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High-Temperature Properties of Concrete for Fire Resistance Modeling of Structures

by Venkatesh K. R. Kodur, M. M. S. Dwaikat, and M. B. Dwaikat

Fire represents one of the most severe environmental conditions to which structures may be subjected and, hence, the provision of appropriate fire safety measures for structural members is an important aspect of design. The recent introduction of performance-based codes has increased the focus on fire resistance evaluation through computer models. One of the key components for fire resistance evaluation of reinforced concrete (RC) members is the high-temperature properties of concrete and reinforcing (prestressing) steel. Even to date, there is limited information on high-temperature constitutive relationships. This paper presents a comparative study of the high-temperature concrete constitutive relationships and illustrates the variations in the high-temperature properties of concrete. The available constitutive relationships for concrete and steel in the American and European standards are reviewed and compared to published experimental results. The different fire resistance predictions that result due to variations in the high-temperature materials properties are illustrated through case studies. Recommendations are drawn for adopting the suitable constitutive relationships for performance-based design of RC members.

Keywords: constitutive relationships; design; fire resistance models; high-temperature properties.

INTRODUCTION

Structural fire safety is one of the primary considerations in the design of high-rise buildings and built infrastructure, where concrete is often the material of choice for structural members. At present, the fire resistance (structural fire safety) of reinforced concrete (RC) members is generally established using prescriptive approaches that are based on either standard fire resistance tests or empirical calculation methods. These approaches have major drawbacks and do not provide rational and realistic fire safety assessment. As the world is moving toward performance-based fire codes, there is an increased focus on the use of numerical methods for evaluating fire performance of structural members. Because the fire performance of structural members depends on the properties of the constituent materials, knowledge of high-temperature properties of concrete is critical for fire resistance assessment under performance-based codes.

Concrete generally has excellent fire resistance properties. The temperature-dependent properties that are important for establishing an understanding of the fire response of RC structures include thermal, mechanical, and material-specific properties such as spalling of concrete and critical temperature for reinforcing steel. Much of the current knowledge on the high-temperature properties of normal-strength concrete (NSC) is based on limited material property tests. There are either limited test data on some high-temperature properties, or there are considerable variations and discrepancies in the high-temperature test data for other properties of concrete.[1-3] These discrepancies and variations are mainly due to the differences in test methods, conditions and procedures, and the environmental parameters (for example, curing and relative humidity) accompanying the tests.[4] Thus, at present, there are no reliable constitutive relationships in codes and standards for many of the high-temperature properties of concrete.[2]

While some limited information is available on high-temperature properties such as strength, modulus of elasticity, thermal conductivity, and specific heat, there are no reliable data on properties such as high-temperature creep and porosity of concrete. Further, until recently, there were no standard test methods for evaluating the high-temperature thermal, mechanical, and special properties of concrete. Only in the last few years have efforts been underway by RILEM and other organizations to develop test methods for characterizing high-temperature properties.[5] Thus, the current available test data and constitutive relations for concrete properties have large variability, because researchers used different test parameters, such as heating rate and loading conditions, for measuring the high-temperature properties.

In addition, there has been a significant amount of material research in recent years, aimed at making concrete durable, strong, energy efficient, and economical. The newly developed concretes, including high-strength concrete (HSC), have questionable performance under fire conditions due to factors such as occurrence of fire-induced spalling.[6,7] While there are different ranges for classifying concrete as NSC and HSC, concrete up to a compressive strength of 70 MPa (10,150 psi) is referred to as NSC, while concrete with compressive strength that exceeds 70 MPa (10,150 psi) is referred to as HSC.[8] Further, many of the preliminary studies show that there are well-defined differences between the properties of HSC and NSC at elevated temperatures, with faster degradation of properties for HSC.[9-11] Therefore, the applicability of existing high-temperature properties, which were developed mostly from the limited fire tests on conventional NSC, to HSC needs to be critically examined.[2] While many of the design standards for concrete structures have been updated with detailed material properties and specifications under ambient conditions, even to date there are no reliable material properties (or constitutive relationships) for the high-temperature properties of HSC in the codes and standards.[8,12-16]

Given the aforementioned variations (for NSC) and scarcity of information (for HSC) with regards to high-temperature properties and constitutive relations, there is an urgent need to characterize constitutive relationships for modeling the fire response of concrete members. This paper reviews the

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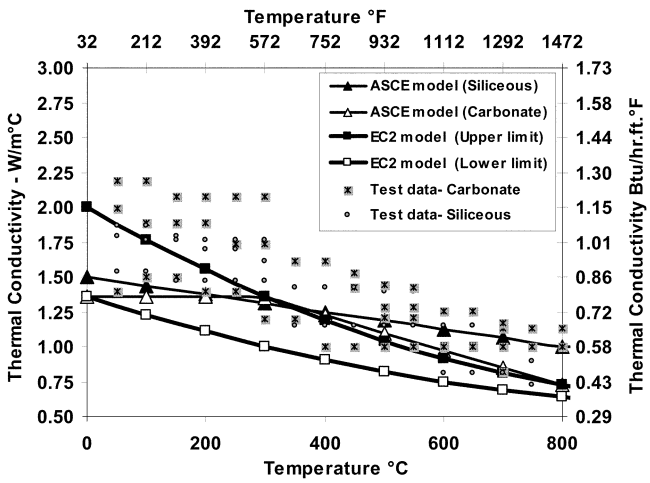

Fig. 1—Variation in thermal conductivity of normal-strength concrete as predicted by different models and as measured in different test programs.

main causes for the high degree of variations in the adopted constitutive relationships for high-temperature properties in the standards and codes. The paper also illustrates the effect of variation of high-temperature properties of concrete on the fire-resistance predictions of RC members.

RESEARCH SIGNIFICANCE

While the development of calculation techniques for predicting the fire response of building components have progressed in recent years, research related to supplying input information (material properties) into these calculations has not kept pace.[10] Much of the information in ACI 216R[14] is based on test data undertaken during 1950s and 1960s, and it contains no comprehensive constitutive relationships for high-temperature properties. In this paper, the available high-temperature constitutive relationships for concrete are compared and recommendations are made on the applicability of these relations in fire engineering design. This will facilitate the use of rational approaches to fire engineering of concrete structures and promote performance-based fire safety design.

PROPERTIES AT ELEVATED TEMPERATURE—STATE OF THE ART

Material properties for fire safety design

The temperature-dependent properties of constituent materials, concrete, and reinforcing (or prestressing) steel are critically important for understanding the fire response of RC structures. These properties include: a) thermal; b) mechanical; and c) material-specific properties such as spalling in concrete. The thermal properties determine the extent of heat transfer inside the material, whereas the mechanical properties determine the extent of strength loss and stiffness deterioration. In addition, spalling can play a significant role in some types of concrete. These properties vary as a function of temperature and depend on the composition and characteristics of concrete.

