strength of concrete. At these higher temperatures, specimens can often be broken up into gravel by hand [37]. The rate of strength loss above 600 °C, however, is slightly less severe than in the mid-range temperatures. This lessening rate may be attributed to the calcination or crystallization of aggregates [12].

4.2. Influence of Heating Rate

During concrete heating, a thermal gradient develops between a section's outer layers and inner core. This gradient induces thermal stresses between the different constituents of the concrete, which in turn produces micro-cracking and compressive strength loss. It is by this mechanism that variable rates of heating can influence concrete strength.

For evaluation, experimental work is divided into low and high heating rates. A low heating rate is defined as a rate less than 3 °C/min, with high heating being that greater than 3 °C/min. This definition of low and high rates is based off the median heating rate of the available experimental work. For comparison, the standard fire has an average heating rate of 33 °C/min (between 0 °C and 800 °C) and the Cardington fire tests give an average rate of 18 °C/min for a typical compartment fire [38]. Although 3 °C/min is a comparably much lower rate of heating, the experimental work has focused on this level due to the relative simplicity of its application. These low heating rate tests are also not without merit, as they are still valid for potentially smaller natural fire events.

To control for the effects of the other fire characteristics, only tests with a similar exposure duration have been included. Hot and residual tests have been separated for comparison. All residual tests feature a similar cooling regime.

Figures 4 and 5 present the relative concrete strength of hot tested specimens for high and low rates of heating. The average profile for the plotted tests is indicated by the dashed line. The experimental work is found to be in good agreement, with only a few outliers from the test average. Figures 6 and 7 similarly present the relative concrete strength of residually tested specimens. The low heating rate tests show very good agreement, but greater fluctuation is observed for high heating. This may be due to the wider selection of heating rates presented on the plot, ranging from 5 °C/min to instantaneous heating.

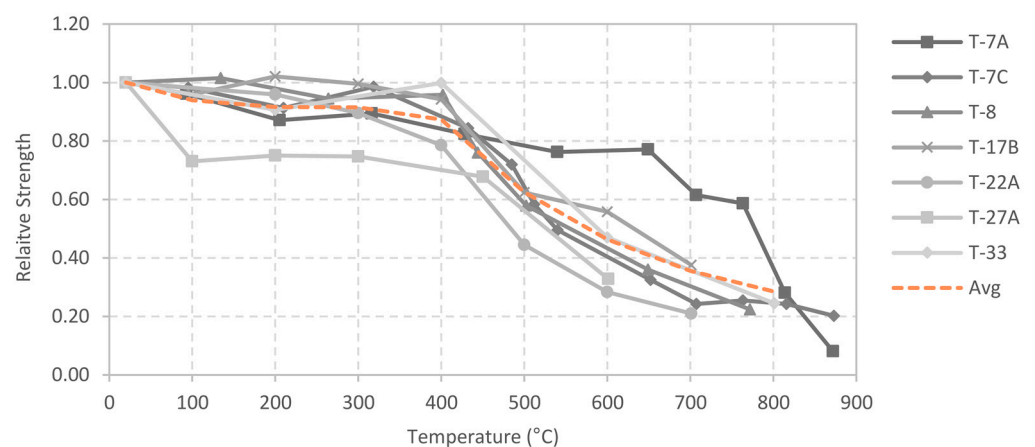


Figure 4. Relative strength of hot tested concrete with high heating rates.

