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Review Article

Properties of Concrete at Elevated Temperatures

Venkatesh Kodur

Department of Civil and Environmental Engineering, Michigan State University, East Lansing, MI 48824, USA

Correspondence should be addressed to Venkatesh Kodur; kodur@egr.msu.edu

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Fire response of concrete structural members is dependent on the thermal, mechanical, and deformation properties of concrete. These properties vary significantly with temperature and also depend on the composition and characteristics of concrete batch mix as well as heating rate and other environmental conditions. In this chapter, the key characteristics of concrete are outlined. The various properties that influence fire resistance performance, together with the role of these properties on fire resistance, are discussed. The variation of thermal, mechanical, deformation, and spalling properties with temperature for different types of concrete are presented.

1. Introduction

Concrete is widely used as a primary structural material in construction due to numerous advantages, such as strength, durability, ease of fabrication, and noncombustibility properties, it possesses over other construction materials. Concrete structural members when used in buildings have to satisfy appropriate fire safety requirements specified in building codes [1–4]. This is because fire represents one of the most severe environmental conditions to which structures may be subjected; therefore, provision of appropriate fire safety measures for structural members is an important aspect of building design.

Fire safety measures to structural members are measured in terms of fire resistance which is the duration during which a structural member exhibits resistance with respect to structural integrity, stability, and temperature transmission [5, 6]. Concrete generally provides the best fire resistance properties of any building material [7]. This excellent fire resistance is due to concrete's constituent materials (i.e., cement and aggregates) which, when chemically combined, form a material that is essentially inert and has low thermal conductivity, high heat capacity, and slower strength degradation with temperature. It is this slow rate of heat transfer and strength loss that enables concrete to act as an effective fire shield not only between adjacent spaces but also to protect itself from fire damage.

The behaviour of a concrete structural member exposed to fire is dependent, in part, on thermal, mechanical, and deformation properties of concrete of which the member is composed. Similar to other materials the thermophysical, mechanical, and deformation properties of concrete change substantially within the temperature range associated with building fires. These properties vary as a function of temperature and depend on the composition and characteristics of concrete. The strength of concrete has significant influence on its properties at both room and high temperatures. The properties of high strength concrete (HSC) vary differently with temperature than those of normal strength concrete (NSC). This variation is more pronounced for mechanical properties, which are affected by strength, moisture content, density, heating rate, amount of silica fume, and porosity.

In practice, fire resistance of structural members used to be evaluated mainly through standard fire tests [8]. In recent years, however, the use of numerical methods for the calculation of the fire resistance of structural members is gaining acceptance because these calculation methods are far less costly and time consuming [9]. When a structural member is subjected to a defined temperature-time exposure during a fire, this exposure will cause a predictable temperature distribution in the member. Increased temperatures cause deformations and property changes in the constitutive materials of a structural member. With knowledge of deformations and property changes, the usual methods of structural mechanics can be applied to predict the fire resistance performance of a structural member. The availability of material properties at an elevated temperature permits a mathematical approach for predicting fire resistance of structural members [10, 11].

Clearly, the generic information available on properties of concrete at room temperature is seldom applicable in fire resistance design [12]. It is imperative, therefore, that the fire safety practitioner knows how to extend, based on a priori considerations, the utility of the scanty property data that can be gathered from the technical literature. Also, knowledge of unique characteristics, such as fire induced spalling in concrete, is critical to determine the fire performance of concrete structural members.

2. Properties Influencing Fire Resistance

2.1. General. The fire response of reinforced concrete (RC) members is influenced by the characteristics of constituent materials, namely, concrete and reinforcing steel. These include (a) thermal properties, (b) mechanical properties, (c) deformation properties, and (d) material specific characteristics such as spalling in concrete. The thermal properties determine the extent of heat transfer to the structural member, whereas the mechanical properties of constituent materials determine the extent of strength loss and stiffness deterioration of the member. The deformation properties, in conjunction with mechanical properties, determine the extent of deformations and strains in the structural member. In addition, fire induced spalling of concrete can play a significant role in the fire performance of RC members [13]. All these properties vary as a function of temperature and depend on the composition and characteristics of concrete as well as those of the reinforcing steel [12]. The temperature induced variation in properties in concrete is much more complex than that in reinforcing steel due to moisture migration as well as significant variation of ingredients in different types of concrete. Thus, the primary focus of this chapter is on the effect of temperature on properties of concrete. The effect of temperature on properties of steel reinforcement can be found elsewhere [4, 12].

Concrete is available in various forms and it is often grouped under different categories based on weight (as normal weight and light weight concrete), strength (as normal strength, high strength, and ultrahigh strength concrete), presence of fibers (as plain and fiber-reinforced concrete), and performance (as conventional and high performance concrete). Fire safety practitioners further subdivide normal-weight concretes into silicate (siliceous) and carbonate (limestone) aggregate concrete, according to the composition of the principal aggregate. Also, when a small amount of discontinuous fibers (steel or polypropylene) is added to a concrete batch mix to improve performance, this concrete is referred to as fiber-reinforced concrete (FRC). In this section, the various properties of concrete are mainly discussed for conventional concrete. The effect of strength, weight, and fibers on properties of concrete at elevated temperatures is highlighted.

Traditionally, the compressive strength of concrete used to be around 20 to 50 MPa, which is classified as normal-strength concrete (NSC). In recent years, concrete with a compressive strength in the range of 50 to 120 MPa has become widely available and is referred to as high-strength concrete (HSC). When compressive strength exceeds 120 MPa, it is often referred to as ultrahigh performance concrete (UHP). The strength of concrete degrades with temperature and the rate of strength degradation is highly influenced by the compressive strength of concrete.

2.2. Thermal Properties. The thermal properties that influence temperature rise and distribution in a concrete structural member are thermal conductivity, specific heat, thermal diffusivity, and mass loss.

Thermal conductivity is the property of a material to conduct heat. Concrete contains moisture in different forms, and the type and the amount of moisture have a significant influence on thermal conductivity. Thermal conductivity is usually measured by means of "steady state" or "transient" test methods [14]. Transient methods are preferred to measure thermal conductivity of moist concrete over steady-state methods [15–17], as physiochemical changes of concrete at higher temperatures cause intermittent direction of heat flow. On average, the thermal conductivity of conventional normal strength concrete, at room temperature, ranges between 1.4 and 3.6 W/m-°C [18].

Specific heat is the amount of heat per unit mass, required to change the temperature of a material by one degree and is often expressed in terms of thermal (heat) capacity which is the product of specific heat and density. Specific heat is highly influenced by moisture content, aggregate type, and density of concrete [19–21]. The variation of specific heat with temperature used to be determined through adiabatic calorimetry until 1980s. Since the 1980s, differential scanning calorimetry (DSC) has been the most commonly used technique for mapping the curve in a single temperature sweep at a desired rate of heating [22, 23]. Unfortunately, the accuracy of the DSC technique in determining the sensible heat contribution to the apparent specific heat may not be particularly good (sometimes it may be as low as ±20 percent). The rate of temperature rise in DSC tests is usually 5°C·min^{-1}. At higher heating rates, the peaks in the DSC curves tend to shift to higher temperatures and become sharper. For temperatures above 600°C, a high-temperature differential thermal analyzer (DTA) is also used to evaluate specific heat.

The thermal diffusivity of a material is defined as the ratio of thermal conductivity to the volumetric specific heat of the material [24]. It measures the rate of heat transfer from an exposed surface of a material to inner layers. The larger the diffusivity, the faster the temperature rise at a certain depth in the material [12]. Similar to thermal conductivity and specific heat, thermal diffusivity varies with temperature rise in the material. Thermal diffusivity, α, can be calculated using the relation

$$\alpha = \frac{k}{\rho c_p}, \tag{1}$$

where k is thermal conductivity, ρ is density, and c_p is specific heat of the material.

