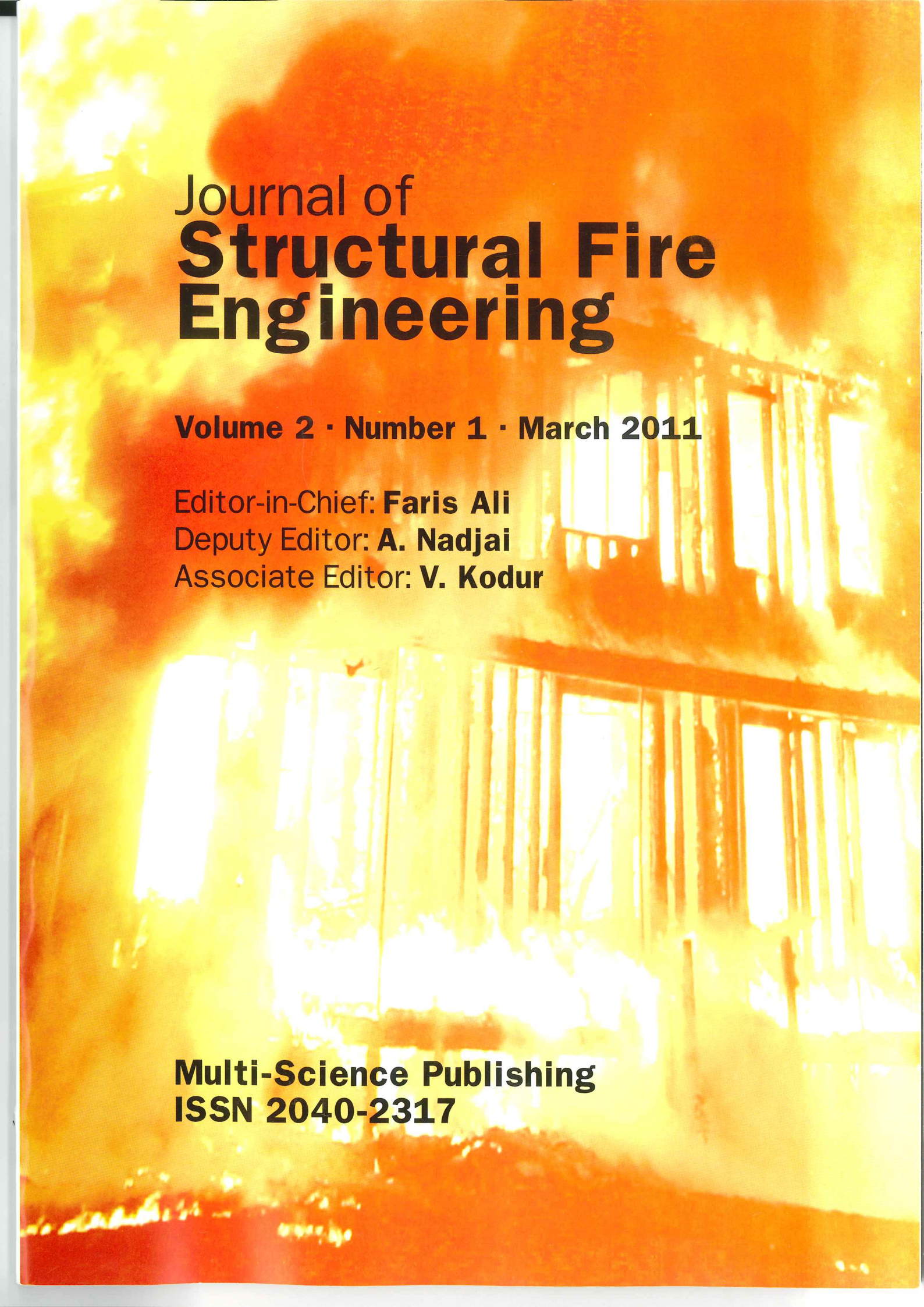

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Test Results and Model for the Residual Compressive Strength of Concrete After a Fire

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ABSTRACT

An investigation into temperature induced degradation of the compressive strength of concrete including that under cooling phase is carried out. The paper gathers and reviews a considerable amount of test data, considering the influence of different test parameters such as initial compressive strength, aggregate type, cooling regime and specimen shape. It is found that the compressive strength of concrete at high temperature is in accordance with the model proposed in the Eurocodes for calcareous concrete. However, during cooling phase, an additional reduction of compressive strength in concrete is observed, which can be as high as 20% of the initial strength for elevated temperatures around 500°C. Finally, a generic concrete model for temperature dependent compressive strength, accounting for both growth and cooling phase of temperature is proposed. The model can be used for simulating fire response of concrete structures subjected to natural fires or for the evaluation of residual load capacity of concrete structures after fire.

Keywords: concrete; fire; compressive strength; residual strength; cooling

1. INTRODUCTION

When concrete is heated, its mechanical properties get modified, leading to reduction in compressive strength and stiffness with increasing temperature. On the contrary, ductility, defined here as the strain required for reaching compressive strength, increases with temperature. During heating, thermal elongation is also observed and an additional strain that is particular to concrete heated under load and is normally referred to as *transient creep* develops.

All these properties have been studied and documented during first heating and some models have been proposed in the literature. The knowledge of the evolution of the properties during first heating is sufficient when the fire resistance of a structural member has to be simulated under a standard fire curve such as those defined in ISO 834 or ASTM E119. According to these time-temperature curves, the gas temperature is constantly increasing and so is the case for the concrete temperatures in the concrete member.

When the structural fire performance of a structure has to be evaluated in a real fire, additional information about the evolution of the mechanical properties of concrete during cooling is required. Real fires indeed exhibit a so called *cooling down* phase, i.e., a phase when the gas temperatures decrease. Responding to these changing conditions, temperatures in a concrete member will also eventually decrease.

If the behaviour of the structure has to be modelled with an advanced calculation model, information is required about all mechanical properties. If simple calculation models are used, the compressive

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strength plays a critical role. In fact, in short columns, the fire resistance may be evaluated on the base of this sole mechanical property. Other properties may play a role, such as the stiffness in slender columns, but the compressive strength is present in all models and used for all situations. A decision has thus to be taken in the constitutive model about the evolution of the compressive strength of concrete during cooling.

Whereas tensile strength of steel may be considered as reversible, at least when the maximum temperature reached does not exceed a certain value (600°C is often quoted)[1], experimental observations show that this hypothesis is not realistic for concrete. In concrete structures that have been severely attacked by a fire, a certain layer of damaged concrete is systematically observed at the surface[2].