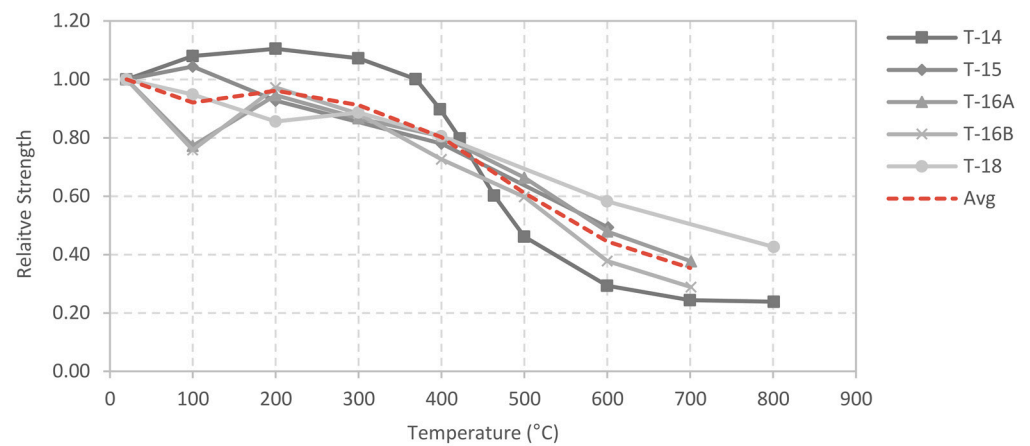


Figure 5. Relative strength of hot tested concrete with low heating rates.

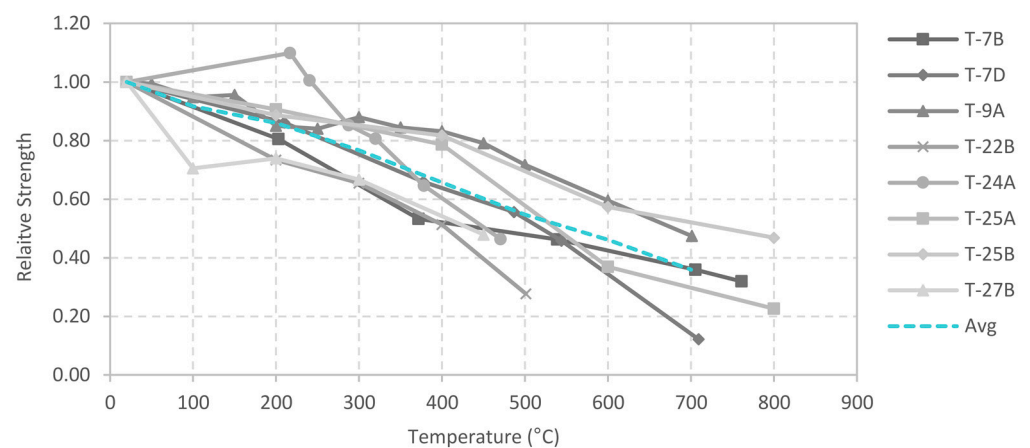


Figure 6. Relative strength of residually tested concrete with high heating rates.

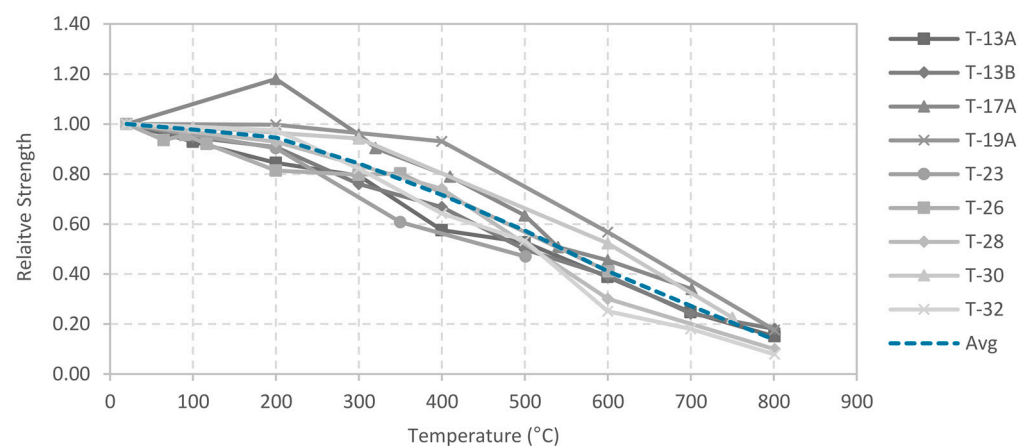


Figure 7. Relative strength of residually tested concrete with low heating rates.