The density, in an oven-dry condition, is the mass of a unit volume of the material, comprising the solid itself and the air-filled pores. With increasing temperature, materials such as concrete that have high amount of moisture will experience loss of mass resulting from evaporation of moisture due to chemical reactions. Assuming that the material is isotropic with respect to its dilatometric behavior, its density (or mass) at any temperature can be calculated from thermogravimetric and dilatometric curves [24].

2.3. Mechanical Properties. The mechanical properties that determine the fire performance of RC members are compressive and tensile strength, modulus of elasticity, and stress-strain response of constituent materials at elevated temperatures.

Compressive strength of concrete at an elevated temperature is of primary interest in fire resistance design. Compressive strength of concrete at ambient temperature depends upon water-cement ratio, aggregate-paste interface transition zone, curing conditions, aggregated type and size, admixture types, and type of stress [25]. At high temperature, compressive strength is highly influenced by room temperature strength, rate of heating, and binders in batch mix (such as silica fume, fly ash, and slag). Unlike thermal properties at high temperature, the mechanical properties of concrete are well researched. The strength degradation in HSC is not consistent and there are significant variations in strength loss, as reported by various authors.

The tensile strength of concrete is much lower than compressive strength, due to ease with which cracks can propagate under tensile loads [26]. Concrete is weak in tension, and for NSC, tensile strength is only 10% of its compressive strength and for HSC tensile strength ratio is further reduced. Thus, tensile strength of concrete is often neglected in strength calculations at room and elevated temperatures. However, it is an important property, because cracking in concrete is generally due to tensile stresses and the structural damage of the member in tension is often generated by progression in microcracking [26]. Under fire conditions tensile strength of concrete can be even more crucial in cases where fire induced spalling occurs in a concrete structural member [27]. Tensile strength of concrete is dependent on almost same factors as compressive strength of concrete [28, 29].

Another property that influences fire resistance is the modulus of elasticity of concrete which decreases with temperature. At high temperature, disintegration of hydrated cement products and breakage of bonds in the microstructure of cement paste reduce elastic modulus and the extent of reduction depends on moisture loss, high temperature creep, and type of aggregate.

2.4. Deformation Properties. The deformation properties that determine the fire performance of reinforced concrete members are thermal expansion and creep of the concrete and reinforcement at elevated temperatures. In addition, transient strain that occurs at elevated temperatures in concrete can enhance deformations in fire exposed concrete structural members.

Thermal expansion characterizes the expansion (or shrinkage) of a material caused by heating and is defined as the expansion (shrinkage) of unit length of a material when the temperature of concrete is raised by one degree. The coefficient of thermal expansion is defined as the percentage change in length of a specimen per degree temperature rise. The expansion is considered to be positive when the material elongates and is considered negative (shrinkage) when it shortens. In general, the thermal expansion of a material is dependent on the temperature and is evaluated through the dilatometric curve, which is a record of the fractional change of a linear dimension of a solid at a steadily increasing or decreasing temperature [24]. Thermal expansion is an important property to predict thermal stresses that get introduced in a structural member under fire conditions. Thermal expansion of concrete is generally influenced by cement type, water content, aggregate type, temperature, and age [15, 30].

Creep, often referred to as creep strain, is defined as the time-dependent plastic deformation of the material. At normal stresses and ambient temperatures, deformations due to creep are 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 temperature, the stress level, and their duration [31]. The creep of concrete is due to the presence of water in its microstructure [32]. There is no satisfactory explanation for the creep of concrete at elevated temperatures.

Transient strain occurs during the first time heating of concrete and is independent of time. It is essentially caused by thermal incompatibilities between the aggregate and the cement paste [6]. Transient strain of concrete, similar to that of high temperature creep, is a complex phenomenon and is influenced by factors such as temperature, strength, moisture content, loading, and mix proportions.

2.5. Spalling. In addition to thermal, mechanical, and deformation properties, another property that has a significant influence on the fire performance of a concrete structural member is spalling [33]. This property is unique to concrete and can be a governing factor in determining the fire resistance of an RC structural member [34]. Spalling is defined as the breaking up of layers (pieces) of concrete from the surface of a concrete member when it is exposed to high and rapidly rising temperatures such as those encountered in fires. The spalling can occur soon after exposure to rapid heating and can be accompanied by violent explosions or it may happen during later stages of fire when concrete has become so weak after heating such that, when cracks develop, pieces of concrete fall off from the surface of concrete member. The consequences are limited as long as the extent of damage is small, but extensive spalling may lead to early loss of stability and integrity. Further, spalling exposes deeper layers of concrete to fire temperatures, thereby increasing the rate of transmission of heat to the inner layers of the member, including the reinforcement. When the reinforcement is directly exposed to fire, the temperatures in the reinforcement rise at a very high rate leading to a faster decrease in strength (capacity) of the structural member. The loss of strength in the reinforcement, combined with the loss of concrete due to spalling, significantly decreases the fire resistance of a structural member [35, 36].

While spalling might occur in all concrete types, HSC is more susceptible to fire induced spalling than NSC because of its low permeability and lower water-cement ratio, as compared to NSC. The fire induced spalling is further dependent on a number of factors including permeability of concrete, type of fire exposure, and tensile strength of concrete [34, 37–40]. Thus, information on permeability and tensile strength of concrete, which vary with temperature, are crucial for predicting fire induced spalling in concrete members.

3. Thermal Properties of Concrete at Elevated Temperatures

Thermal properties that govern temperature dependent properties in concrete structures are thermal conductivity, specific heat (or heat capacity), and mass loss. These properties are significantly influenced by the aggregate type, moisture content, and composition of concrete mix. There have been numerous test programs for characterizing thermal properties of concrete at elevated temperatures [16, 41–44]. A detailed review on the effect of temperature on thermal properties of different concrete types is given by Khaliq [45], Kodur et al. [46], and Flynn [47].

3.1. Thermal Conductivity. Thermal conductivity of concrete at room temperature is in the range of 1.4 and 3.6 W/m°K and varies with temperature [18]. Figure 1 illustrates the variation of thermal conductivity of NSC as a function of temperature based on published test data and empirical relations. The test data is compiled by Khaliq [45] from different sources based on experimental data [16, 20, 21, 24, 44, 48] and empirical relations in different standards [4, 15]. The variation in measured test data is depicted through the shaded area in Figure 1 and this variation in reported data on thermal conductivity is mainly attributed to moisture content, type of aggregate, test conditions, and measurement techniques used in experiments [15, 18–20, 41]. It should be noted that there are very few standardized methods available for measuring thermal properties. Also plotted in Figure 1 is both the upper and lower bound values of thermal conductivity as per EC2 provisions and this range is for all aggregate types. However, thermal conductivity shown in Figure 1, as per ASCE relations, is applicable for carbonate aggregates concrete.

Overall thermal conductivity decreases gradually with temperature and this decrease is dependent on the concrete mix properties, specifically moisture content and permeability. This decreasing trend in thermal conductivity can be attributed to variation of moisture content with increase in temperature [18].