The thermal properties that influence the temperature rise and distribution in a concrete structural section are thermal conductivity, specific heat, and mass loss. The mechanical properties that determine the fire performance of RC members are strength, modulus of elasticity, thermal expansion, and creep of the constituent materials at elevated temperatures. Creep, often referred to as creep strain, is defined as the time-dependent plastic deformation of the material. At normal stresses and ambient temperatures, the deformation due to creep is not significant. At higher stress levels and at elevated temperatures, however, the rate of deformation caused by creep can be substantial. Hence, the main factors that influence creep are the temperatures, the stress level, and their duration.

In addition to thermal and mechanical properties, other properties, such as spalling in concrete and bond between concrete and reinforcing bar, influence the fire performance of a structural member. To predict the spalling in concrete under fire conditions, additional properties such as porosity are required. These properties are unique to concrete and are critical for predicting fire performance.

All of the aforementioned properties vary as a function of temperature and have to be properly accounted for in tracing the fire response of RC members and for determining fire resistance. Further, the variations of many of the properties at high temperatures are quite sensitive to small changes in concrete ingredients and environmental conditions (humidity and temperature rise). As an example, the thermal properties are significantly influenced by the type of aggregate and composition of the concrete mixture.[4]

Thermal properties—test data

Thermal properties that have a direct influence on the thermal behavior of any concrete-made structural member include thermal conductivity, specific heat, and mass loss. There have been limited test programs for characterizing thermal properties of concrete materials under high-temperatures.[17-23] Based on these test data, empirical formulas for thermal properties of different types of concrete have been developed.[12,17,22,24]

Figure 1 illustrates the large variation in thermal conductivity of NSC obtained in different tests plotted as a function of temperature, and Fig. 2 shows compiled test data related to heat capacity of NSC. The test data are compiled from different studies.[17,18,21,22,25-27]

The standard deviation of the compiled test data of thermal conductivity ranged from 0.22 W/m·°C (0.127 Btu/h·ft·°F) at room temperature to 0.18 W/m·°C (0.104 Btu/h·ft·°F) at 800 °C (1472 °F) for siliceous aggregate concrete. For carbonate aggregate concrete, the standard deviation for thermal conductivity ranged from 0.4 W/m·°C (0.23 Btu/h·ft·°F) at room temperature to 0.16 W/m·°C (0.092 Btu/h·ft·°F) at 800 °C (1472 °F). On the other hand, for heat capacity, the standard deviation of the compiled test data ranged from 0.56 MJ/m^3·°C (6.72 Btu/ft^3·°F) at room temperature to

1.1 MJ/m^3·°C (13.2 Btu/ft^3·°F) for siliceous aggregate concrete. The corresponding values of standard deviation for the heat capacity of carbonate aggregate concrete ranged from 0.6 W/m·°C (7.2 Btu/h·ft·°F) at room temperature to 7.8 MJ/m^3·°C (93.6 Btu/ft^3·°F) at 750 °C (1382 °F). These ranges of values illustrate the significant variation in thermal properties of concrete.

The large variation in these data can be attributed to the differences in test methods, conditions, procedures, and measurement techniques. It can be seen from Fig. 2 that the aggregate type has significant influence on thermal capacity of concrete. The high heat capacity for carbonate aggregate concrete in temperature range of 600 to 800 °C (1112 to 1472 °F) is due to the endothermic reaction that absorbs a significant amount of energy. This high heat capacity in carbonate aggregate concrete helps in minimizing spalling and enhancing fire resistance.

Very few tests were carried out to characterize other high-temperature thermal properties of concrete, such as density changes, porosity structure, and spalling. These tests usually came along with elemental tests and were not characterized for performance-based design. Therefore, such properties were not addressed properly in the literature.

Moreover, the vast majority of the existing data on thermal properties of concrete has its origin to the studies carried out in 1960s and 1970s. Most of these tests were limited to room-temperature experimentation; the remaining tests were either for residual testing or at-elevated-temperature testing.[4] A major portion of these reported test data belong to rocks and minerals and nonstructural concrete components,[4] and a large percent of them serve low-heating-rate applications such as nuclear vessels.[3]

Thermal properties—constitutive models

Thermal conductivity and heat capacity from the available constitutive models in Eurocode 2[24] and the ASCE *Manual* No. 78 on structural fire protection[12] are also plotted in the Fig. 1 and 2. The high-temperature constitutive models for thermal properties of concrete are presented in Table A.1 in the Appendix. These properties are generally expressed as a function of temperature in the Eurocode and as a function of aggregate type and temperature in ASCE manual. Whereas the ASCE constitutive model accounts for significant heat capacity that occurs in carbonate aggregate concrete in the temperature range of 600 to 800 °C (1112 to 1472 °F), the Eurocode constitutive model ignores this effect and has the same heat capacity constitutive model for both siliceous and carbonate aggregate concrete. Also, the Eurocode provides two limits for the thermal conductivity of concrete but with no reference to concrete type. In addition, the Eurocode distinguishes between the concrete types for thermal elongation only. The ASCE constitutive model was developed for NSC only, whereas the Eurocode constitutive model was developed for both NSC and HSC. Based on experimental studies, Kodur et al.[28] extended the ASCE constitutive model for HSC. On the other hand, the reported test data show that the thermal conductivity and heat capacity of concrete are also dependent on moisture content and concrete porosity, not on temperature only.[3,4] Such interdependencies are not taken into consideration in most constitutive models. Therefore, the current models do not reflect realistic properties of concrete at elevated temperatures.

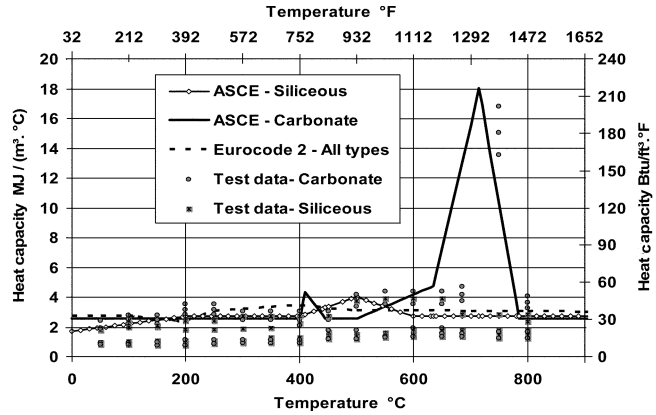

Fig. 2—Variation in heat capacity of normal-strength concrete as predicted by different models and test data.