Figure 8 records the average strengths of the experimental work for direct comparison. The average profiles have been truncated at 700 °C due to a shortage of available tests beyond this temperature. The Eurocode prescribed profiles for hot and residual siliceous concrete are also given as a baseline [34,35].

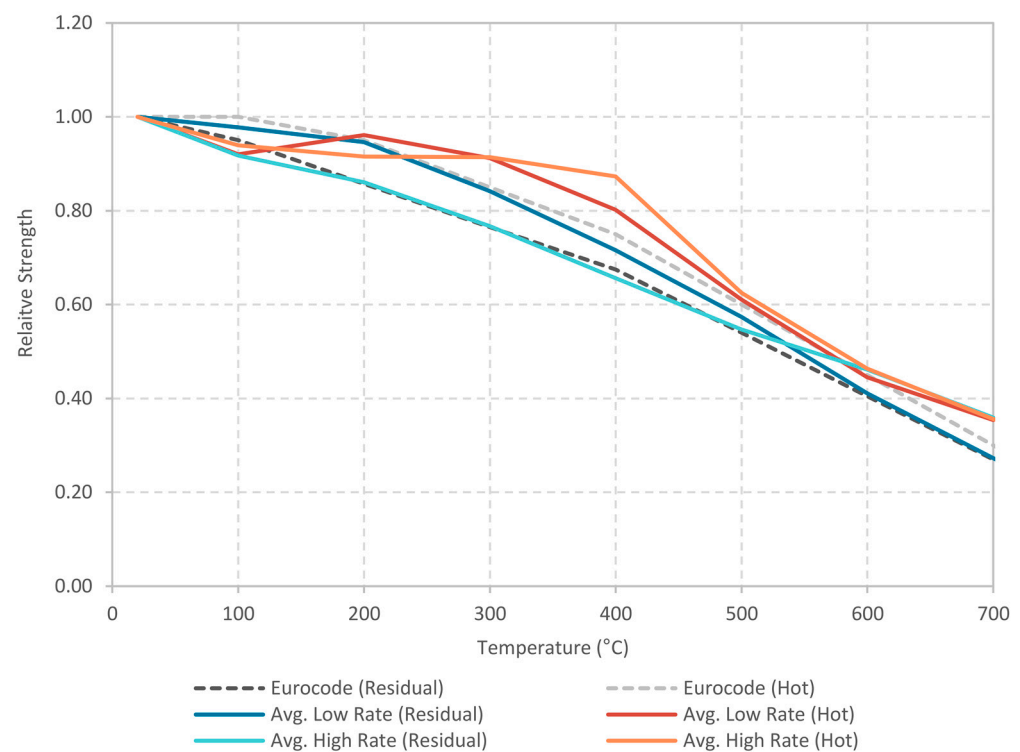


Figure 8. Relative strength of concrete due to low or high rates of heating.

Considering the average profiles, no clear trend emerges. In general, high rates appear to result in slightly greater strength reduction. This is most notable for the residually tested concrete at lower temperatures around 200 °C. However, at any given temperature, the effect of heating can produce higher, lower, or identical strengths. In particular, beyond 500 °C, all four heating regimes converge and result in comparable strength levels.

A justification for this minor and fluctuating influence may be due to the conflicting nature of heating mechanisms. At higher heating rates, large thermal gradients develop, causing greater strength reduction due to extensive micro-cracking. However, at the same time, the rapid expulsion of moisture from the concrete strengthens the adhesive action of the cement gel. These two mechanisms act in contrast resulting in similar concrete strengths for low and high heating rates. Mohamidbhai [22] proposed that at temperatures above 600 °C, the majority of the moisture is removed, and the micro-cracking occurs regardless of the heating rate. Therefore, low and high rates can be expected to result in similar strength losses at high temperatures, which is reflected in Figure 8.