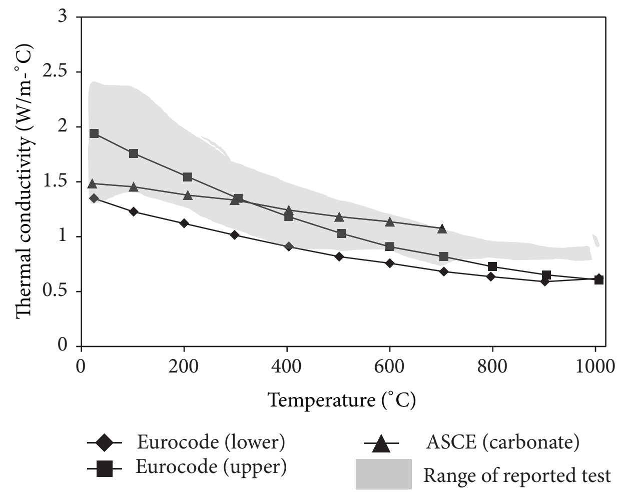

Figure 1: Variation in thermal conductivity of normal strength concrete as a function of temperature.

Thermal conductivity of HSC is higher than that of NSC due to low w/c ratio and use of different binders in HSC [49]. Generally, thermal conductivity of HSC is in the range between 2.4 and 3.6 W/m°K at room temperature. Thermal conductivity for fiber-reinforced concretes (with both steel and polypropylene fibers) almost follow a similar trend as that of plain concrete and lie closer to that of HSC. Therefore, it is deduced that there is no significant effect of fibers on thermal conductivity of concrete in a 20–800°C temperature range [27].

3.2. Specific Heat. The specific heat of concrete at room temperature varies in the range of 840 J/kg·K and 1800 J/kg·K for different aggregate types. Often specific heat is expressed in terms of thermal capacity which is the product of specific heat and density of concrete. The specific heat property is sensitive to various physical and chemical transformations that take place in concrete at elevated temperatures. This includes the vaporization of free water at about 100°C, the dissociation of $Ca(OH)_2$ into CaO and H_2O between 400–500°C, and the quartz transformation of some aggregates above 600°C [24]. Specific heat is therefore highly dependent on moisture content and considerably increases with higher water to cement ratio.

Khaliq and Kodur [27] compiled measured specific heat of different concretes from various studies [16, 20, 24, 41, 44, 48]. Figure 2 illustrates the variation of specific heat for NSC with temperature as reported in various studies based on test data and different standards. The specific heat of concrete type remains almost constant up to 400°C, followed by increases of up to about 700°C and then remains constant between 700 and 800°C range. Of the various factors, aggregate type has a significant influence on the specific heat (thermal capacity) of concrete. This effect is captured in ASCE specified relations for specific heat of concrete [15]. Carbonate aggregate concrete has higher specific heat (heat capacity) in 600–800°C temperature range and this is caused by an endothermic reaction, which results from decomposition of

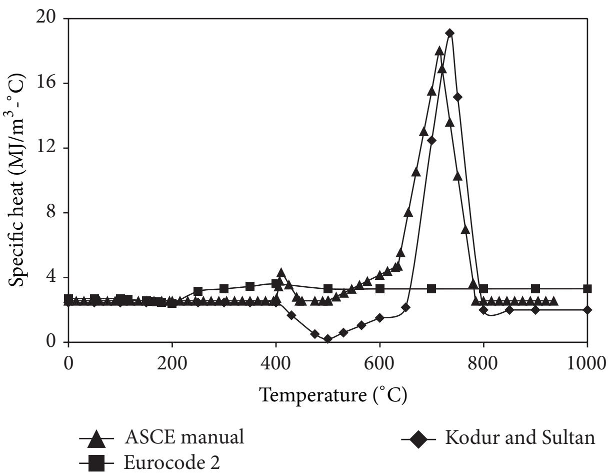

Figure 2: Variation in specific heat of normal strength concrete as a function of temperature.

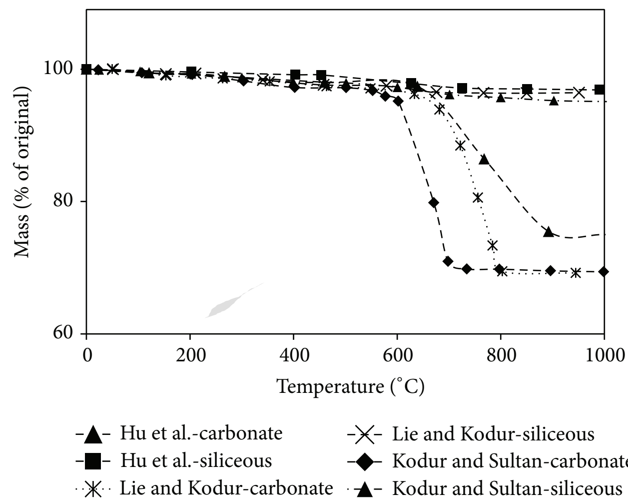

Figure 3: Variation in mass of concrete with different aggregates as a function of temperature.

dolomite and absorbs a large amount of energy [12]. This high heat capacity in carbonate aggregate concrete helps to minimize spalling and enhance fire resistance of structural members.

As compared to NSC, HSC exhibits slightly lower specific heat throughout the 20–800°C temperature range [41]. The presence of fibers also has a minor influence on the specific heat of concrete. For concrete with polypropylene fibers, the burning of polypropylene fibers produces microchannels for release of vapor; and hence the amount of heat absorbed is less for dehydration of chemically bound water; thus its specific heat is reduced in the temperature range of 600–800°C. However, concrete with steel fibers displays a higher specific heat in the 400–800°C temperature range, which can be attributed to additional heat absorbed for dehydration of chemically bound water.

3.3. Mass Loss. Depending on the density, concretes are usually subdivided into two major groups: (1) normal-weight concretes with densities in 2150 to 2450 $\mathrm{kg{\cdot}m^{-3}}$ range; and (2) lightweight concretes with densities between 1350 and 1850 $\mathrm{kg{\cdot}m^{-3}}$. The density or mass of concrete decreases with increasing temperature due to loss of moisture. The retention in mass of concrete at elevated temperatures is highly influenced by the type of aggregate [21, 44].

Figure 3 illustrates the variation in mass of concrete as a function of temperature for concretes made with carbonate and siliceous aggregates. The mass loss is minimal for both carbonate and siliceous aggregate concretes up to about 600°C. However, the type of aggregate has significant influence on mass loss in concretes beyond 600°C. In the case of siliceous aggregate concrete, mass loss is insignificant even above 600°C. However, beyond 600°C, carbonate aggregate concrete experiences a larger percentage of mass loss as compared to siliceous aggregate concrete. This higher percentage of mass loss in carbonate aggregate concrete is attributed to dissociation of dolomite in carbonate aggregate at around 600°C [12].

The strength of concrete does not have a significant influence on mass loss, and hence HSC exhibits a similar trend in mass loss as that of NSC. The mass loss for fiber-reinforced concrete is also similar to plain concrete of up to about 800°C. Above 800°C, the mass loss in steel fiber-reinforced HSC is slightly lower than that of plain HSC.

4. Mechanical Properties of Concrete at Elevated Temperatures

The mechanical properties that are of primary interest in fire resistance design are compressive strength, tensile strength, elastic modulus, and stress-strain response in compression. Mechanical properties of concrete at elevated temperatures have been studied extensively in the literature in comparison to thermal properties [12, 39, 50–52]. High temperature mechanical property tests are generally carried out on concrete specimens that are typically cylinders or cubes of different sizes. Unlike room temperature property measurements, where there are specified specimen sizes as per standards, the high temperature mechanical properties are usually carried out on a wide range of specimen sizes due to a lack of standardized test specifications for undertaking high temperature mechanical property tests [53, 54].