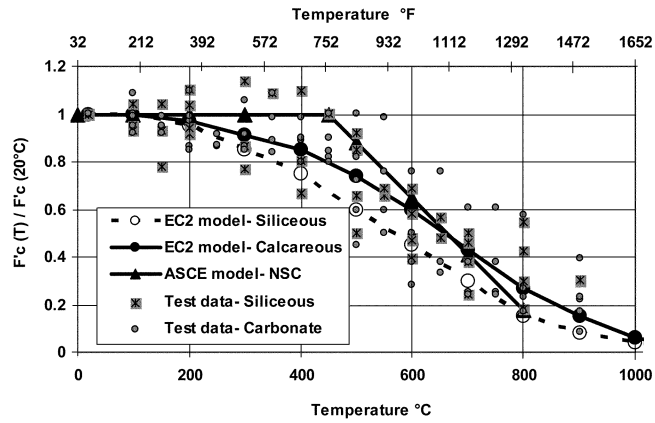

Fig. 3—Variation of compressive strength with temperature for normal-strength concrete.

Mechanical properties—test data

High-temperature mechanical properties of concrete have been studied more widely than those of thermal properties. The property tests were carried out in two ways; namely, measuring the response during exposure to elevated temperatures[22,29-40] and after exposure to elevated temperatures.[41-49] There is a good amount of high-temperature test data on both NSC and HSC with different types of aggregates.

Figures 3 and 4 show variation of concrete compressive strength with temperature for NSC and HSC, respectively. The average value of the standard deviation of the compiled test data for the compressive strength ratio ($f'_c(T)/f'_c$ (20 °C)) of the NSC was 0.1 and 0.14 for siliceous and carbonate aggregate concrete, respectively, whereas for HSC, the average value of the standard deviation was 0.13. Figure 3 shows large and uniform variation of the compiled test data for NSC, as seen in the shaded area. Figure 4, however, shows a larger variation in the compressive strength with temperature for HSC in the range 200 to 500 °C (392 to 932 °F), and less variation above 500 °C (932 °F). This is partly because fewer test data points were reported for temperatures higher than 500 °C (932 °F), either due to spalling problems or test apparatus limitations. Such variations seem to be more uniform throughout the temperature range compared to NSC in Fig. 3. This can be attributed to the higher number of test data points reported for NSC in the literature with lower tendency of NSC to spall under fire. It can be seen that the variations in the mechanical properties of concrete at high

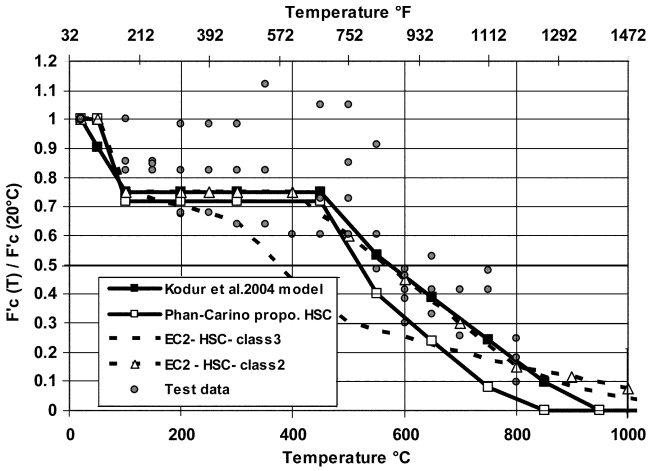

Fig. 4—Variation of compressive strength with temperature for high-strength concrete.

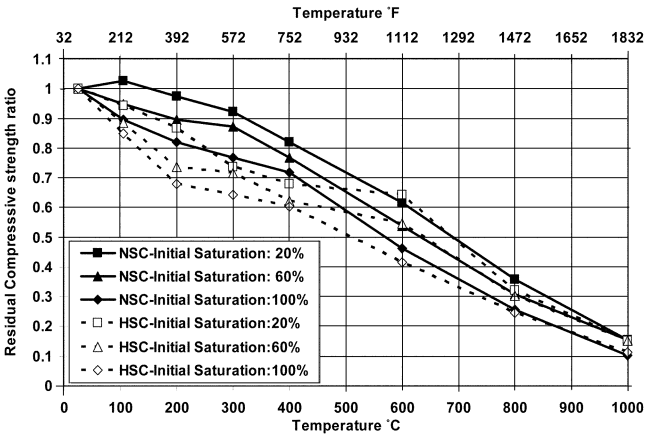

Fig. 5—Effect of initial saturation on residual compressive strength of concrete.[41]

temperatures are quite high. These variations from different tests can further be attributed to using different heating or loading rates, specimen curing and condition at testing, and the use of admixtures with different quantities.

Among the factors that directly affect the mechanical test results at high temperatures are the initial curing, moisture content at the time of testing, and the use of chemicals (admixtures, silica fume, and fly ash) to improve workability and strength. These factors were not addressed in the literature, and there are no test data that show the influence of these factors on the high-temperature concrete properties except that most HSC contains silica fume and/or fly ash. Residual raw test data from Lau and Anson[41] were used to study the effect of such factors on the residual compressive strength of concrete. Figures 5 and 6 show the influence of initial saturation and the use of chemicals on residual test results of concrete compressive strength as a function of temperature. In these tests from Lau and Anson,[41] the initial degree of saturation was defined as the ratio of the evaporated water to the dry weight of a 28-day cured concrete specimen. In residual tests, the concrete specimen is heated until it reaches a desired temperature, then the temperature is held constant for certain time duration, and then the specimen is left to cool down. The rates of heating and cooling and the hold time have a significant influence on the concrete mechanical properties.

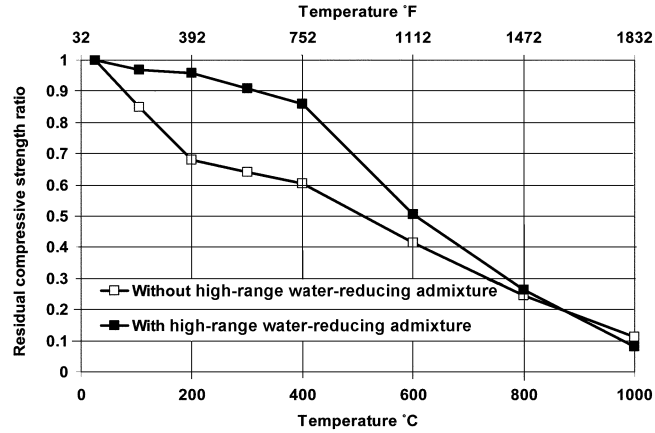

Fig. 6—Effect of naphthalene-based high-range water-reducing admixtures on residual compressive strength of high-strength concrete.[41]

Figure 5 shows that the initial degree of saturation (due to curing) for concrete specimens can make a difference up to 33% for NSC and 55% for HSC in the residual compressive strength. In addition, the effect of adding high-range water-reducing admixtures (1.5% by cement weight only) to the concrete mixture can make a difference in the reported results up to 40% in the residual compressive strength of HSC, as shown in Fig. 6. To explain this effect, it can be argued that the presence of such chemicals can affect the rate of hydration at elevated temperatures and, hence, lead to an apparent increase in the compressive strength, as shown in Fig 6. Further research, however, is needed to study the effect of such chemicals on the high-temperature properties of concrete.