The hypothesis of reversible compressive strength, if it could be accepted, would be given by,

$$f_c = f_1(T) \tag{1}$$

where f_c is the compressive strength at temperature T and f_1 is the function that describes the reduction of compressive strength during heating. Different standards may propose different forms for the function f_1.

Eurocode 2[3] does not accept the hypothesis of reversible compressive strengths and recommends, on the contrary, that *possible strength gain of concrete in the cooling phase should not be taken into account*. The term "*strength gain*" in this sentence has to be understood as "a gain compared with the strength at maximum temperature". The most optimistic model that complies with this recommendation is based on the hypothesis that the compressive strength maintains during cooling the value that was obtained at maximum experienced temperature. This can be represented by,

$$f_c = f_1(T_{\max}) \tag{2}$$

where T_{max} is the maximum temperature experienced in the concrete and f_1 is the function that, in Eq. (1), describes the reduction of compressive strength during heating.

Eurocode 4[4] recommends that an additional loss of 10% of the value at T_{max} be applied when the maximum temperature exceeds 300°C. The evolution of the compressive strength is taken as varying linearly from T_{max} down to 20°C. The detailed expression of the model is given in Eurocode 4 and may be conceptually expressed by,

$$f_c = f_2(f_1(T_{\max}),T) \tag{3}$$

Fig. 1 shows graphically the recommendations of Eurocode 4: in dashed line is the function f_1 that gives the reduction of compressive strength during heating from 20 to 1200°C and in solid line is the function f_2 that gives the reduction of compressive strength for a heating up to 500°C followed by cooling to 20°C. The linear variation from a value of 0.60 at 500°C to a value of 0.54 at 20°C can be observed on this Figure.

In order to illustrate the consequence of choosing different material behaviour of concrete subjected to a fire with cooling phase, an example is presented in Figs 2-4. Fig. 2 shows in a dashed line a parametric fire curve obtained from Annex A of Eurocode 1[5] with G = 1.0 and $q_{t,d}$ = 223 MJ/m^2. If, for example, a 32 cm diameter column is subjected to this fire curve, the temperature distribution along the radius will vary as shown in Fig. 3 when calculated using the thermal properties of Eurocode 2. Fig. 2 also shows the evolution of the temperature in the centre, at 1/4, 1/2, 3/4 of the radius and on the surface. The temperature in the centre is still around 28°C after 12 hours and around 21.4°C after 24 hours. Fig. 4 shows the evolution of the normalised compressive strength in the section calculated

using different models (where only concrete strength is considered and the contribution of reinforcing steel is neglected).

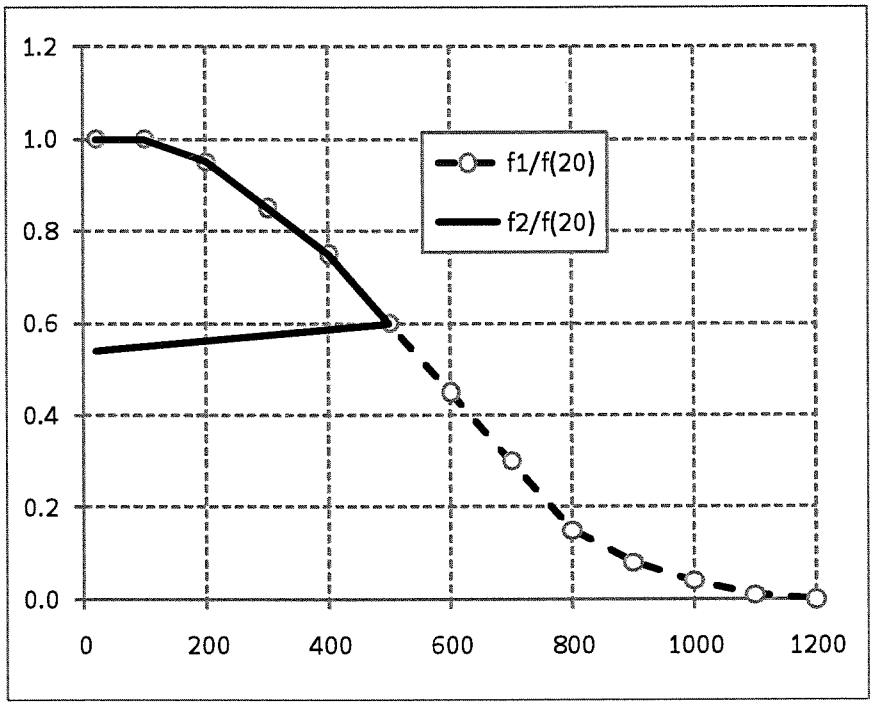

Figure 1. Reduction of compressive strength during heating and cooling

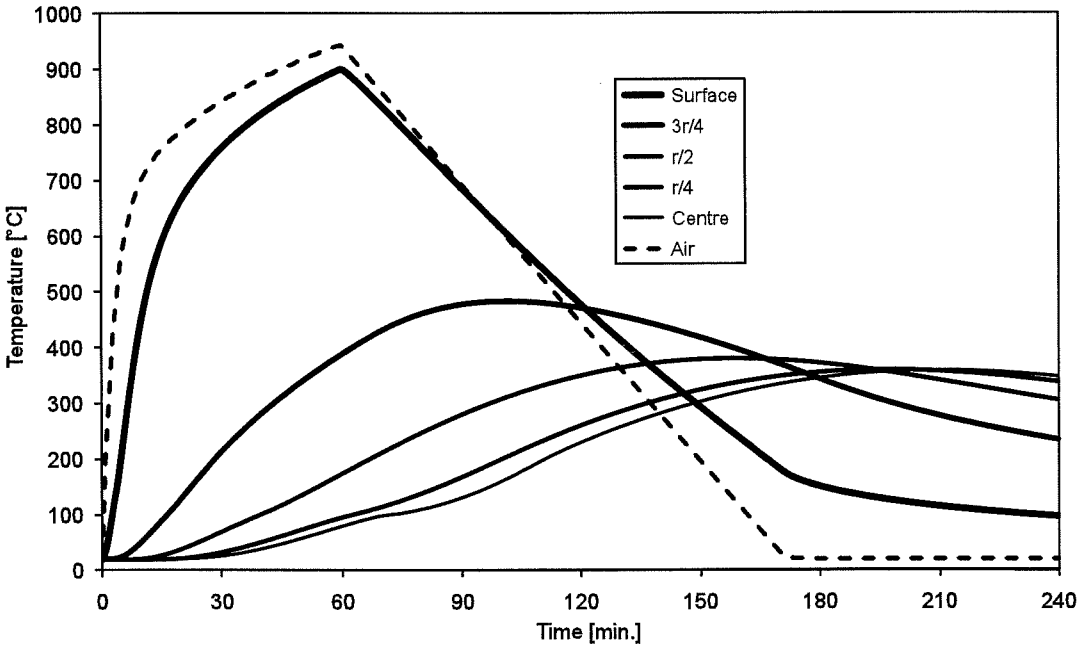

Figure 2. Natural fire curve according to Eurocode 1 and temperatures in the section.