4.1. Compressive Strength. Figures 4 and 5 illustrate the variation of compressive strength ratio for NSC and HSC at elevated temperatures, respectively, with upper and lower bounds (of shaded area) showing range variation in reported test data. Also plotted in these figures is the variation of compressive strength as obtained using Eurocode [4], ASCE [15], and Kodur et al. [46] relations; Figure 4 shows a large but uniform variation of the compiled test data for NSC throughout a 20–800°C temperature range. However, Figure 5 shows a larger variation in compressive strength of HSC with a temperature in the range of 200°C to 500°C and less variation above 500°C. This is mainly because fewer

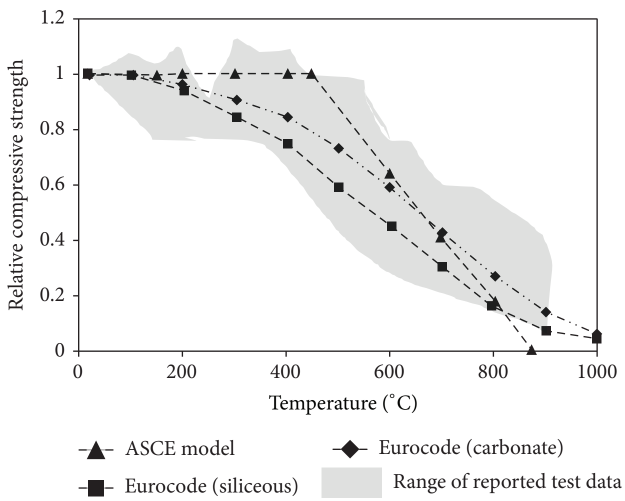

Figure 4: Variation of relative compressive strength of normal strength concrete as a function of temperature.

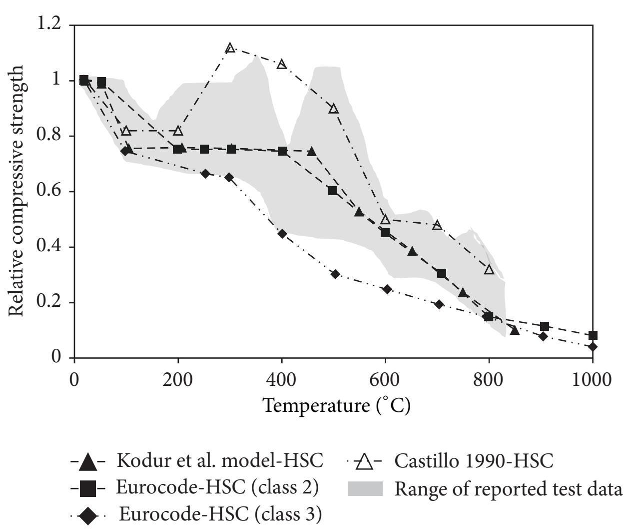

Figure 5: Variation in relative compressive strength of high strength concrete as a function of temperature.

test data points were reported for HSC for temperatures higher than 500°C, either due to the occurrence of spalling in concrete or due to limitations in the test apparatus. However, a wider variation is observed for NSC in this temperature range (above 500°C) when compared to HSC as seen from Figures 4 and 5. This is mainly because of the higher number of test data points reported for NSC in the literature and also due to the lower tendency of NSC to spall under fire. Overall the variation in compressive strength mechanical properties of concrete at high-temperatures is quite high. These variations from different tests can be attributed to using different heating or loading rates, specimen size and curing, condition at testing (moisture content and age of specimen), and the use of admixtures.

In the case of NSC, the compressive strength of concrete is marginally affected by a temperature of up to 400°C. NSC is usually highly permeable and allows easy diffusion of pore pressure as a result of water vapor. On the other hand, the use of different binders in HSC produces a superior and dense microstructure with less amount of calcium hydroxide which ensures a beneficial effect on compressive strength at room temperature [55]. Binders such as use of slag and silica fume give the best results to improve compressive strength at room temperature which is attributed to a dense microstructure. However, as mentioned earlier, the compact microstructure is highly impermeable and under high temperature becomes detrimental as it does not allow moisture to escape resulting in build-up of pore pressure and rapid development of microcracks in HSC leading to a faster deterioration of strength and occurrence of spalling [27, 56, 57]. The presence of steel fibers in concrete helps to slow down strength loss at elevated temperatures [44, 58].

Among the factors that directly affect compressive strength at elevated temperatures are initial curing, moisture content at the time of testing, and the addition of admixtures and silica fume to the concrete mix [59–63]. These factors are not addressed in the literature and there is no test data that shows the influence of these factors on the high-temperature mechanical properties of concrete.

Another main reasoning for the significant variation in the high-temperature strength properties of concrete is the use of different testing conditions (such as heating rate and strain rate) and test procedures (hot strength test and residual strength test) due to a lack of standardized test methods for carrying out property tests [46].

4.2. Tensile Strength. The tensile strength of concrete is much lower than that of compressive strength, and hence tensile strength of concrete is often neglected in strength calculations at room and elevated temperatures. However, from fire resistance point of view, it is an important property, because cracking in concrete is generally due to tensile stresses and the structural damage of the member in tension is often generated by progression in microcracking [26]. Under fire conditions, tensile strength of concrete can be even more crucial in cases where fire induced spalling occurs in concrete member [27]. Thus, information on tensile strength of HSC, which varies with temperature, is crucial for predicting fire induced spalling in HSC members.

Figure 6 illustrates the variation of splitting tensile strength ratio of NSC and HSC as a function of temperature as reported in previous studies and Eurocode provisions [4, 64–66]. The ratio of tensile strength at a given temperature, to that at room temperature, is plotted in Figure 6. The shaded portion in this plot shows a range of variation in splitting tensile strength as obtained by various researchers for NSC with conventional aggregates. The decrease in tensile strength of NSC with temperature can be attributed to weak microstructure of NSC allowing initiation of microcracks. At 300°C, concrete loses about 20% of its initial tensile strength. Above 300°C, the tensile strength of NSC decreases at a rapid rate due to a more pronounced thermal damage in the form of microcracks and reaches to about 20% of its initial strength at 600°C.

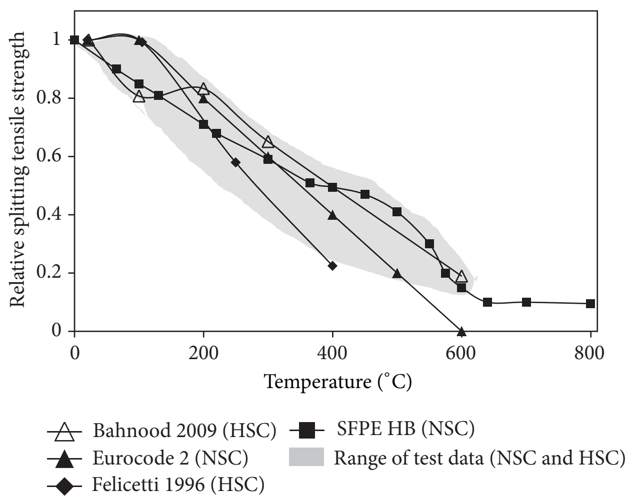

Figure 6: Variation in relative splitting tensile strength of concrete as a function of temperature.