It should be noted that although heating rate does not have a large impact on concrete strength, it is often cited as having a significant impact on explosive spalling [39]. Explosive spalling is a phenomenon in which exterior portions of a concrete specimen violently spall off during heating. This effect significantly reduces the elements cross-section and potentially exposes internal reinforcement, greatly reducing sectional strength. Castillo and Durrani [12], Noumowe et al. [26], and Phan and Carino [27] all reported major spalling in their high-strength concrete (HSC) samples but none in their NSC. Noumowe et al. [39] observed explosive spalling in HSC specimens at heating rates as low as 1 °C/min. It is well documented that NSC is often unaffected by spalling compared with HSC. However, in view of the potential severity of explosive spalling, heating rate is a factor that should be given due consideration.

4.3. Influence of Exposure Duration

Exposure duration refers to the time for which concrete is subjected to elevated temperatures. For a natural fire, exposure duration would intuitively be taken from the

time when the fire starts to when it is fully extinguished. This overall duration, however, is not often reported in the literature. Instead, exposure duration is typically reported as the time from when heating ends to the time when hot testing or residual cooling begins. During this period, the concrete is exposed consistently to the maximum temperature. Defining exposure duration in this way makes temperature control easier during testing. It also has the added benefit of allowing its influence on concrete strength to be separated from that of variable heating and cooling rate.

To evaluate the influence of exposure duration, this section focuses on the work of Carette et al. [11] and Mohamidbhai [22]. Both studies specifically investigated variable exposure durations, ranging from hours to months. For comparison, complimentary experimental work has been selected with similar heating, residual cooling, calcareous aggregates, and specimen sizes.

Figure 9 presents the relative strength reductions for concrete when exposed to a maximum temperature of 400 °C for various durations. Figure 10 provides the same for a 600 °C temperature. An exposure duration of “uniform” indicates continuous exposure was applied until the specimen’s internal temperatures were measured to match the furnace temperature. An exposure duration of “0-hr” indicates the specimen began cooling immediately after maximum furnace temperature was reached.

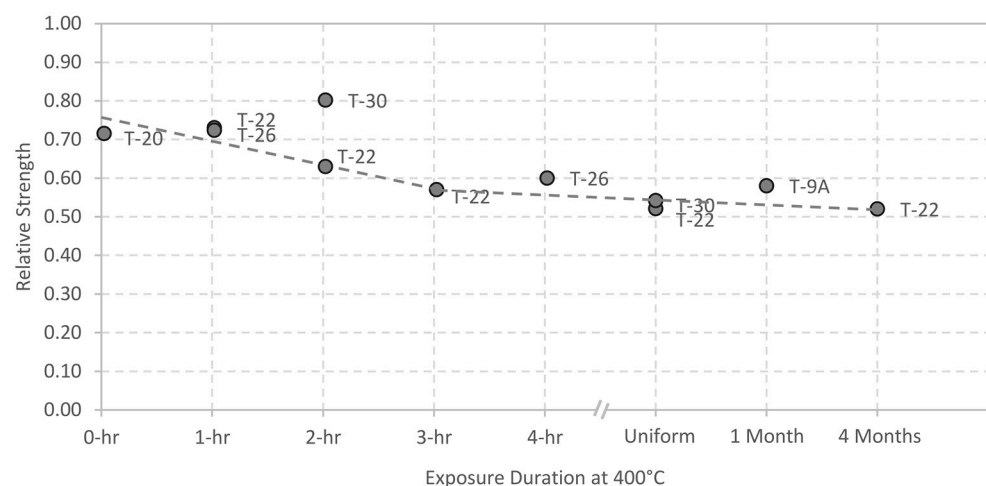


Figure 9. Relative strength of concrete at 400 °C with various exposure durations.