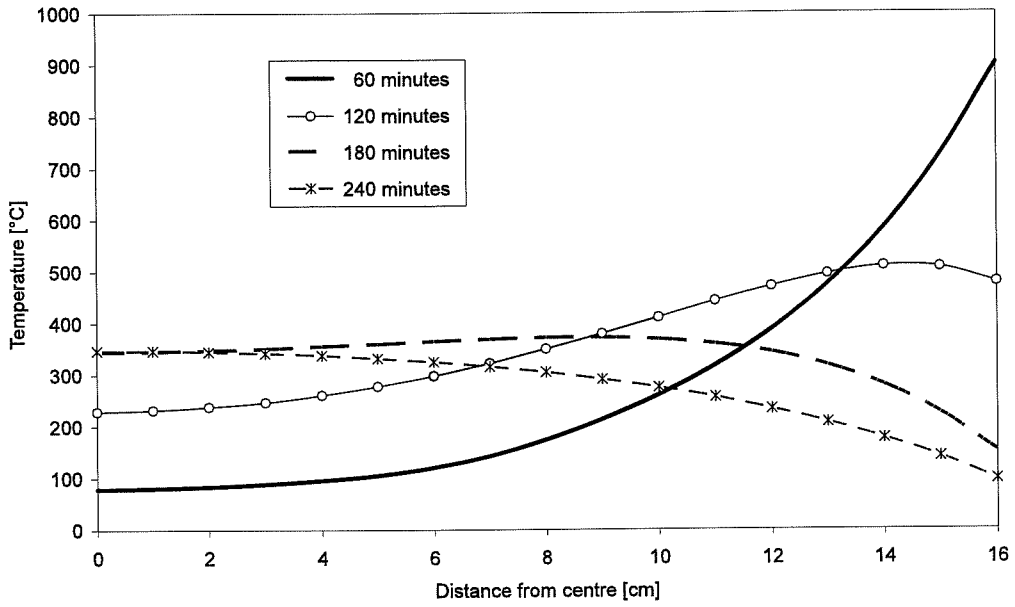

Figure 3. Evolution of the temperature along the radius.

In Fig. 4, the solid line has been calculated using Eq. (1) with the reduction function taken from Eurocode 2. The minimum strength of 0.68 is reached soon after the occurrence of peak fire temperature. The compressive strength of the section recovers progressively during cooling phase, up to full capacity when the temperature is back to ambient in the whole section. The dashed curve in Fig. 4 has been calculated using Eq. (2). The compressive strength of the section continues to decrease for two hours after the occurrence of maximum fire temperature - because temperatures in the section continue increasing after the time of maximum fire temperature - and levels off at 0.58 after three hours of fire exposure. The thin solid line in Fig. 4, which is based on using Eq. (3), shows an additional decrease that develops very slowly; this additional decrease is due to the 10% loss of strength of concrete that develops during cooling to ambient temperature. The difference between the results obtained using Eq. (3) and those obtained using Eq. (2) is hardly visible in the Figure during 4 hours of fire exposure. However, when the temperature of concrete section cools down to ambient, the residual strength of the section according to Eq. (2) has decreased to 0.52.

The fact that it takes a long time for this additional decrease to develop makes this phenomenon dangerous, more than the amplitude of the decrease itself. Indeed, if the applied load ratio happens to be in range from 0.52 to 0.58 in the fire situation, structural failure can occur several hours after the fire has completely finished and at a time when a first inspection of the building may be under way. Such a tragic incident occurred in Switzerland in 2004 when seven members of a fire brigade were killed by the sudden collapse of the concrete structure in an underground car park in which they were present after having successfully put down the fire[6].

Other models than those described by Eq. (2) or (3) can be used for the variation of compressive strength of concrete during cooling, but the fact is that experimental evidence has not been clearly presented to substantiate any of these models, including the models of Eurocode 2 or Eurocode 4. The second author had made some investigation from the literature in the 90's that tended to support a 10% reduction but this information has never been published and has been lost.

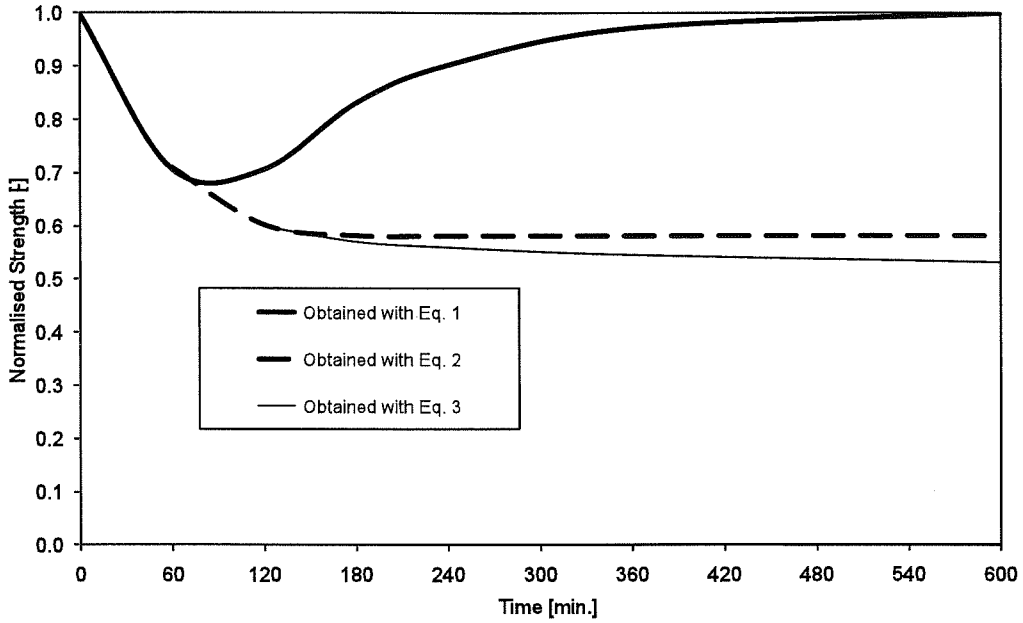

Figure 4. Evolution of the strength in the section based on Eurocode properties.

2. RESEARCH SIGNIFICANCE

Research on residual strength of fire-exposed concrete has been reported extensively over the past few decades. However, the scattered data of the test results with different test parameters makes it difficult to evaluate the residual capacity of concrete after fire. The aim of this paper is to investigate the variation of the compressive strength of concrete after cooling in the light of experimental results, including recent tests if available. The authors believe that the review of the available experimental results and proposing a generic concrete model will contribute to the future development of the codes and guidelines.