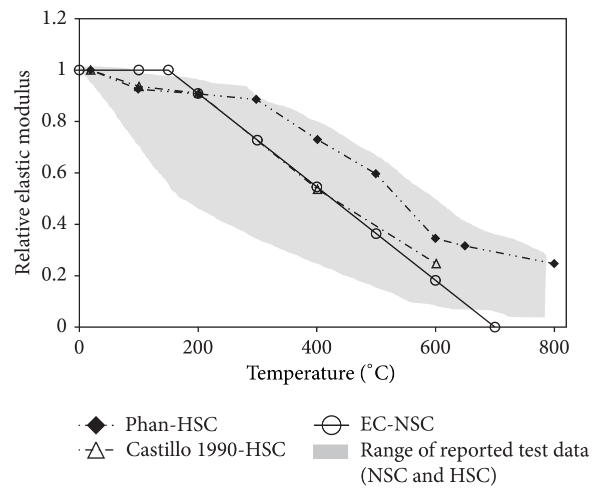

Figure 7: Variation in elastic modulus of concrete as a function of temperature.

HSC experiences a rapid loss of tensile strength at higher temperatures due to development of pore pressure in dense microstructured HSC [55]. The addition of steel fibers to concrete enhances its tensile strength and the increase can be up to 50% higher at room temperature [67, 68]. Further, the tensile strength of steel fiber-reinforced concrete decreases at a lower rate than that of plain concrete throughout the temperature range of 20–800°C [69]. This increased tensile strength can delay the propagation of cracks in steel fiber-reinforced concrete structural members and is highly beneficial when the member is subjected to bending stresses.

4.3. Elastic Modulus. The modulus of elasticity (E) of various concretes at room temperature varies over a wide range, 5.0 $\times 10^3$ to 35.0 $\times 10^3$ MPa, and is dependent mainly on the water-cement ratio in the mixture, the age of concrete, the method of conditioning, and the amount and nature of the aggregates. The modulus of elasticity decreases rapidly with the rise of temperature, and the fractional decline does not depend significantly on the type of aggregate [70]. From other surveys [38, 71], it appears, however, that the modulus of elasticity of normal-weight concretes decreases at a higher pace with the rise of temperature than that of lightweight concretes.

Figure 7 illustrates variation of ratio of elastic modulus at target temperature to that at room temperature for NSC and HSC [4, 19, 72]. It can be seen from the figure that the trend of loss of elastic modulus of both concretes with temperature is similar, but there is a significant variation in the reported test data. The degradation modulus in both NSC and HSC can be attributed to excessive thermal stresses and physical and chemical changes in concrete microstructure.

4.4. Stress-Strain Response. The mechanical response of concrete is usually expressed in the form of stress-strain relations, which are often used as input data in mathematical models for evaluating the fire resistance of concrete structural members.

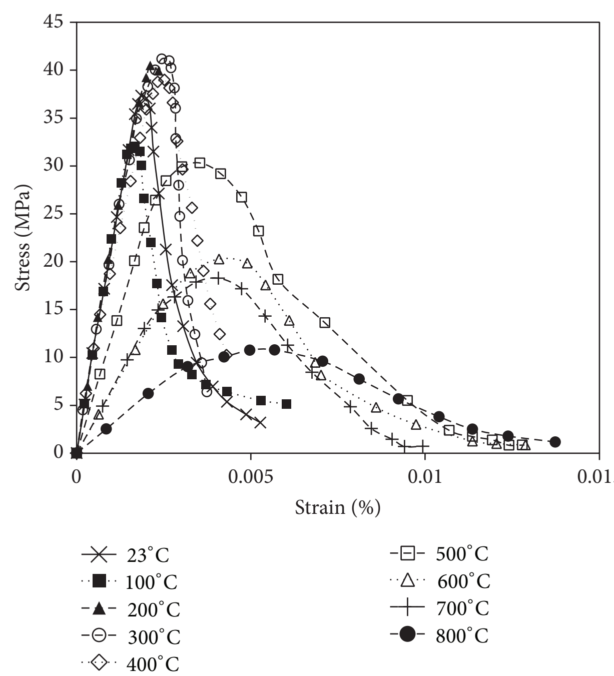

Figure 8: Stress-strain response of normal strength concrete at elevated temperatures.

Generally, because of a decrease in compressive strength and increase in ductility of concrete, the slope of stress-strain curve decreases with increasing temperature. The strength of concrete has a significant influence on stress-strain response both at room and elevated temperatures.

Figures 8 and 9 illustrate stress-strain response of NSC and HSC, respectively, at various temperatures [72, 73]. At all temperatures both NSC and HSC exhibit a linear response followed by a parabolic response till peak stress, and then a quick descending portion prior to failure. In general, it is established that HSC has steeper and more linear stress-strain

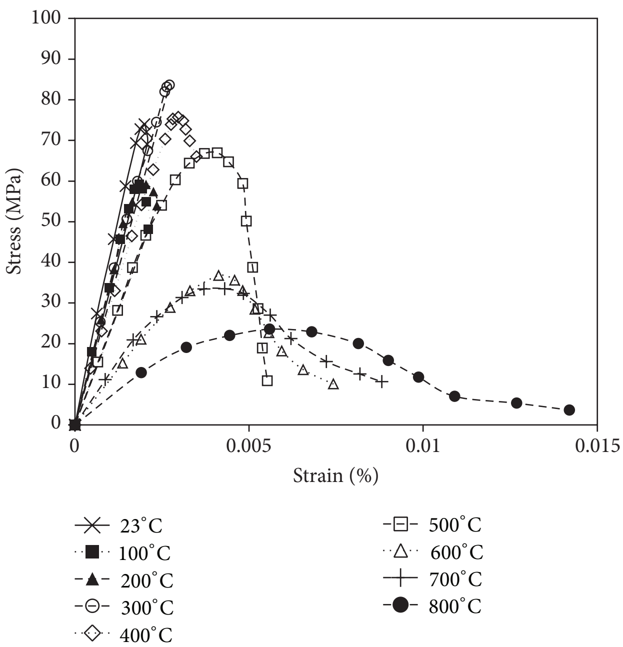

Figure 9: Stress-strain response of high strength concrete at elevated temperatures.

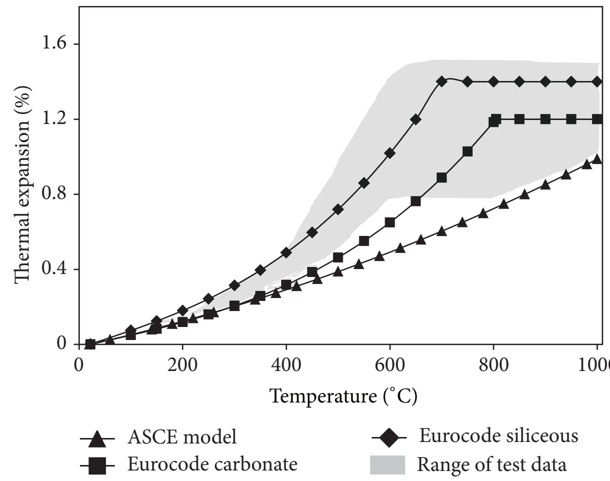

Figure 10: Variation in linear thermal expansion of normal strength concrete as a function of temperature.

curves in comparison to NSC in 20–800°C. The temperature has a significant effect on the stress-strain response of both NSC and HSC, as with the rate of rise in temperature. The strain corresponding to peak stress starts to increase, especially above 500°C. This increase is significant and the strain at peak stress can reach four times the strain at room temperature. HSC specimens exhibit a brittle response as indicated by postpeak behavior of stress-strain curves shown in Figure 9 [74]. In the case of fiber-reinforced concrete, especially with steel fibers, the stress-strain response is more ductile.

5. Deformation Properties of Concrete at Elevated Temperatures

Deformation properties that include thermal expansion, creep strain, and transient strain are highly dependent on the chemical composition, the type of aggregate, and the chemical and physical reactions that occur in concrete during heating [75].