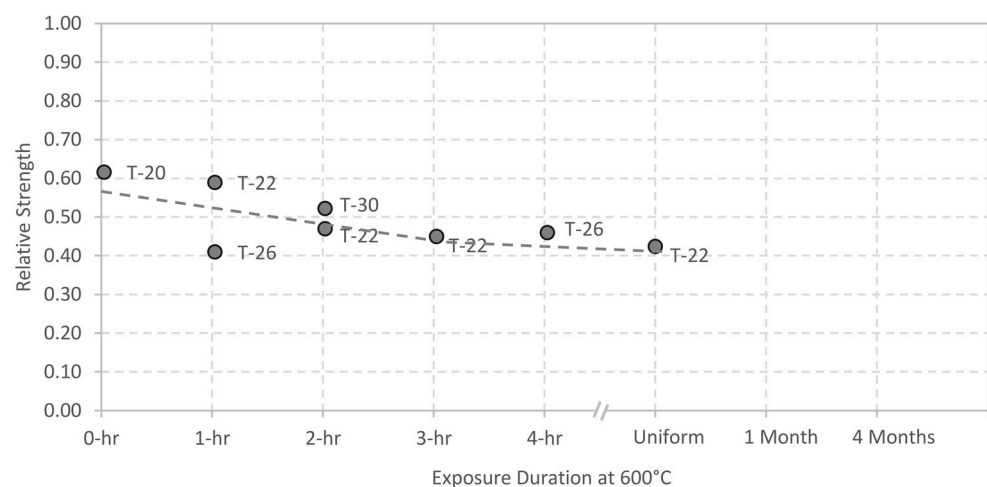


Figure 10. Relative strength of concrete at 600 °C with various exposure durations.

The results show that the majority of strength loss occurs early in the exposure process. Test T-20 with an exposure duration of 0 h, exhibited relative strength loss of 29% at 400 °C and 38% at 600 °C.

As exposure duration increases, strength reduction follows two trends. Up until 3 h, moderate strength reduction continues to occur. Beyond 3 h, insignificant further strength reduction is observed. Even at extreme durations of one and four months, strength levels are comparable to the 3 h and 4 h exposure durations. Those two trends are presented in Figures 9 and 10 by the dashed lines.

The rationale behind the relationship can be attributed to the internal temperatures within the concrete. At shorter durations, there is a temperature lag between the outside surfaces of the concrete and the inside. During this period, continued cracking and strength degradation occurs as the internal temperature increases. Once a uniform internal temperature is reached, the mechanisms of strength loss become minimal.

Based on the reviewed experiments, a uniform internal temperature can be expected in typical laboratory test specimens after 3 h of constant exposure. For larger concrete cross-sections, the time it takes to reach a uniform internal temperature varies greatly.

4.4. Influence of Cooling Rate

As previously observed in Figure 8, the residual strength of concrete after cooling is notably lesser when compared with its hot strength. The cause of this additional strength loss is due to the development of internal temperature gradients, similar to the heating process. Because these gradients form in the opposite direction of heating, they generate new stresses and new cracks that further reduce concrete strength [26].

Considering the effects of a natural environment, variable rates of cooling can be present, ranging from slow cooling in a smoldering compartment to rapid cooling from firefighting efforts. To evaluate the effect of cooling, the reviewed experimental testing is divided into two rates: slow and rapid cooling. In this paper, cooling rate is taken from the time furnace temperature begins to decline until the furnace reaches ambient temperature.

Slow cooling occurs when a test specimen is either cooled within the test furnace or taken outside into the ambient environment. Internal specimen temperature by Lee et al. [19] showed that these two different cooling methods produce very comparable cooling rates. Savaa et al. [30] and Morita et al. [23] indicated that slow cooling results in a rate of 0.4 °C/min to 1.0 °C/min. Slow cooling can subsequently be defined as having a rate less than or equal to 1.0 °C/min.

Rapid cooling is achieved in experimental work by exposing the specimen to water during the cooling stage. Water quenching or spraying techniques are typically applied by submerging or spraying the specimen with ambient temperature water for a prolonged duration. In the specific case of 150 mm cubed specimens, Botte and Caspele [10] identified that from an elevated temperature of 600 °C, quenching is equivalent to a cooling rate of 30–40 °C/min. The results of this experiment demonstrate the magnitude of possible cooling rates that can occur during natural fire scenarios.