3. METHODOLOGY

It is theoretically possible to analyse all thermo-hydro-mechanical processes that develop during heating and cooling of concrete and to establish a mathematical model that would represent these physical process in detail. With such a model could the evolution of all mechanical properties of any concrete type during the whole time-temperature history be predicted. Such a detailed analysis is yet beyond reach at the time being, if only because it would require determining the evolution of basic properties of the aggregates and of the cement paste for a very wide set of parameters.

One possible methodology to investigate the issue above is to perform a series of specific experimental tests. If a batch of concrete is mixed in a sufficient quantity to cast a large number of specimens, then the tests could be conducted according to the RILEM recommendations[7]. Some specimens should be tested in hot condition at different temperatures in order to yield the curve that describes the decrease of compressive strength in the heating phase. Other specimens should then be tested at ambient temperature after having been heated to different elevated temperatures, in order to yield the curve that describes the residual strength after heating and cooling. This procedure has been followed, for example, in Abrams[8] and Malhotra[9], quoted in Schneider[10]. The drawback of this procedure is that the results would be valid only for the particular combination of parameters investigated: concrete mix, curing conditions (sealed or unsealed), load level during heating, heating rate and cooling rate. This procedure is rather appropriate for determining precisely the properties of a well defined type of concrete to be used in well defined conditions, usually for a very big project, e.g.

the concrete vessel for a nuclear reactor that is supposed to be subjected to a well defined fire scenario.

For more general applications, generic properties of concrete have to be established. Generic properties are used, for example, when the mechanical behaviour of two structural systems has to be compared, with no reference to a particular concrete mix. Typical questions are: what is the influence of the length of the reinforcing bars on the intermediate supports of statically indeterminate beams? What is the influence of the buckling length on the fire resistance of a slender column? What is the influence of the position and area of the reinforcing bars in the membrane behaviour of a composite slab? Generic properties are also needed at the preliminary stage of a design, when no information is yet available on the particular mix that will be used. Most of the time, the company that will deliver the concrete - if the building is ever erected - is not even known at that stage. Generic properties are also needed for determining the fire resistance of an element in a small project, where the cost to conduct experimental tests would by far outweigh the budget allocated for the design studies of the building.

This is why another methodology has been used in this study, considered to be more appropriate for deriving a generic material model than the ones described earlier. All the results that could be gathered from the literature have been collected and introduced into a database. It was then possible to differentiate between the tests conducted under hot conditions and those tests conducted after heating and cooling. A statistical treatment of these results should enable determining whether there is a general tendency for one of the two groups to present lower or higher results and, if this is the case, the difference should be quantified statistically. It has to be noted that, with this method, some of the collected results are from studies that only tested under one of the two testing conditions and there is no corresponding to results obtained under the other condition for the same combination of parameters (batch, curing conditions, pre-heating load level, heating and cooling rates). The method is thus valid only if a significantly high number of results can be collected. The hypothesis is that there is no reason why the scientists who conducted the tests would have *systematically* chosen more severe conditions with one method then with the other.

When the results from experimental tests that are reported in a publication present a continuous curve showing the evolution of the strength as a function of the temperature, sample points of the curve have been taken for the statistical evaluation at every 100°C interval.

When the results are presented at temperatures that are not multiples of 100°C, linear interpolation has been used to generate fictitious test results for temperatures that are multiples of 100°C.

Some authors have tested three combinations, namely:

a) stressed during heating and tested at high temperatures
b) unstressed during heating and tested at high temperatures
c) unstressed during heating and tested after cooling.

In this paper, only the results of combination b) and c) have been considered because they differentiate exactly the parameter that is the object of this study. To consider the results of combination a) would yield to the situation that relatively high strengths are attributed to the "tested at high temperature" category, whereas the reason for these high values to be obtained is probably the fact that the specimens were loaded during heating. In fact, if a combination d) "stressed during heating and tested after cooling" had also been tested then combination a) and this combination d) would have been considered.

Only values of residual strength at short term have been considered, in the order of one day after the exposition to high temperatures. Some authors have observed a partial recovery in the long term, with different post-fire curing methods[11,12]. These results have not been considered here because the objective is to derive a model for assessing the load bearing capacity of concrete structures during or immediately after the cooling phase, at a time when the design loads are still being evaluated in the accidental fire situation. This study does not consider the evaluation of the reusability of a burnt building in the long term, when the load combinations for normal situation have to be considered. However, strength recovery of concrete after exposure to fire is favourable and neglecting this effect in the evaluation of long term load bearing capacity is on the safe side.

4. RESULTS OVERVIEW

Fig. 5 and 6 show the normalized compressive strength of concrete at high temperature and after cooling to room temperature, respectively. All test results, together with average values are displayed in the figures. The curves labeled as "+SIGMA" and "-SIGMA" represent the average plus and minus one standard deviation, respectively. A total number of 209 results were collected for tests at elevated temperatures whereas 709 results were collected for tests after cooling to room temperature[8-10, 13-37]. In fact, because part of the results found in the literature are an average of the values of several individual specimens, the information found is gathered from an estimated number of around 400 and 1500 individual specimens for each group.

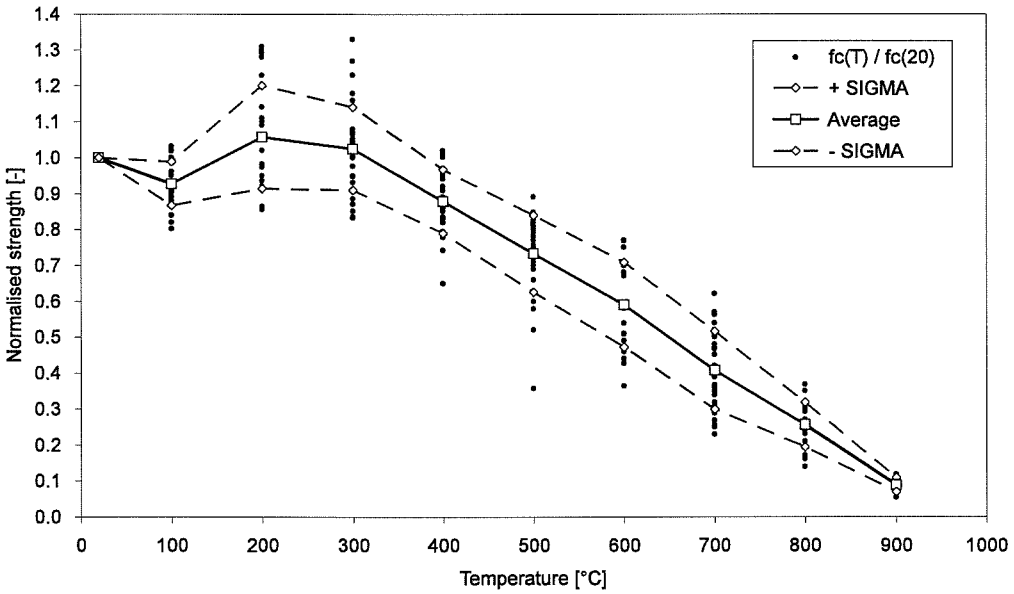

Figure 5. Normalised strength during heating as a function of temperature.