Another main reasoning for the significant variation in the concrete high-temperature properties is the lack of standardized test methods for carrying out property tests. Further, much of the data reported in literature does not provide full details on the type of aggregate, moisture content, heating, or loading rates under which the tests were conducted. Therefore, the constitutive models developed from such test data are not fully reliable.

Mechanical properties—constitutive models

Based on some experimental studies, constitutive relationships for the mechanical properties were developed in the Eurocode[24] and ASCE *Manual* No. 78.[12] The most known constitutive models are shown in Table A.1 in the Appendix, where the ASCE model is developed for NSC, the Eurocode model is developed for both NSC and HSC, and Kodur et al.[28] model is developed for HSC. These relations give the rate of degradation of concrete as a function of temperature only, and without any consideration to other significant parameters such as rate of loading, heating, and material composition. Figures 3 and 4 show variation of concrete strength with temperature for NSC and HSC, respectively, according to the models in Eurocode 2,[24] the ASCE manual,[12] and Kodur et al.[28] and from test data reported in literature.

It can be seen from Fig. 3 and 4 that there is a large variation in the reported test data and also from the different models present in ASCE, Eurocode, and other standards. For example, the reduction in NSC compressive strength with temperature based on the ASCE model[12] seems to be closer to the upper bound of the reported experimental data. The Eurocode 2[24]

curve for the strength reduction of siliceous NSC, however, is closer to the lower bound of the reported test results.

In 2003, Phan and Carino[2] reviewed the limits at which the current high-temperature constitutive models for the reduction in compressive strength of HSC are conservative. They proposed a lower bound model for the reduction in the compressive strength of HSC and this is shown in Fig. 4. However, even this model is not conservative throughout the temperature range of 200 °C to 450 °C (392 to 842 °F), as can be seen from Fig. 4. Overall, the model proposed in Eurocode 2[24] for Class 3 of HSC (the definition of HSC classes in the Eurocode is given in the Appendix) gives conservative values for concrete compressive strength reduction with temperature. The aforementioned review clearly illustrates the large variation in the available test data and relationships in codes and standards for high-temperature strength properties. Consequently, performance-based design using these models is expected to produce diverse and inaccurate fire resistance predictions.

Current treatment in codes and standards

The aforementioned review clearly illustrates that the lack of reliable constitutive relationships for high-temperature properties of concrete in codes and standards makes it difficult to undertake performance-based fire safety design. This is especially true with ACI and ASCE standards, which do not provide any high-temperature properties or specifications for high-temperature constitutive relationships for new types of concrete.[2]

There are clear differences in the constitutive relationships for high-temperature properties of concrete that are widely used in European and American standards. These constitutive relationships for NSC and HSC are given in the Eurocode,[24] whereas the ASCE *Manual* No. 78[12] contains relationships for NSC only. The constitutive relationships for high-temperature properties of concrete specified in the Eurocode and the ASCE manual are summarized in Table A.1 in the Appendix. In addition to these constitutive models, Kodur et al.[28] proposed constitutive relations for HSC, which are an extension to ASCE[12] relations for NSC. These relations for HSC are also included in Table A.1.

A major difference between the European and the American high-temperature material models for concrete is the effect of aggregate type on the concrete properties. The Eurocode does not specifically account for the effect of aggregate type on the thermal capacity of concrete at high temperatures. In the Eurocode, properties such as specific heat, density changes, and, hence, heat capacity, are considered to be the same for all aggregate types used in concrete. For the thermal conductivity of concrete, the Eurocode proposes upper and lower limits without indicating which limit to use for a given aggregate type in concrete.[24]

EFFECT OF VARIATIONS IN CONSTITUTIVE MODELS ON FIRE RESISTANCE

To illustrate the effect of varying constitutive relationships on fire resistance, numerical studies are carried out on a set of RC columns. Eleven RC columns made of different types of concrete were analyzed by changing the concrete constitutive models. The results from the analysis are then compared to the test data for these RC columns. More details about the columns and the analysis are given in the following sections.

Table 1—Summary of parameters and results from analysis of the reinforced concrete columns

Column no.	Aggregate type	f'_c, MPa (psi)	Load, kN (kips)	Fire resistance, minutes		
				Test	Analytical ASCE model	Analytical EN1992-1-2
RC1	Siliceous	31.0 (4995)	1333 (300)	170	197	179
RC2	Siliceous	34.8 (5047)	800 (180)	218	250	259
RC3	Siliceous	31.7 (4598)	711 (160)	220	264	282
RC4	Siliceous	35.3 (5120)	1067 (240)	208	224	218
RC5	Siliceous	33.5 (4859)	1778 (400)	146	139	119
RC6	Siliceous	36.8 (5337)	1333 (300)	187	202	186
RC7	Siliceous	53.2 (7716)	1178 (265)	227	254	264
RC8	Siliceous	51.4 (7455)	1067 (240)	234	257	270
RC9	Carbonate	33.6 (4873)	800 (180)	510	466	343
RC10	Carbonate	33.5 (4859)	1067 (240)	366	358	278
RC11	Carbonate	34.2 (4960)	1778 (400)	216	250	164

Columns for analysis

The RC columns selected for numerical studies are taken from the reported fire tests carried out by the National Research Council Canada (NRCC) and Portland Cement Association (PCA).[50] Eleven RC columns that failed by concrete crushing in fire resistance tests were selected for the analysis. A summary of the design parameters and properties for the selected RC columns is presented in Table 1. In addition, Table 1 shows the measured (in fire tests) and predicted (from current analysis) fire resistances of the selected RC columns.

All of the columns had square cross sections of 305 x 305 mm (12.5 x12.5 in) size and were 3810 mm (12.5 ft) long. The columns had four bars of 25.4 mm (No. 8) diameter at the corners with a yield strength of 444 MPa (65 ksi), and the effective concrete cover was 48 mm (1.88 in.) to the center of main reinforcing bars. The columns were exposed to the ASTM E119 standard fire[51] from four sides and were made of varying concrete strengths, as shown in Table 1.