Figures 11 and 12 display the relative concrete strength of specimens exposed to slow and rapid cooling. Only tests of similar heating rate and exposure duration are presented. All the rapid cooling studies were conducted immediately after cooling was complete, avoiding the influence of potential strength recovery. The overall profile of the experiments for both cooling regimes were found to be in good agreement with one another. Due to the additional inconvenience of conducting rapid cooling tests, their number in the literature is very small.

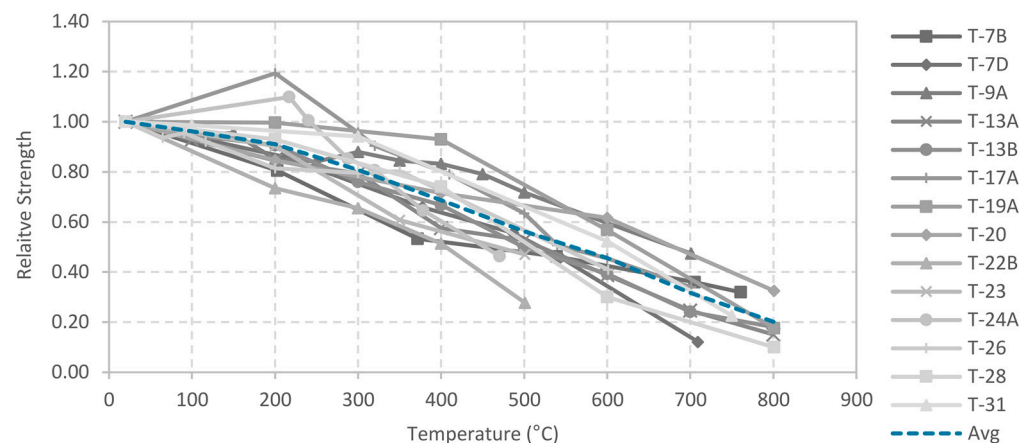


Figure 11. Relative strength of concrete with slow cooling.

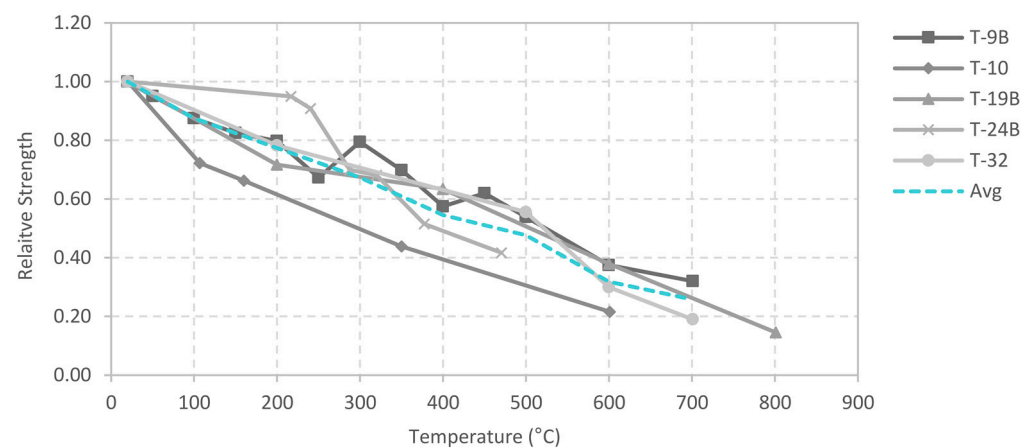


Figure 12. Relative strength of concrete with rapid cooling.

Figure 13 presents the averaged cooling profiles along with the Eurocode siliceous baseline and the averaged profile of the hot tested concrete from Figure 7. It can be seen that an increased cooling rate results in greater strength reduction. Even at a slow rate of cooling, lower residual strengths can be expected when compared with the hot tested specimens. On average, 6% greater strength loss is observed between hot to slow and an additional 10% is observed between slow to rapid.