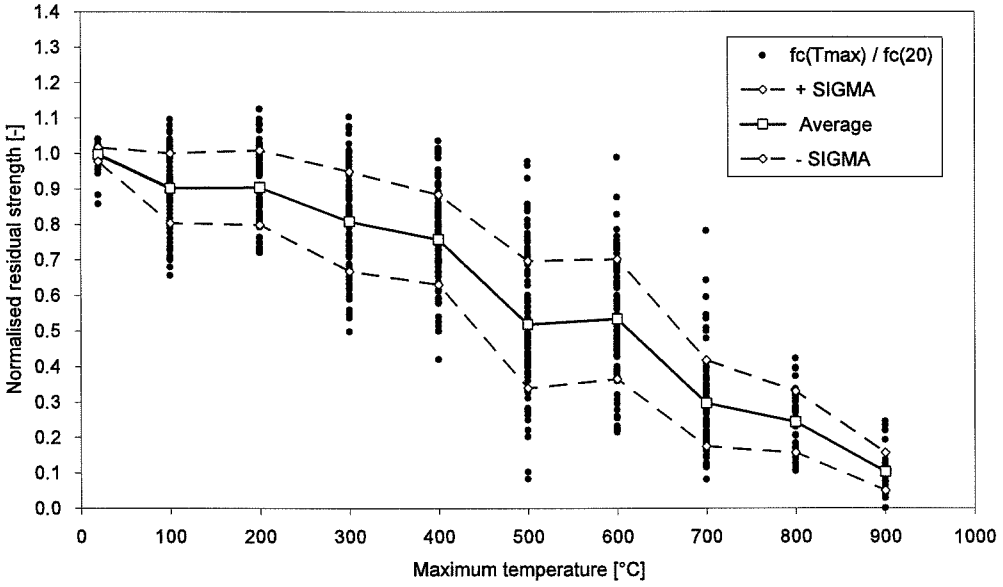

Figure 6. Normalised residual strength after cooling as a function of the maximum temperature.

The largest number of test results is found on residual properties, which is likely due to the fact that this kind of test is technically much easier and also much cheaper than testing the compressive strength in the hot condition.

The original compressive strength at room temperature varies from 20 MPa to 88.5 MPa, of which the majority is under 60 MPa. Aggregate types include carbonate, calcareous, limestone, siliceous, granite and flint. Different cooling methods such as natural cooling, furnace cooling and water cooling were employed in the tests. All the experiments have been performed on cylindrical, prismatic or cubic samples.

It can be observed that the scatter is much wider for the residual strength (Fig. 6) than for the strength at the high temperature conditions (Fig. 5). This could be due to the influence of an additional factor, namely the cooling regime, which will be discussed in a subsequent section.

Fig. 7 shows a comparison between the curve that, taking the average values into account, depicts the decrease of the compressive strength of concrete with temperature (solid line) and the curve that depicts the evolution of the residual strength as a function of the maximum temperature (dashed line). It is clear that an additional decrease of compressive strength can be observed when cooling to ambient temperature. This may be due to the fact that further micro cracks form during cooling either because of temperature gradient in the sample sections or because of different thermal elongation properties in the aggregates and in the cement paste.

From these experimental results, it is now possible to establish a reasonably accurate predictive model.

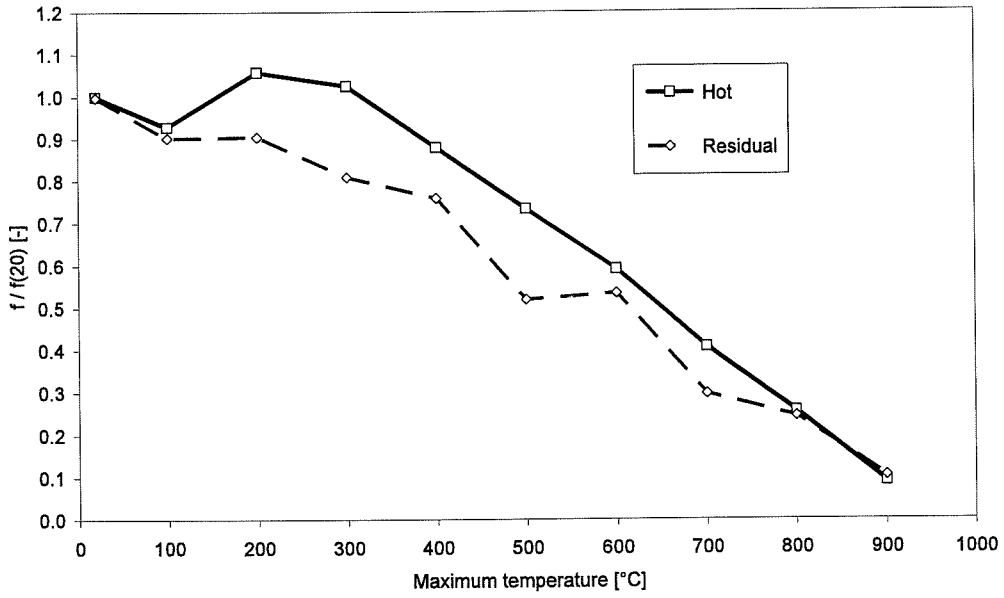

Figure 7. Comparison between strength in the hot state and residual strength.

Fig. 8 shows the reduction (δ, solid line) and relative reduction ($\delta_{rel.}$, dashed line) during cooling calculated from the average curves of Fig. 7 according to Eq. (4) and Eq. (5) as follows:

$$\delta = \frac{f_c(T_{max}) - f_{res}(T_{max})}{f_c(20)} \tag{4}$$

$$\delta_{rel} = \frac{f_c(T_{max}) - f_{res}(T_{max})}{f_c(T_{max})} \tag{5}$$

where $f_c(20)$ is the compressive strength at ambient temperature, $f_c(T_{max})$ is the compressive strength at maximum concrete temperature T_{max} and $f_{res}(T_{max})$ is the residual strength after cooling from T_{max} to ambient temperature.