Method of analysis

The fire response of the aforementioned selected columns was simulated by exposing them to a standard ASTM E119 fire. The analysis was carried out using the concrete and reinforcing steel constitutive relationships (both thermal and mechanical high-temperature properties) given in the ASCE *Manual* No. 78[12] and Eurocode.[24] A computer program written in FORTRAN was used to carry out the numerical analysis for the selected columns. The analysis was carried out by incrementing time steps until the failure was attained in the columns. At each time step, a set of calculations was performed to derive temperature and strength of the column. In the computer program, the numerical procedure was carried out as follows.

Fire temperature calculation—The first step in the analysis was to compute the fire temperatures at every time step. The fire temperature was evaluated assuming the fire scenario to be ASTM E119[51] standard fire exposure. The time-temperature relationship for the ASTM E119 standard fire can be approximated by the following equation

$$T_f = T_0 + 750(1 - \exp(-3.79553\sqrt{t_h})) + 170.41\sqrt{t_h} \quad (1)$$

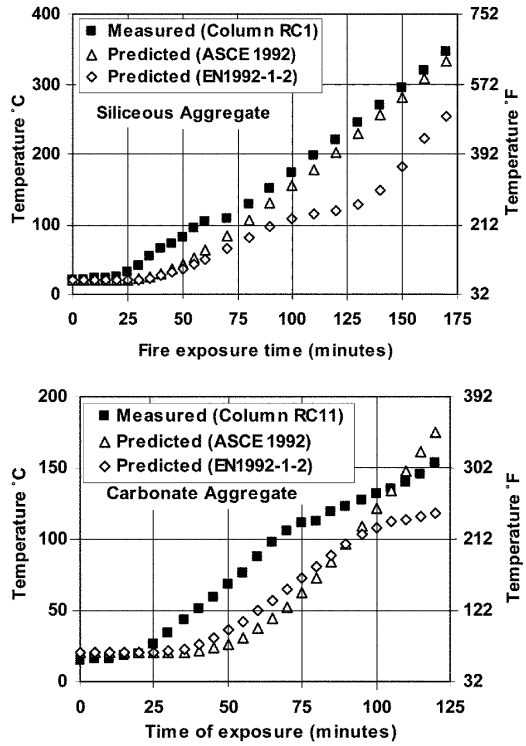

Fig. 7—Temperature as a function of fire exposure time at the center of Columns RC1 and RC11.

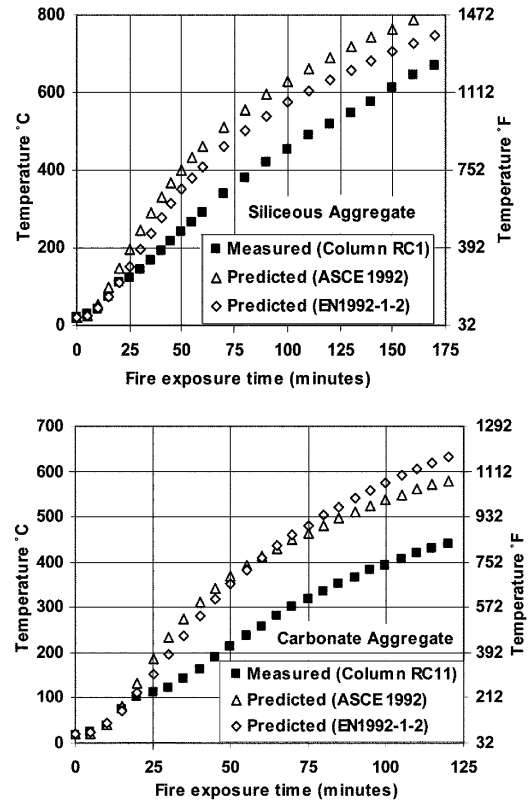

Fig. 8—Temperature in the reinforcing bar as a function of fire exposure time of Columns RC1 and RC11.

where t_h is the time (hours); T_0 is the initial temperature (°C); and T_f is the fire temperature (°C).

Heat transfer analysis—The next step was to compute the temperature distribution within the cross section of the column at every time step. The thermal analysis was performed using a finite element technique. The cross section was meshed into rectangular elements, and the temperature rise at every element inside the section was evaluated by establishing heat balance for every element at every time step. The temperature-dependent thermal properties specified in ASCE[12] and Eurocode 2[24] were implemented in the analysis.

Strength analysis—The cross-sectional temperatures generated from thermal analysis were used as input data to calculate the column strength at every time step. A sectional analysis approach was used to derive the strength of the column at various time steps. The strength of the column was evaluated by computing the maximum axial force a column section can sustain before the occurrence of crushing. In the analysis, various strain components (mechanical, transient, thermal, and creep strains) were considered. Axial deformations at every time step were obtained by integrating, over the column length, the axial strain corresponding to the applied axial load on the column.

The temperature and strength capacities were used to derive failure time of the columns. The failure time is said to occur when the strength of the column at any time step falls below the applied axial load on the columns. The results of the thermal and mechanical analyses for the two types of concrete are discussed in the following sections.

Thermal analysis predictions

Typical prediction from the thermal analysis is illustrated in Fig. 7, which shows the temperature rise at the center of the concrete columns (RC1 and RC11) as a function of fire exposure time. Both Eurocode and ASCE models underestimate the temperature as compared to measured values obtained from tests. For both concrete types, the Eurocode predictions diverge after 140 °C (284 °F), whereas the ASCE model predictions compare well with the test data. This could be attributed to the fact that the Eurocode formulas for the thermal conductivity of concrete (as seen in Fig. 1) are based on lower bound values of the test data, and hence the resulting temperature predictions are lower than the measured temperatures. Further, in the temperature range of 20 °C to 450 °C (68 to 842 °F), the Eurocode formula generally predicts higher values of heat capacity for concrete than does the ASCE heat capacity model, as can be seen from Fig. 2. This explains the tendency of the Eurocode to underestimate the temperature rise in concrete in either type of concretes.