The strength loss due to cooling is not constant with temperature. At low temperatures circa 100 °C, the residual concrete exhibits only minorly lesser strength levels as compared with hot tested concrete. However, in the mid-temperature range of 200 °C to 500 °C, the influence of cooling becomes significant. The maximum difference between hot and rapid cooling is 29% when at 300 °C. This trend indicates the extreme importance of considering the influence of cooling rates in moderate fire events. At the higher temperatures above 600 °C, the three profiles display some convergence. Due to a shortage of test data, the rapid cooling profile is discontinued early. Specific testing by Lee et al. [19] indicates that at temperatures of 800 °C, slow and rapid cooling continue to converge and reach comparable strength levels.

The lack of agreement between the rapid cooling and Eurocode profile should also be noted from Figure 13 [34]. This is the only fire characteristic for which a significant and unconservative relationship is observed between the code and test results. When assessing the residual strength of concrete, this potential limitation in the code prescribed values should be considered.

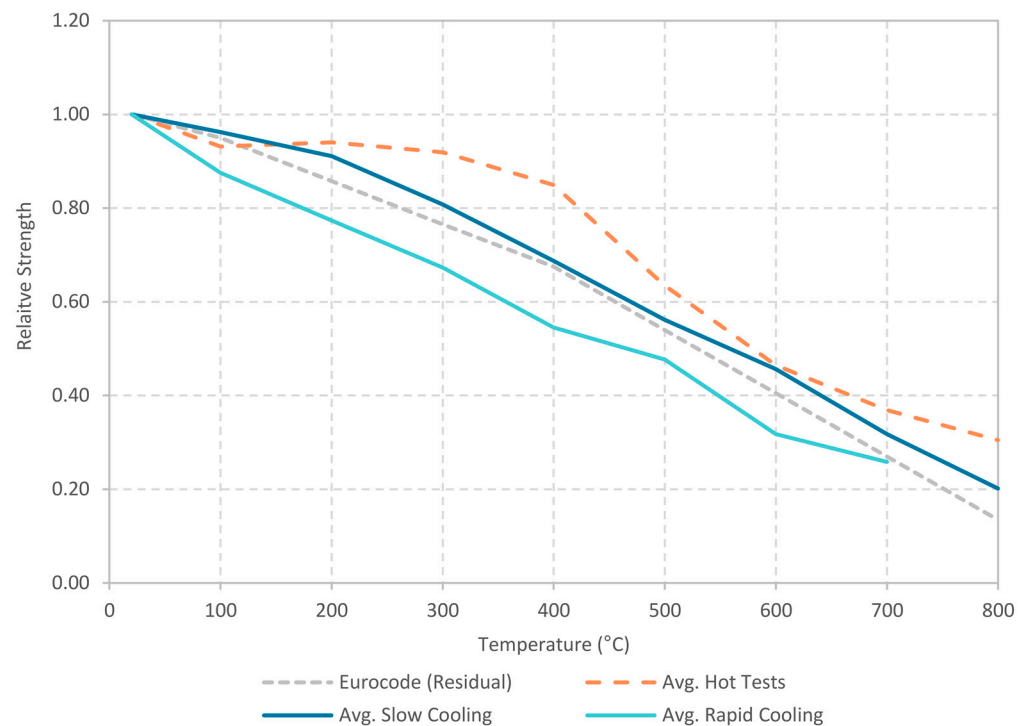


Figure 13. Relative strength of concrete due to slow or rapid cooling.

Post-Fire Strength Recovery Due to Cooling Rate

Post-fire strength recovery is a process by which fire damaged concrete can significantly regain strength when cooled with water. This recovery is attributed to the rehydration of the cement [40]. Maximizing water exposure and allowing time for recuring are important factors in facilitating recovery.

The concept of strength recovery has been well investigated in the literature since it was first observed in 1970 by Crook and Murray [41]. Experimental work and reviews often focus on the influence of long-term recuring techniques, such as soaking specimens for weeklong durations [40]. From the perspective of a natural fire event, this duration of water exposure is unlikely. The following experimental work has been reviewed to demonstrate the influence of short-term recuring.