It appears that the curves are not continuous, displaying two peaks at 500°C and 700°C. These peaks are caused by the non regular pattern of the residual strength curve of Fig. 7. Note that in Fig. 7, for example, the residual strength at 600°C is slightly higher than the residual strength at 500°C, which may seem to be unexpected. This irregular pattern of the evolution of the residual strength may be due to a statistically insufficient number of test results or to physico-chemical phenomena that may occur in the concrete during cooling. This issue has not been addressed in the study presented here.

If a simple mathematical model has to be proposed to represent the differences between the experimental results at elevated temperature and after cooling and if, for the sake of simplicity, it is preferred to have a model that shows a regular variation of the reduction, then the reduction expressed in term of δ seems to be a better candidate, because the variations between the ups and the downs are less pronounced than those observed when the reduction is expressed in term of d_{rel}. The curve shown for d on Fig. 8 is the base of the simple model given by Eq. 6.

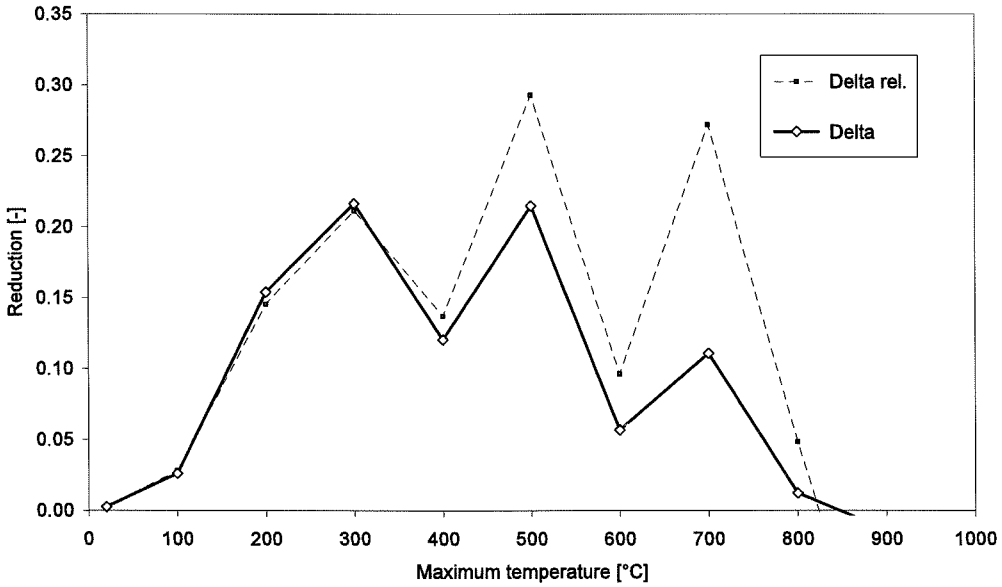

Figure 8. Reduction and relative reduction.

5. DETAILED RESULTS AND DISCUSSIONS

In the analyses, in the previous section, all results of each regime, elevated temperature or residual, have been considered as belonging to a single set, whatever the combination of parameters. In this section, a distinction is made between some parameters to see if some of them might lead to a particular behaviour with regard to the reduction of compressive strength.

5.1. Aggregate type

The first parameter that was investigated is the type of aggregate. In the Eurocode recommendations, two different curves are proposed for the reduction of compressive strength as a function of temperature: one for calcareous aggregates and the other for siliceous aggregates. Fig. 9 shows that, at elevated temperatures, the experimental values are very similar, with the exception of values below 400°C for which the results of siliceous aggregates are markedly higher than the values of calcareous

aggregates. For the residual strength, the values of siliceous aggregates are slightly below the values of calcareous aggregates. At all temperatures, the reduction of strength in siliceous aggregates is higher than the reduction of strength in calcareous aggregates. Since the difference between the two types of concrete is not substantial, and because the type of aggregate may not be known when a generic model is being used, the model proposed in this paper will not make a distinction between those two types of aggregate and both types are included. It must be noted that lightweight aggregate concretes are not covered by this study.

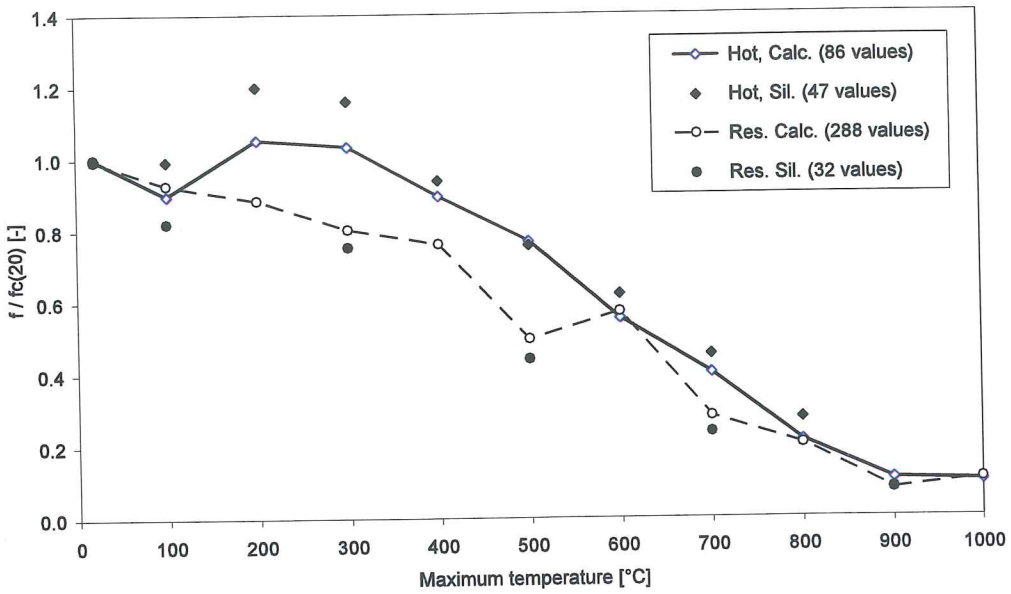

Figure 9. Reduction of strength for two types of aggregate.

5.2. Compressive strength

Most test results have been obtained from normal strength concrete specimens while a few were obtained from high strength concrete specimens. The analysis has been repeated with a distinction between normal and high strength concrete, the boundary being fixed arbitrarily at 60 MPa. Fig. 10 shows that the differences between values measured at elevated temperature and residual values are much lower for high-strength concrete than for normal-strength concrete. For high-strength concrete, there is virtually no difference up to 400°C and, for higher temperatures from 600°C up to 1000°C, the residual strength after heating and cooling even has a tendency to be higher than the strength at elevated temperature. On the other hand, it seems that the relative strength measured at elevated temperature decreases somehow faster for high-strength concrete than for normal-strength concrete.