Figure 8 shows a comparison of predicted reinforcing bar temperatures with the measured temperatures for Columns RC1 and RC11. Both Eurocode and ASCE models predict higher reinforcing bar temperatures than the measured ones, as shown in Fig. 8. In addition to the fact that most of the heat flux is transferred through conduction, moisture migration to the center of the column section enhances the convective heat transfer within the column cross section.[52] This leads to actually higher temperatures than what the models predict. This can be partially attributed to the fact that the evaporation of water in the heated concrete regions around the reinforcing bars absorbs a significant amount of incoming energy and, hence, results in a lower reinforcing bar temperature than that predicted by the models. Also, the thermal conductivity in the models may be lower than that of the concrete in the fire tests. The lower conductivity used in the model leads to low internal temperatures and somewhat high temperatures at the location of steel reinforcing bars. It should be noted

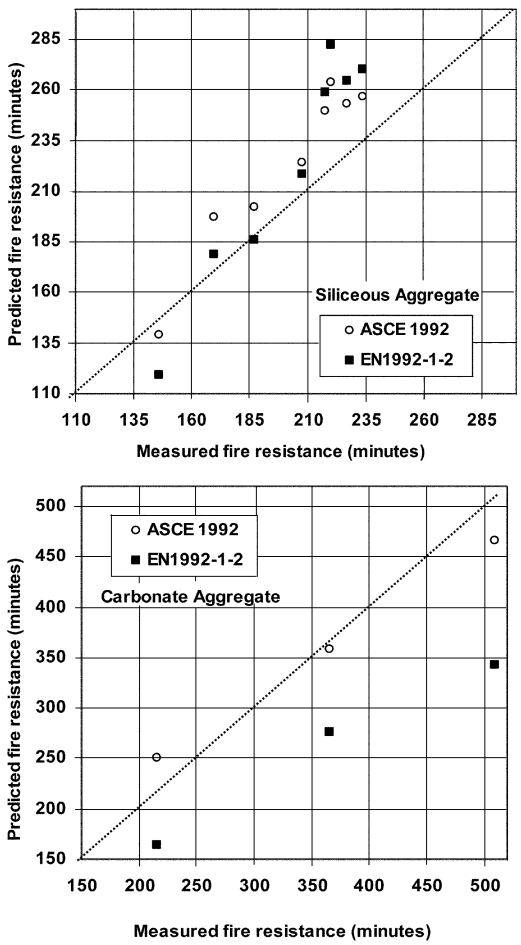

Fig. 9—Measured versus predicted failure times of siliceous and carbonate aggregate concrete columns.

that moisture migration at elevated temperatures inside concrete is a complex process and depends on many material properties such as porosity and permeability of concrete that are not well known at elevated temperatures.

Mechanical analysis predictions

Figures 9 through 11 show the result of the mechanical analysis for the RC columns. The fire resistance values (failure times) for the RC columns as predicted by ASCE and Eurocode are compared with measured failure times from tests in Fig. 9. The predictions from both Eurocode and ASCE generally overestimate the failure time in columns made of siliceous aggregate concrete, as can be seen in Fig. 9. This is because both constitutive models underestimate the temperature rise in the concrete and, hence, apparently overestimate its strength under fire exposure. Thus, the fire resistance predicted based on both material models may not be conservative. For columns made of carbonate aggregate concrete, however, the predictions are conservative, as can be seen in Fig. 9. This is because when comparing cross-sectional temperatures of columns with the two types of aggregates, the differences between the measured and the predicted temperatures at high temperatures (above 100 °C [212 °F]) are smaller in the concrete and higher in the reinforcing bar for columns made with carbonate aggregate concrete. Overall, predicting fire resistance using ASCE material properties is closer to that obtained from tests. Thus, the use of ASCE material properties may lead to better predictions.

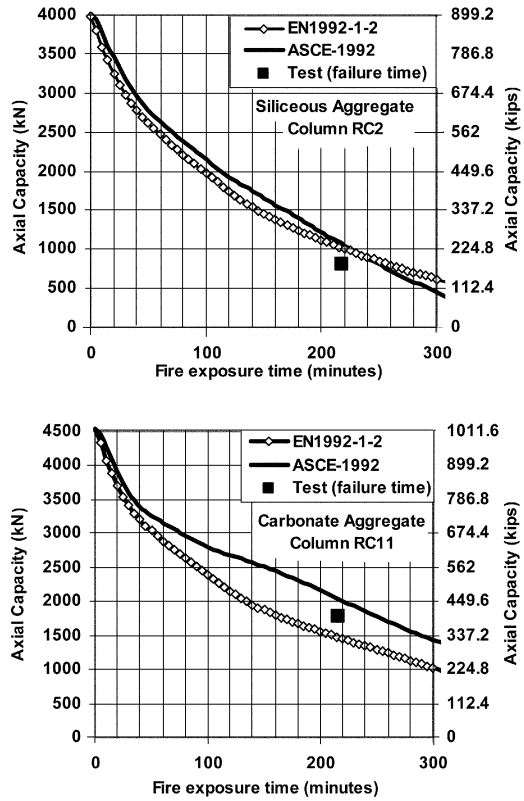

Fig. 10—Strength degradation as a function of fire exposure time for Columns RC2 and RC11.

Figure 10 shows the degradation in axial capacities as a function of fire exposure time for Columns RC2 and RC11 based on Eurocode[24] and ASCE[12] material models. It can be seen that the differences between the two models for carbonate concrete (Column RC11) are more evident than for the siliceous aggregate (Column RC2). This is mainly due to the fact that the Eurocode does not account for the high thermal capacity of carbonate aggregate concrete (in the temperature range of 650 to 800°C [1202 to 1472 °F]), which results from endothermic reaction occurring due to phase change (refer to Fig. 2). However, the ASCE material model accounts for this increased heat capacity.

The predicted axial deformations based on Eurocode and ASCE material constitutive models are compared to the measured deformations for Columns RC2 and RC11 in Figure 11. It can be seen from the figure that the predictions based on ASCE material model match well with the test data when compared to the Eurocode material model. This can be attributed to the fact that the ASCE material model accounts for the increased heat capacity in the temperature range of 650 to 800 °C (1202 to 1472 °F).

Summary

The high variation in the concrete material models has a significant influence on fire resistance predictions for RC columns. The results show that using the ASCE or the Eurocode material models for predicting the fire resistance of siliceous concrete columns may not be conservative in all cases. For carbonate concrete columns, however, predictions based on the two models could be conservative with the predictions based on the ASCE model being closer to the experimental results. It is also shown that both models

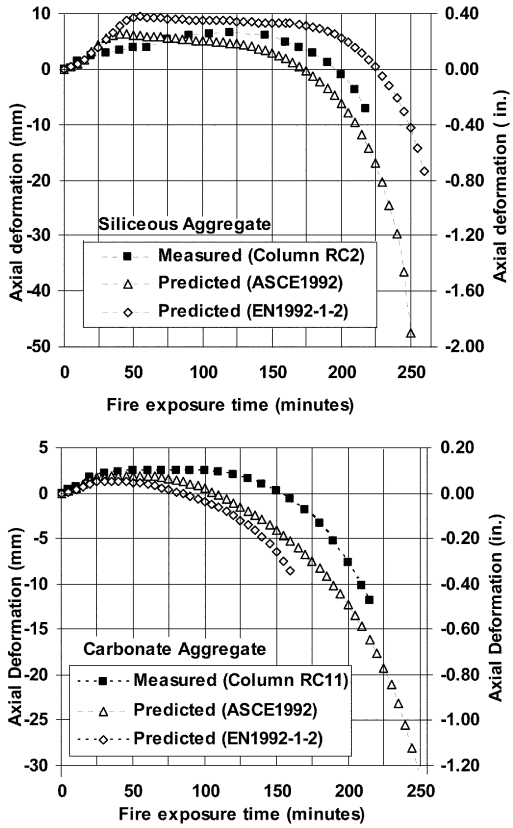

Fig. 11—Axial deformation as a function of fire exposure time for Columns RC2 and RC11.

underestimate the temperature rise at the center of concrete column, but overestimate the temperature rise in the reinforcing bar. In addition, the deflection predictions based on the ASCE material model are closer to the test results than those computed based on the Eurocode.