5.1. Thermal Expansion. Concrete generally undergoes expansion when subjected to elevated temperatures. Figure 10 illustrates the variation of thermal expansion in NSC with temperature [4, 15], where the shaded portion indicates the range of test data reported by different researchers [46, 76]. The thermal expansion of concrete increases from zero at room temperature to about 1.3% at 700°C and then generally remains constant through 1000°C. This increase is substantial in the 20–700°C temperature range and is mainly due to high thermal expansion resulting from constituent aggregates and cement paste in concrete. Thermal expansion of concrete is complicated by other contributing factors such as additional volume changes caused by variation in moisture content, by chemical reactions (dehydration, change of composition), and by creep and microcracking resulting from nonuniform thermal stresses [18]. In some cases, thermal shrinkage can also result from loss of water due to heating, along with thermal expansion, and this might lead to the overall volume change to be negative, that is, shrinkage rather than expansion.

Eurocodes [4] accounts for the effect of type of aggregate on variation of thermal expansion than of concrete with temperature. Concrete made with siliceous aggregate has a higher thermal expansion than that of carbonate aggregate concrete. However, ASCE provisions [15] provide only one variation for both siliceous and carbonate aggregate concrete.

The strength of concrete and presence of fiber have moderate influence on thermal expansion. The rate of expansion for HSC and fiber-reinforced concrete slows down between 600–800°C; however the rate of thermal expansion increases again above 800°C. The slowdown in thermal expansion in the 600–800°C range is attributed to the loss of chemically bound water in hydrates, and the increase in expansion above 800°C is attributed to a softening of concrete and excessive micro- and macrocrack development [77].

5.2. Creep and Transient Strains. Time-dependent deformations in concrete such as creep and transient strains get highly enhanced at elevated temperatures under compressive stresses [18]. Creep in concrete under high temperatures increases due to moisture movement out of concrete matrix. This phenomenon is further intensified by moisture dispersion and loss of bond in cement gel (C–S–H). Therefore, the process of creep is caused and accelerated mainly by two processes: (1) moisture movement and dehydration of concrete due to high temperatures and (2) acceleration in the process of breakage of bond.

Transient strain occurs during the first time heating of concrete, but it does not occur upon repeated heating [78]. Exposure of concrete to high temperature induces complex changes in the moisture content and chemical composition of the cement paste. Moreover, there exists a mismatch in the thermal expansion between the cement paste and the aggregate. Therefore, factors such as changes in chemical composition of concrete and mismatches in thermal expansion lead to internal stresses and microcracking in the concrete constituents (aggregate and cement paste) and results in transient strain in the concrete [75].

A review of the literature shows that there is limited information on creep and transient strain of concrete at elevated temperatures [46]. Some data on the creep of concrete at elevated temperatures is available from the work of Cruz, [70], MareÂchal [79], Gross [80], and Schneider et al. [81]. Anderberg and Thelandersson [82] carried out tests to evaluate transient and creep strains under elevated temperatures. They found that predried specimens at 45 and 67.5% of load stress level were less liable to deformation in the "positive direction" (expansion) under load. At 22.5% preload, specimens displayed no significant difference of strains. They also found that the influence of water saturation was not very significant except for free thermal expansion (0% preload), which was found to be smaller for water saturated specimens.

Khoury et al. [78] studied creep strain of initially moist concretes at four load levels measured during first heating at 1°C/min. An important feature of these results was that a considerable contraction under load was observed as compared to free (unloaded) thermal strains. This contraction is referred to as the "load-induced thermal strain" and the actual thermal strain is considered to consist of total thermal strain minus the load-induced thermal strain.

Schneider [75] also investigated the effect of transient and creep restraint on deformation of concrete. He concluded that the transient test for measuring total deformation or restraint of concrete has the strongest relation to building fires and is supposed to give the most realistic data with direct relevance to fire. The important conclusions from the study are that (1) water to cement ratio and original strength are of little importance on creep deformations under transient conditions, (2) aggregate to cement ratio has a great influence on the strains and critical temperatures: the harder the aggregate the lower the thermal expansion; therefore total deformation in transient state will be lower; and (3) curing conditions are of great importance in the 20–300°C range: air-cured and oven-dried specimens have lower transient and creep strains than water cured specimens.

Anderberg and Thelandersson [82] developed constitutive models for creep and transient strains in concrete at elevated temperatures. These equations for creep and transient strain at elevated temperatures as suggested by Anderberg and Thelandersson [82] are

$$\varepsilon_{\mathrm{cr}} = \beta_1 \frac{\sigma}{f_{c,T}} \sqrt{t} e^{d(T-293)},$$
$$\varepsilon_{\mathrm{tr}} = k_2 \frac{\sigma}{f_{c,20}} \varepsilon_{\mathrm{th}}, \tag{2}$$

where $\varepsilon_{\mathrm{cr}}$ = creep strain, $\varepsilon_{\mathrm{tr}}$ = transient strain, $\beta_1 = 6.28 \times 10^{-6}\,\mathrm{s}^{-0.5}$, $d = 2.658 \times 10^{-3}\,\mathrm{K}^{-1}$, T = concrete temperature (°K) at time t (s), $f_{c,T}$ = concrete strength at temperature T, σ = stress in the concrete at the current temperature, k_2 = a constant ranges between 1.8 and 2.35, $\varepsilon_{\mathrm{th}}$ = thermal strain, and $f_{c,20}$ = concrete strength at room temperature.

The above discussed information on high temperature creep and transient strain is mostly developed for NSC. There is still a lack of test data and models on the effect of temperature on creep and transient strain in HSC and fiber-reinforced concrete.

6. Fire Induced Spalling

A review of the literature presents a conflicting picture on the occurrence of fire induced spalling and also on the exact mechanism of spalling in concrete. While some researchers reported explosive spalling in concrete structural members exposed to fire, a number of other studies reported little or no significant spalling. One possible explanation for this confusing trend of observations is the large number of factors that influence spalling and their interdependency. However, most researchers agree that major causes for fire induced spalling in concrete are low permeability of concrete and moisture migration in concrete at elevated temperatures.

There are two broad theories by which the spalling phenomenon can be explained [83].

(i) Pressure Build-Up. Spalling is believed to be caused by the build-up of pore pressure during heating [83–85]. The extremely high water vapor pressure, generated during exposure to fire, cannot escape due to the high density and compactness (and low permeability) of higher strength concrete. When the effective pore pressure (porosity times pore pressure) exceeds the tensile strength of concrete, chunks of concrete fall off from the structural member. This pore pressure is considered to drive progressive failure; that is, the lower the permeability of concrete, the greater the fire induced spalling. This falling-off of concrete chunks can often be explosive depending on fire and concrete characteristics [38, 86].

(ii) Restrained Thermal Dilatation. This hypothesis considers that spalling results from restrained thermal dilatation close to the heated surface, which leads to the development of compressive stresses parallel to the heated surface. These compressive stresses are released by brittle fracture of concrete (spalling). The pore pressure can play a significant role on the onset of instability in the form of explosive thermal spalling [87].