It has to be noted that the number of test results on high-strength concrete considered at each temperature in this study is lower than the number of tests considered for normal strength concrete. More tests should be included in order to allow better characterization of the behaviour of high strength concrete. Nevertheless, because there is an indication of different behaviour, the test results obtained from high-strength concrete will be excluded from the final conclusions and the generic model presented is limited to normal-strength concrete.

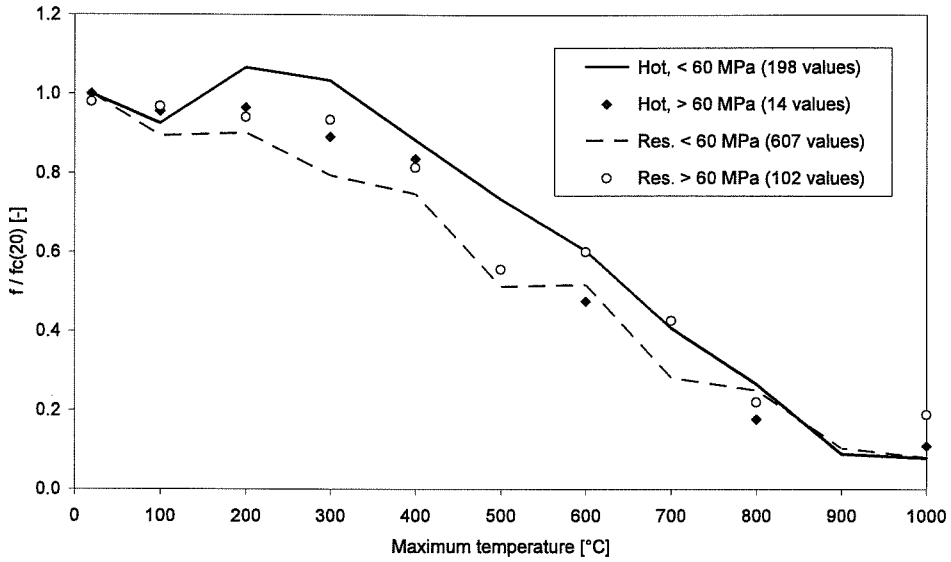

Figure 10. Reduction of strength for normal- and high-strength concrete.

5.3. Cooling regime

Fig. 11 shows the evolution of the residual strength for two different cooling regimes: specimens cooled in water and specimens cooled naturally in air. It can be seen that water cooling causes more damage in strength when compared to air cooling. The figure indicates that loss of strength during cooling may be due to thermal stresses that develop during cooling. Local stresses may appear between the aggregates and the cement paste even if the specimen is at a uniform temperature at any time, due to different thermal expansion between the two constituents. The difference between the two curves of Fig. 11 is likely due also to the temperature gradients generated between the external and the internal parts of the specimen, especially when the cooling rate is faster, e.g. with cooling in water.

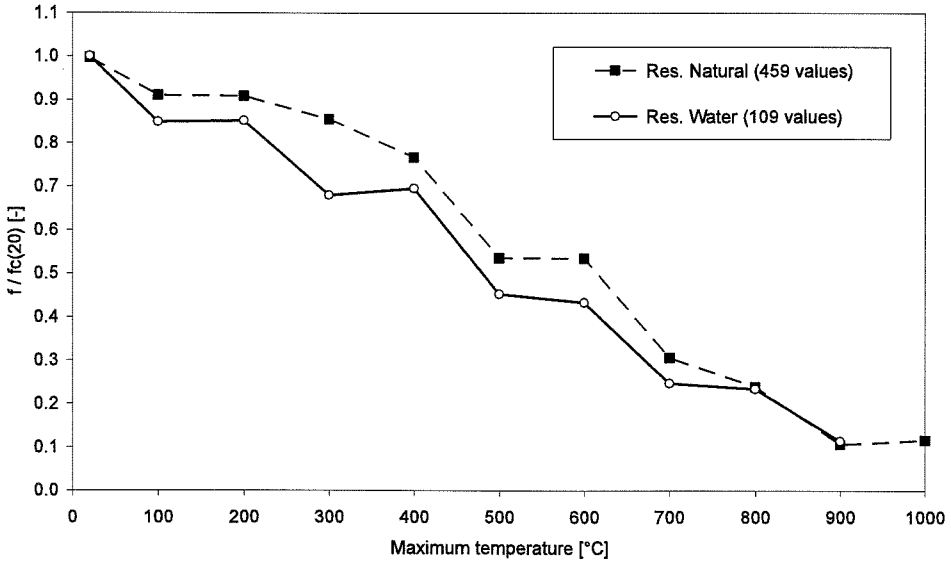

Figure 11. Residual strength for different cooling regimes.

One may develop a model where the influence of the cooling rate is included. More research would be needed to reach this goal. One essential question to be answered would be to know what is measured exactly when testing a specimen after cooling in conditions of non uniform temperature (rapid cooling). Is the more severe strength reduction really a material property, or is it a structural effect occurring at the level of the specimen? Note that, in a structure, different parts of the sections cool down at different rates, see Fig. 2. In this study, results obtained with different cooling regimes will be considered together.

5.4. Shape of the specimens

Fig. 12 represents the evolution of the residual strength for two different shapes of the specimens. The majority of the tests were performed on cubic specimens, either 100 mm or 150 mm in size. Other specimens were tested with a prismatic shape, either with a circular or a square base; the ratio between the height and the dimension of the base is in the order of 2 to 3.

Fig. 12 shows that, at elevated temperatures as well as after cooling, the decrease of normalised strength with increasing temperature is faster in prismatic specimens than in cubic specimens. The failure mechanism is likely to be different in cubic and in slender specimens but this is the case in all conditions, either at room temperature, in the hot condition or after cooling. The difference may be due to a different aspect ratio of the geometrical dimensions leading to a different thermal gradient pattern. Only the cubic specimens show an increase of strength at 200°C and 300°C.

The additional decrease that occurs during cooling is more significant for cubes with low maximum temperatures (500°C and below), whereas it is more significant in prismatic specimen when the maximum temperature is above 400°C. However, because it is not possible to know which specimen shape better represents the behaviour of concrete in a real structure, the shape effect is not considered and all shapes of specimens are considered together in the current study.