DESIGN RECOMMENDATIONS

High-temperature properties of concrete materials have a significant influence on the fire performance of RC structural members. There is significant variability in the reported test data on the high-temperature properties of concrete. Consequently, there are large variations in the current constitutive models for thermal and mechanical properties of concrete.

The Eurocode and the ASCE models tend to overestimate the fire resistance of siliceous aggregate RC columns. However, the ASCE model provides slightly better estimation of the temperature, strength, and deflection of the RC columns when compared to Eurocode predictions. The Eurocode concrete material model tends to underestimate the fire resistance of short RC columns made of carbonate aggregate concrete. The ASCE model, however, provides slightly better fire resistance predictions. As for axial deformation of short RC columns, the ASCE model predictions were more in agreement with the test data than the Eurocode predictions. Thus, instead of better constitutive models available at the present time, the ASCE high-temperature concrete material models can be used for performance-based fire safety design.

CONCLUSIONS

The following conclusions can be drawn concerning the high-temperature properties of concrete:

1. There are significant variability and discrepancies in the reported high-temperature material properties of concrete. Also, the constitutive property models in the Eurocode and ASCE *Manual* No. 78 have many limitations and hence do not provide accurate predictions for the fire resistance;

2. The fire resistance predictions based on the constitutive models specified in the ASCE manual and the Eurocode may not be conservative for siliceous aggregate concrete columns;

3. The fire resistance predictions based on the constitutive models specified in the ASCE manual and the Eurocode are generally conservative for carbonate aggregate concrete columns; and

4. Overall, ASCE material property models seem to give better predictions when compared to the Eurocode for the axial deformation and strength of short RC columns under fire.

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APPENDIX

Table A.1—Constitutive relationships for high-temperature properties of concrete

	ASCE manual 1992 (NSC)[12]	Kodur et al. 2004 (HSC)[28]	EN1992-1-2:2004 (NSC and HSC)[24]
Stress-strain relationships	$\sigma_c = \begin{cases} f'_{c,T}\left[1-\left(\frac{\varepsilon-\varepsilon_{max,T}}{\varepsilon_{max,T}}\right)^2\right], & \varepsilon \le \varepsilon_{max,T} \\ f'_{c,T}\left[1-\left(\frac{\varepsilon_{max,T}-\varepsilon}{3\varepsilon_{max,T}}\right)^2\right], & \varepsilon > \varepsilon_{max,T} \end{cases}$ $f'_{c,T} = \begin{cases} f'_c & , 20\ °C \le T \le 450\ °C \\ f'_c\left[2.011-2.353\left(\frac{T-20}{1000}\right)\right], & 450\ °C < T \le 874\ °C \\ 0 & , 874\ °C < T \end{cases}$ $\varepsilon_{max,T} = 0.025 + (6.0T + 0.04T^2) \times 10^{-6}$	$\sigma_c = \begin{cases} f'_{c,T}\left[1-\left(\frac{\varepsilon_{max,T}-\varepsilon}{\varepsilon_{max,T}}\right)^H\right], & \varepsilon \le \varepsilon_{max,T} \\ f'_{c,T}\left[1-\left(\frac{30(\varepsilon-\varepsilon_{max,T})}{(130-f'_c)\varepsilon_{max,T}}\right)^2\right], & \varepsilon > \varepsilon_{max,T} \end{cases}$ $f'_{c,T} = \begin{cases} f'_c\,[1.0-0.003125(T-20)] & , T < 100\ °C \\ 0.75f'_c & , 100\ °C \le T \le 400\ °C \\ f'_c\,[1.33-0.00145T] & , 400\ °C < T \end{cases}$ $\varepsilon_{max,T} = 0.0018 + (6.7f'_c + 6.0T + 0.03T^2) \times 10^{-6}$ $H = 2.28 - 0.012f'_c$	$\sigma_c = \frac{3\varepsilon f'_{c,T}}{\varepsilon_{c1,T}\left(2+\left(\frac{\varepsilon}{\varepsilon_{c1,T}}\right)^3\right)}, \varepsilon \le \varepsilon_{cu1,T}$ For $\varepsilon_{c1,T} < \varepsilon \le \varepsilon_{cu1,T}$, the Eurocode permits the use of linear as well as nonlinear descending branch in the numerical analysis. For the parameters in this equation, refer to Table A.2.
Thermal capacity	*Siliceous aggregate concrete:* $\rho c = \begin{cases} 0.005T + 1.7 & 20\ °C \le T \le 200\ °C \\ 2.7 & 200\ °C < T \le 400\ °C \\ 0.013T - 2.5 & 400\ °C < T \le 500\ °C \\ 10.5 - 0.013T & 500\ °C < T \le 600\ °C \\ 2.7 & 600\ °C < T \end{cases}$ *Carbonate aggregate concrete:* $\rho c = \begin{cases} 2.566 & 20\ °C \le T \le 400\ °C \\ 0.1765T - 68.034 & 400\ °C < T \le 410\ °C \\ 25.00671 - 0.05043T & 410\ °C < T \le 445\ °C \\ 2.566 & 445\ °C < T \le 500\ °C \\ 0.01603T - 5.44881 & 500\ °C < T \le 635\ °C \\ 0.16635T - 100.90225 & 635\ °C < T \le 715\ °C \\ 176.07343 - 0.22130T & 715\ °C < T \le 785\ °C \\ 2.566 & 785\ °C < T \end{cases}$	*Siliceous aggregate concrete:* $\rho c = \begin{cases} 0.005T + 1.7 & 20\ °C \le T \le 200\ °C \\ 2.7 & 200\ °C < T \le 400\ °C \\ 0.013T - 2.5 & 400\ °C < T \le 500\ °C \\ 10.5 - 0.013T & 500\ °C < T \le 600\ °C \\ 2.7 & 600\ °C < T \le 635\ °C \end{cases}$ *Carbonate aggregate concrete:* $\rho c = \begin{cases} 2.45 & 20\ °C \le T \le 400\ °C \\ 0.026T - 12.85 & 400\ °C < T \le 475\ °C \\ 0.0143T - 6.295 & 475\ °C < T \le 650\ °C \\ 0.1894T - 120.11 & 650\ °C < T \le 735\ °C \\ -0.263T - 212.4 & 735\ °C < T \le 800\ °C \\ 2 & 800\ °C < T \le 1000\ °C \end{cases}$	*Specific heat (J/kg·°C):* $c = 900$, for $20\ °C \le T \le 100\ °C$ $c = 900 + (T - 100)$, for $100\ °C < T \le 200\ °C$ $c = 900 + (T - 200)/2$, for $200\ °C < T \le 400\ °C$ $c = 1100$, for $400\ °C < T \le 1200\ °C$ *Density change (kg/m³):* $\rho = \rho(20\ °C)$ = reference density for $20\ °C \le T \le 115\ °C$ $\rho = \rho(20\ °C)(1 - 0.02(T - 115)/85)$ for $115\ °C < T \le 200\ °C$ $\rho = \rho(20\ °C)(0.98 - 0.03(T - 200)/200)$ for $200\ °C < T \le 400\ °C$ $\rho = \rho(20\ °C)(0.95 - 0.07(T - 400)/800)$ for $400°C < T \le 1200\ °C$ *Thermal capacity* $= \rho \times c$
Thermal conductivity	*Siliceous aggregate concrete:* $k_c = \begin{cases} -0.000625T + 1.5 & 20\ °C \le T \le 800\ °C \\ 1.0 & 800\ °C < T \end{cases}$ *Carbonate aggregate concrete:* $k_c = \begin{cases} 1.355 & 20\ °C \le T \le 293\ °C \\ -0.001241T + 1.7162 & 293\ °C < T \end{cases}$	*Siliceous aggregate concrete:* $k_c = 0.85(2 - 0.0011T)$ $20\ °C < T \le 1000\ °C$ *Carbonate aggregate concrete:* $k_c = \begin{cases} 0.85(2 - 0.0013T) & 20\ °C \le T \le 300\ °C \\ 0.85(2.21 - 0.002T) & 300\ °C < T \end{cases}$	All types: *Upper limit:* $k_c = 2 - 0.2451(T/100) + 0.0107(T/100)^2$ for $20\ °C \le T \le 1200\ °C$ *Lower limit:* $k_c = 1.36 - 0.136(T/100) + 0.0057(T/100)^2$ for $20\ °C \le T \le 1200\ °C$
Thermal strain	All types: $\varepsilon_{th} = [0.004(T^2 - 400) + 6(T - 20)] \times 10^{-6}$	All types: $\varepsilon_{th} = [0.004(T^2 - 400) + 6(T - 20)] \times 10^{-6}$	*Siliceous aggregate:* $\varepsilon_{th} = -1.8 \times 10^{-4} + 9 \times 10^{-6}T + 2.3 \times 10^{-11}T^3$ for $20\ °C \le T \le 700\ °C$ $\varepsilon_{th} = 14 \times 10^{-3}$ for $700\ °C < T \le 1200\ °C$ *Siliceous aggregate:* $\varepsilon_{th} = -1.2 \times 10^{-4} + 6 \times 10^{-6}T + 1.4 \times 10^{-11}T^3$ for $20\ °C \le T \le 805\ °C$ $\varepsilon_{th} = 12 \times 10^{-3}$ for $805\ °C < T \le 1200\ °C$