Although spalling might occur in all concretes, high-strength concrete is believed to be more susceptible to spalling than normal-strength concrete because of its low permeability and low water-cement ratio [88, 89]. The high water vapor pressure, generated due to a rapid rise in temperature, cannot escape due to high density (and low permeability) of HSC, and this pressure build-up often reaches the saturation vapor pressure. At 300°C, the pore pressure can reach up to 8 MPa; such internal pressures are often too high to

Figure 11: Relative spalling in NSC and HSC columns under fire conditions.

be resisted by the HSC mix having a tensile strength of approximately 5 MPa [84]. The drained conditions at the heated surface and the low permeability of concrete lead to strong pressure gradients close to the surface in the form of the so-called "moisture clog" [38, 86]. When the vapor pressure exceeds the tensile strength of concrete, chunks of concrete fall off from the structural member. In a number of test observations on HSC columns, it has been found that spalling is often of an explosive nature [19, 90]. Hence, spalling is one of the major concerns in the use of HSC in building applications and should be properly accounted for in evaluating fire performance [91]. Spalling in NSC and HSC columns is compared in Figure 11 using data obtained from full-scale fire tests on loaded columns [92]. It can be seen that the spalling is quite significant in fire exposed HSC column.

The extent of spalling depends on a number of factors including strength, porosity, density, load level, fire intensity, aggregate type, relative humidity, amount of silica fume, and other admixtures [34, 93, 94]. Many of these factors are interdependent and this makes prediction of spalling quite complex. The variation of porosity with temperature is the most important property needed for predicting spalling performance of HSC [33]. Noumowé et al. carried out porosity measurements on NSC and HSC specimens, using a mercury porosimeter, at various temperatures [88, 95].

Based on limited fire tests, researchers have suggested that spalling in HSC can be minimized by adding polypropylene fibres to the HSC mix [85, 96–101]. The polypropylene fibers melt when temperatures in concrete reach about 160–170°C and this creates pores in concrete that are sufficient for relieving vapor pressure developed in the concrete. Another alternative for limiting fire induced spalling in HSC columns is through the use of bent ties, where ties are bent at 135° into the concrete core [102].

7. Relations of High Temperature Properties of Concrete

There are limited constitutive relations for high-temperature properties of concrete in codes and standards that can be used for fire design. These relations can be found in the ASCE manual [15] and Eurocode 2 [4]. Kodur et al. [46] have compiled different relations that are available for thermal, mechanical, and deformation of concrete at elevated temperatures.

There are some differences in the constitutive relationships for high-temperature properties of concrete used in European and American standards. The constitutive relations in Eurocode are applicable for NSC and HSC, while the relations in the ASCE manual of practice are for NSC only. The constitutive relationships for high-temperature properties of concrete specified in the Eurocode and the ASCE manual are summarized in Table 1. In addition to these constitutive models, Kodur et al. [93] proposed constitutive relations for HSC, which are an extension to ASCE relations for NSC. These relations for HSC are also included in Table 1.

A major difference between the European and the ASCE high-temperature constituent relations for concrete is the effect of aggregate type on 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 bound limits without indicating which limit to use for a given aggregate type in concrete. Furthermore, Eurocode classifies HSC into three classes, depending on its compressive strength, namely,

(i) class 1 for concrete with compressive strength between C55/67 and C60/75,

(ii) class 2 for concrete with compressive strength between C70/85 and C80/95,

(iii) class 3 for concrete with compressive strength higher than C90/105.

8. Summary

Concrete, at elevated temperatures, undergoes significant physicochemical changes. These changes cause properties to deteriorate at elevated temperatures and introduce additional complexities, such as spalling in HSC. Thus, thermal, mechanical, and deformation properties of concrete change substantially within the temperature range associated with building fires. Furthermore, many of these properties are temperature dependent and sensitive to testing (method) parameters such as heating rate, strain rate, temperature gradient, and so on.

Based on information presented in this chapter, it is evident that high temperature properties of concrete are crucial for modeling fire response of reinforced concrete structures. A good amount of data exists on high temperature thermal, mechanical, and deformation properties of NSC

TABLE 1: Constitutive relationships of high-temperature properties of concrete.