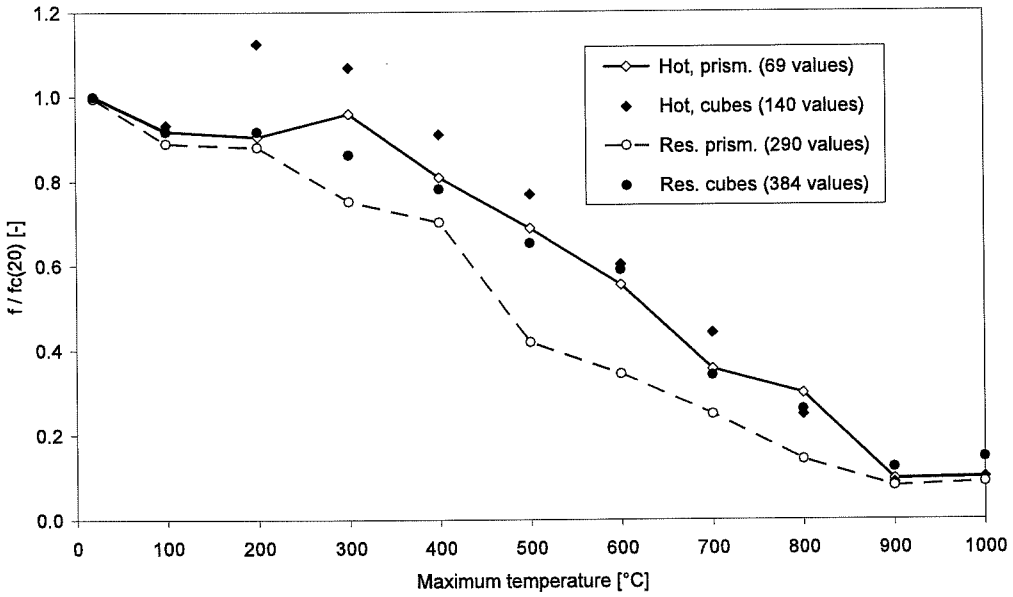

Figure 12. Reduction of strength for different shapes of the specimens.

5. PROPOSED MODEL AND CONCLUSIONS

A statistical evaluation of a large number of tests on the compressive strength of concrete forms the basis of a generic model of concrete in the heating or in the cooling phase of a fire. Table 1 gives the

average value of the relative resistance calculated from the experimental results at elevated temperature (column 2) and after cooling down to 20°C (column 3) when only the specimens with a compressive strength lower than the arbitrarily chosen value of 60 MPa are considered. It can be seen in Fig. 13 that the evolution of the relative strength at elevated temperatures (column 2 in Table 1, curve "Test hot") nearly exactly matches the values recommended in Eurocode 2 for calcareous concrete (column 4 in Table 1, curve "Model hot"), except for the higher values that have been observed at 200°C and 300°C, essentially in cubic specimens. A significant difference between calcareous and siliceous concrete could not be observed in the present study and thus a unified curve is proposed.

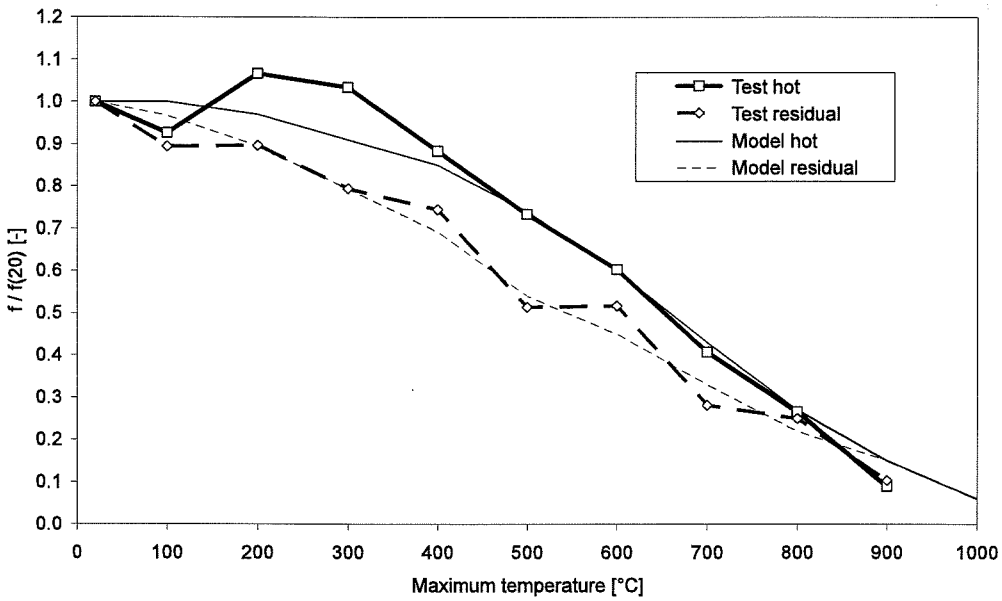

Figure 13. Relative strength from tests and proposed model.

Table 1. Relative strength at high temperature and after cooling to room temperature

Temperature [°C]	Test hot	Test residual	Model hot	Model residual
1	2	3	4	5
20	1.000	1.000	1.000	1.000
100	0.926	0.894	1.000	0.967
200	1.067	0.896	0.970	0.895
300	1.034	0.794	0.910	0.793
400	0.883	0.744	0.850	0.692
500	0.733	0.514	0.740	0.540
600	0.603	0.517	0.600	0.450
700	0.407	0.282	0.430	0.330
800	0.266	0.250	0.270	0.220
900	0.089	0.103	0.150	0.150

If the Eurocode model for calcareous concrete is adopted for the evolution of strength at elevated temperature, then the model presented by Eq. 6 for the additional reduction that occurs during cooling leads to a good fit between the experimental results (column 3 in Table 1, curve "Test residual") and the model (column 5 in Table 1, curve "Model residual") for the residual strength.

$$\delta = \frac{f_c(T_{\max}) - f_{res}(T_{\max})}{f_c(20)} = 0.2\left(\frac{T_{\max} - 20}{480}\right) \quad 20°C \le T_{\max} < 500°C$$
$$\delta = \frac{f_c(T_{\max}) - f_{res}(T_{\max})}{f_c(20)} = 0.2\left(\frac{900 - T_{\max}}{400}\right) \quad 500°C \le T_{\max} \le 900°C \tag{6}$$

CONCLUSIONS

Based on the information presented in this study, the following conclusions can be drawn:

1. During heating regime, the observed strength loss is in good agreement with the model proposed in the Eurocodes for calcareous aggregate concrete.
2. The additional strength loss that occurs during cooling is significantly higher than the 10% of the strength at maximum temperature proposed in Eurocode 4. It can be as high as 20% of the initial compressive strength for temperatures around 500°C.
3. The behavior of high strength concrete may be different - especially under cooling phase - and requires further studies.
4. Further work could also be done with the aim of introducing the influence of the cooling regime on the loss of strength during cooling.

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