Table A.2—Values for the main parameter of the stress-strain relationships of NSC and HSC at elevated temperatures[24]

		Normal-strength concrete						High-strength concrete		
		Siliceous aggregate			Calcareous aggregate			$f_{c,T}'/f_c'$ (20 °C)		
Temperature, °F	Temperature, °C	$f_{c,T}'/f_c'$ (20 °C)	$\varepsilon_{c1,T}$	$\varepsilon_{cu1,T}$	$f_{c,T}'/f_c'$ (20 °C)	$\varepsilon_{c1,T}$	$\varepsilon_{cu1,T}$	Class 1	Class 2	Class 3
68	20	1	0.0025	0.02	1	0.0025	0.02	1	1	1
212	100	1	0.004	0.0225	1	0.004	0.023	0.9	0.75	0.75
392	200	0.95	0.0055	0.025	0.97	0.0055	0.025	0.9	0.75	0.70
572	300	0.85	0.007	0.0275	0.91	0.007	0.028	0.85	0.75	0.65
752	400	0.75	0.01	0.03	0.85	0.01	0.03	0.75	0.75	0.45
932	500	0.6	0.015	0.0325	0.74	0.015	0.033	0.60	0.60	0.30
1112	600	0.45	0.025	0.035	0.6	0.025	0.035	0.45	0.45	0.25
1292	700	0.3	0.025	0.0375	0.43	0.025	0.038	0.30	0.30	0.20
1472	800	0.15	0.025	0.04	0.27	0.025	0.04	0.15	0.15	0.15
1652	900	0.08	0.025	0.0425	0.15	0.025	0.043	0.08	0.113	0.08
1832	1000	0.04	0.025	0.045	0.06	0.025	0.045	0.04	0.075	0.04
2012	1100	0.01	0.025	0.0475	0.02	0.025	0.048	0.01	0.038	0.01
2192	1200	0	—	—	0	—	—	0	0	0

Notes: The Eurocode classifies HSC into three classes, depending on its compressive strength, namely:
· Class 1 for concrete with compressive strength between C55/67 and C60/75;
· Class 2 for concrete with compressive strength between C70/85 and C80/95; and
· Class 3 for concrete with compressive strength higher than C90/150.
The strength notation of C55/67 refers to a concrete grade with characteristic cylinder and cube strengths of 55 and 67 N/mm^2, respectively.
Where the actual characteristic strength of concrete is likely to be of a higher class than that specified in design, the relative reduction in strength for the higher class should be used for fire design.

NOTATION

c = concrete specific heat, J/kg·°C
f_c' = 28-day concrete compressive strength at room temperature, MPa
$f_{c,T}'$ = concrete compressive strength at temperature T, MPa
k_c = concrete thermal conductivity, W/m·°C
T = concrete temperature, °C
ε = concrete mechanical strain
ε_{th} = concrete thermal strain
$\varepsilon_{c1,T}$ = concrete strain at maximum stress at temperature T (Eurocode model)
$\varepsilon_{cu1,T}$ = concrete ultimate strain at temperature T (Eurocode model)
$\varepsilon_{max,T}$ = concrete strain at maximum stress at temperature T (ASCE 1992 and Kodur et al. 2004 models)
$\rho(T)$ = density of concrete temperature T, kg/m^3
ρ(20 °C) = density of concrete at room temperature, kg/m^3
σ_c = stress in concrete, MPa

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