	NSC—ASCE Manual 1992	HSC—Kodur et al. 2004 [10]	NSC and HSC—EN1992-1-2: 2004 [4]
Stress-strain relationships	$\sigma_c = \begin{cases} f'_{c,T}\left[1-\left(\dfrac{\varepsilon-\varepsilon_{\max,T}}{\varepsilon_{\max,T}}\right)^2\right], & \varepsilon \le \varepsilon_{\max,T} \\ f'_{c,T}\left[1-\left(\dfrac{\varepsilon_{\max,T}-\varepsilon}{3\varepsilon_{\max,T}}\right)^2\right], & \varepsilon > \varepsilon_{\max,T}, \end{cases}$ $f'_{c,T} = \begin{cases} f'_c, & 20^\circ\mathrm{C} \le T \le 450^\circ\mathrm{C} \\ f'_c\left[2.011 - 2.353\left(\dfrac{T-20}{1000}\right)\right], & 450^\circ\mathrm{C} < T \le 874^\circ\mathrm{C} \\ 0, & 874^\circ\mathrm{C} < T, \end{cases}$ $\varepsilon_{\max,T} = 0.0025 + \left(6.0T + 0.04T^2\right) \times 10^{-6}.$	$\sigma_c = \begin{cases} f'_{c,T}\left[1-\left(\dfrac{\varepsilon_{\max,T}-\varepsilon}{\varepsilon_{\max,T}}\right)^H\right], & \varepsilon \le \varepsilon_{\max,T} \\ f'_{c,T}\left[1-\left(\dfrac{30\left(\varepsilon-\varepsilon_{\max,T}\right)}{\left(130-f'_c\right)\varepsilon_{\max,T}}\right)^2\right], & \varepsilon > \varepsilon_{\max,T}, \end{cases}$ $f'_{c,T} = \begin{cases} f'_c\left[1.0 - 0.003125\left(T-20\right)\right], & T < 100^\circ\mathrm{C} \\ 0.75 f'_c, & 100^\circ\mathrm{C} \le T \le 400^\circ\mathrm{C} \\ f'_c\left[1.33 - 0.00145T\right], & 400^\circ\mathrm{C} < T, \end{cases}$ $\varepsilon_{\max,T} = 0.0018 + \left(6.7 f'_c + 6.0T + 0.03T^2\right) \times 10^{-6},$ $H = 2.28 - 0.012 f'_c.$	$\sigma_c = \dfrac{3\varepsilon f'_{c,T}}{\varepsilon_{c1,T}\left(2+\left(\varepsilon/\varepsilon_{c1,T}\right)^3\right)}, \quad \varepsilon \le \varepsilon_{cu1,T}.$ 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 2.
Thermal capacity	Siliceous aggregate concrete $\rho c = \begin{cases} 0.005T + 1.7, & 20^\circ\mathrm{C} \le T \le 200^\circ\mathrm{C} \\ 2.7, & 200^\circ\mathrm{C} < T \le 400^\circ\mathrm{C} \\ 0.013T - 2.5, & 400^\circ\mathrm{C} < T \le 500^\circ\mathrm{C} \\ 10.5 - 0.013T, & 500^\circ\mathrm{C} < T \le 600^\circ\mathrm{C} \\ 2.7, & 600^\circ\mathrm{C} < T. \end{cases}$ Carbonate aggregate concrete $\rho c = \begin{cases} 2.566, & 20^\circ\mathrm{C} \le T \le 400^\circ\mathrm{C} \\ 0.1765T - 68.034, & 400^\circ\mathrm{C} < T \le 410^\circ\mathrm{C} \\ 25.00671 - 0.05043T, & 410^\circ\mathrm{C} < T \le 445^\circ\mathrm{C} \\ 2.566, & 445^\circ\mathrm{C} < T \le 500^\circ\mathrm{C} \\ 0.01603T - 5.44881, & 500^\circ\mathrm{C} < T \le 635^\circ\mathrm{C} \\ 0.16635T - 100.90225, & 635^\circ\mathrm{C} < T \le 715^\circ\mathrm{C} \\ 176.07343 - 0.22103T, & 715^\circ\mathrm{C} < T \le 785^\circ\mathrm{C} \\ 2.566, & 785^\circ\mathrm{C} < T. \end{cases}$	Siliceous aggregate concrete $\rho c = \begin{cases} 0.005T + 1.7, & 20^\circ\mathrm{C} \le T \le 200^\circ\mathrm{C} \\ 2.7, & 200^\circ\mathrm{C} < T \le 400^\circ\mathrm{C} \\ 0.013T - 2.5, & 400^\circ\mathrm{C} < T \le 500^\circ\mathrm{C} \\ -0.013T + 10.5, & 500^\circ\mathrm{C} < T \le 600^\circ\mathrm{C} \\ 2.7, & 600^\circ\mathrm{C} < T \le 635^\circ\mathrm{C}. \end{cases}$ Carbonate aggregate concrete $\rho c = \begin{cases} 2.45, & 20^\circ\mathrm{C} \le T \le 400^\circ\mathrm{C} \\ 0.026T - 12.85, & 400^\circ\mathrm{C} < T \le 475^\circ\mathrm{C} \\ 0.0143T - 6.295, & 475^\circ\mathrm{C} < T \le 650^\circ\mathrm{C} \\ 0.1894T - 120.11, & 650^\circ\mathrm{C} < T \le 735^\circ\mathrm{C} \\ -0.263T + 212.4, & 735^\circ\mathrm{C} < T \le 800^\circ\mathrm{C} \\ 2, & 800^\circ\mathrm{C} < T \le 1000^\circ\mathrm{C}. \end{cases}$	Specific heat (J/kg C) $c = 900$, for $20^\circ\mathrm{C} \le T \le 100^\circ\mathrm{C}$, $c = 900 + (T - 100)$, for $100^\circ\mathrm{C} < T \le 200^\circ\mathrm{C}$, $c = 1000 + (T - 200)/2$, for $200^\circ\mathrm{C} < T \le 400^\circ\mathrm{C}$, $c = 1100$, for $400^\circ\mathrm{C} < T \le 1200^\circ\mathrm{C}$. Density change ($\mathrm{kg/m^3}$) $\rho = \rho(20^\circ\mathrm{C})$ = Reference density for $20^\circ\mathrm{C} \le T \le 115^\circ\mathrm{C}$, $\rho = \rho(20^\circ\mathrm{C})(1 - 0.02(T - 115)/85)$ for $115^\circ\mathrm{C} < T \le 200^\circ\mathrm{C}$, $\rho = \rho(20^\circ\mathrm{C})(0.98 - 0.03(T - 200)/200)$ for $200^\circ\mathrm{C} < T \le 400^\circ\mathrm{C}$, $\rho = \rho(20^\circ\mathrm{C})(0.95 - 0.07(T - 400)/800)$ for $400^\circ\mathrm{C} < T \le 1200^\circ\mathrm{C}$, Thermal capacity $= \rho \times c$.
Thermal conductivity	Siliceous aggregate concrete $k_c = \begin{cases} -0.000625T + 1.5, & 20^\circ\mathrm{C} \le T \le 800^\circ\mathrm{C} \\ 1.0, & 800^\circ\mathrm{C} < T, \end{cases}$ Carbonate aggregate concrete $k_c = \begin{cases} 1.355, & 20^\circ\mathrm{C} \le T \le 293^\circ\mathrm{C} \\ -0.001241T + 1.7162, & 293^\circ\mathrm{C} < T. \end{cases}$	Siliceous aggregate concrete $k_c = 0.85\left(2 - 0.0011\,T\right), \quad 20^\circ\mathrm{C} < T \le 1000^\circ\mathrm{C}.$ Carbonate aggregate concrete $k_c = \begin{cases} 0.85\left(2 - 0.0013T\right), & 20^\circ\mathrm{C} \le T \le 300^\circ\mathrm{C} \\ 0.85\left(2.21 - 0.002T\right), & 300^\circ\mathrm{C} < T. \end{cases}$	All types: Upper limit: $k_c = 2 - 0.2451(T/100) + 0.0107(T/100)2$, for $20^\circ\mathrm{C} \le T \le 1200^\circ\mathrm{C}$. Lower limit: $k_c = 1.36 - 0.136(T/100) + 0.0057(T/100)2$, for $20^\circ\mathrm{C} \le T \le 1200^\circ\mathrm{C}$.
Thermal strain	All types: $\varepsilon_{th} = \left[0.004\left(T^2 - 400\right) + 6\left(T - 20\right)\right] \times 10^{-6}.$	All types: $\varepsilon_{th} = \left[0.004\left(T^2 - 400\right) + 6\left(T - 20\right)\right] \times 10^{-6}.$	Siliceous aggregates: $\varepsilon_{th} = -1.8 \times 10^{-4} + 9 \times 10^{-6}T + 2.3 \times 10^{-11}T^3$, for $20^\circ\mathrm{C} \le T \le 700^\circ\mathrm{C}$. $\varepsilon_{th} = 14 \times 10^{-3}$, for $700^\circ\mathrm{C} < T \le 1200^\circ\mathrm{C}$, Calcareous aggregates: $\varepsilon_{th} = -1.2 \times 10^{-4} + 6 \times 10^{-6}T + 1.4 \times 10^{-11}T^3$, for $20^\circ\mathrm{C} \le T \le 805^\circ\mathrm{C}$. $\varepsilon_{th} = 12 \times 10^{-3}$, for $805^\circ\mathrm{C} < T \le 1200^\circ\mathrm{C}$.

Table 2: Values of the main parameters of the stress-strain relationships of NSC and HSC at elevated temperatures as specified in EN1992-1-2: 2004 [4].

Temp. °F	Temp. °C	NSC						HSC		
		Siliceous agg.			Calcareous agg.			$f'_{c,T}/f'_c(20^{\circ}\mathrm{C})$		
		$f'_{c,T}/f'_c(20^{\circ}\mathrm{C})$	$\varepsilon_{c1,T}$	$\varepsilon_{cu1,T}$	$f'_{c,T}/f'_c(20^{\circ}\mathrm{C})$	$\varepsilon_{c1,T}$	$\varepsilon_{cu1,T}$	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

The Eurocode classifies HSC into three classes*, depending on its compressive strength, namely,
(i) class 1 for concrete with compressive strength between C55/67 and C60/75,
(ii) class 2 for concrete with compressive strength between C70/85 and C80/95,
(iii) class 3 for concrete with compressive strength higher than C90/105.
The strength notation of C55/67 refers to a concrete grade with a characteristic cylinder and cube strength of 55 N/mm^2 and 67 N/mm^2, respectively.
*Note: where the actual characteristic strength of concrete is likely to be of a higher class than that specified in the design; the relative reduction in strength for the higher class should be used for fire design.

and HSC. However, there is very limited property data on high temperature properties of new types of concrete such as self-consolidated concrete and fly ash concrete at elevated temperatures.

The review on material properties provided in this chapter is a broad outline of currently available information. Additional details related to specific conditions on which these properties are developed can be found in cited references. Also, when using the material properties presented in this chapter, due consideration should be given to batch mix properties and other characteristics, such as heating rate and loading level, because the properties at elevated temperatures depend on a number of factors.

Disclaimer

Certain commercial products are identified in this paper in order to adequately specify the experimental procedure. In no case does such identification imply recommendations or endorsement by the author, nor does it imply that the product or material identified is the best available for the purpose.

Conflict of Interests

The author declares that there is no conflict of interests regarding the publication of this paper.

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