Poon et al. [40] performed experimentation involving a continuation of the test data presented in Table 1 for T-28. After slow air cooling from 600 °C, NSC specimens were recured by water spraying for 2 hrs and then tested after 7, 28, and 56 days. The results show that after 7 days, the concrete recovered 14% of its strength, and after 56 days recovered 19%. This represents a significant recovery. The researchers identified that the 2 hr spraying duration was selected after many trials to be the minimum soaking time for optimized results.

Abramowicz and Kowalski [6] explored the concept of very short duration water cooling. Specimens were either slow cooled in ambient air or rapid cooled by quenching for 10 s, followed by further slow cooling in ambient air. Strength testing was completed the next day. This very short duration immersion and quick testing time produced no significant effect on the specimen's strength compared with the baseline slow cooled specimens.

Based on these findings, rapid water cooling is not sufficient to induce notable strength recovery that is reliably useful for design purposes. This is due to two reasons. Firstly, recuring requires time. When considering the strength of concrete during the natural fire event and the safety of occupants and first responders, insufficient time will have been provided for recuring regardless of water exposure. Secondly, it is important to also consider the geometry of the concrete involved. Far larger amounts of water would be

required for a building, versus 100 mm specimens. To reliably recreate the findings of Poon et al. [40], an extended and intentional recurring effort would be required.

5. Conclusions

Based on the reviewed literature, the following conclusions can be made regarding the influence of each of the four natural fire characteristics on concrete compressive strength:

1. Maximum temperature causes the most significant strength reduction to concrete. At temperatures below 350 °C, strength losses are relatively minor. Strength increase is even possible in the low temperature range depending on concrete mix properties. Beyond 350 °C, strength drops rapidly. By 600 °C, hot and residual tested concrete can be expected to have lost 55% of their ambient strength. In the high temperature ranges, maximum temperature dominates the other fire characteristics and is the principal source of strength reduction.
2. Heating rates have minimal influence on the strength of concrete. At lower temperatures, higher heating rates were found to result in marginally lower strengths. However, the findings fluctuated greatly such that no decisive conclusion can be made. At higher temperatures above 500 °C, both low and high heating rates produced comparable strength losses. The impact of explosive spalling is likely the primary concern when considering the influence of heating rate on the strength of a specific concrete section.
3. Exposure duration was found to have a major but diminishing impact on concrete strength. The majority of strength loss happens very rapidly, within the first minutes and hours of exposure. After several hours of constant exposure, strength loss is comparable to that of concrete exposed for month-long durations. This finding demonstrates the importance of understanding and defining a section's internal temperature gradient. Once a section's internal temperature becomes uniform, negligible further degradation is expected regardless of extended exposure. One item for future consideration that was not found in the literature is the influence of very short duration high-temperature heating.
4. Cooling rate was found to have an important influence on strength. On average, residual tested concrete exhibited 10% greater strength loss compared with hot tested concrete. Comparing slow and rapid cooled specimens demonstrated that higher cooling rates result in even further strength loss. The greatest impact on cooling rate was in the mid temperature range of 200 °C to 500 °C. Above 500 °C, hot tested, slow cooled, and rapid cooled profiles begin to converge and reach comparable strength levels. The possibility of strength recovery due to rapid water cooling was found to be unlikely using typical water-cooling techniques alone.

The intent of this paper is to provide a general understanding of NSC behaviour during natural fire exposure. This allows designers to focus on the parameters that have the largest impact on concrete behaviour and researchers to identify when newly introduced parameters present unusual responses. To achieve this goal, several assumptions were made which limit the validity of the conclusions of this paper. The reviewed experimental work was limited to: unstressed tests, unconfined tests, unsealed tests, ordinary Portland cement, and NSC. Additionally, other NSC parameters were left uncontrolled and broadly accepted within this review. These parameters include water-cement ratio, aggregate-cementitious material ratio, aggregate type, size and content, and geometric dimensions. Future research is needed to address these limitations.

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