Mechanical properties

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Chapter 5
Mechanical Properties

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Abstract This chapter report the most recent experimental results on mechanical behaviour at high temperature of high-performance concretes. After a short introduction, Sect. 5.2 describes the main testing methods that were used in the analysed studies. Section 5.3 collects and compares the temperature-dependency of the compressive strength and modulus of elasticity of many experimental studies. The influence of parameters such as the initial compressive strength, the type of aggregate, the presence of additions, the W/C ratio, the moisture content and the way the mechanical test was performed is analysed. Section 5.4 presents the experimental results obtained under a constant temperature, i.e. creep tests at high temperature.

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Section 5.5 presents experimental results obtained under increasing temperature. These results allow assessing the free thermal strain of concrete (when no mechanical load is applied) and the so-called “transient thermal strain”. Finally, Sect. 5.6 collects and analyses the few results concerning the temperature-dependency of the tensile strength of high-performance concretes.

5.1 Introduction

Pierre Pimienta and Jean-Christophe Mindeguia

When it comes to justify the fire resistance of a concrete structure by calculation, by using either an engineering model (Eurocode 2 for example) or an advanced model, the temperature-dependency of the mechanical properties must be known. As part of an elastic calculation for example, assessing the modulus of elasticity is required, as well as its evolution with temperature, which is often representative of the thermal degradation of the material. In the context of advanced calculation models, e.g. taking into account the plasticity of the material, knowledge of compressive and tensile strength is also necessary.

When doing an advanced calculation, the thermo-mechanical behaviour of the material under fire is simulated. Knowledge of thermal properties is therefore necessary in order to correctly simulate the heat diffusion in the material under the effect of a fire (thermal properties previously presented in Chap. 4). Then, the coupling with the mechanical behaviour is done through the thermal expansion coefficient, obtained as the derivative of the free thermal expansion curve of the material (free means without mechanical loading). Moreover, the behaviour of a concrete element initially mechanically loaded, and simultaneously subjected to a gradual rise in temperature (as in the case of columns) strongly depends on the so-called transient thermal strain (TTS). This phenomenon is specific to concrete and constitutes an independent component of the total deformation under fire of a loaded structure element. Finally, in some special cases, concrete structures can be subjected to a constant high temperature, and a constant mechanical load (in most cases compression). We can cite, for example, the containment structures for nuclear power plants

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in accidental situation which can last several days. In this case, knowledge of creep, and its temperature dependence, is necessary.

It is in this context that the most recent experimental results on mechanical behaviour at high temperature of high-performance concretes are collected and analysed in this section.

This chapter is composed of 6 sub-sections. After a short introduction, Sect. 5.2 describes the main testing methods that were used in the analysed studies. Details are given concerning the heating rates, the hydric boundary conditions and the way the mechanical tests can be carried out (hot or residual testing). A distinction is also made concerning the heating regime during the mechanical test: either constant temperature (steady-state test) or increasing temperature (transient tests). Section 5.3 collects and compares the temperature-dependency of the compressive strength and modulus of elasticity of many experimental studies. The influence of parameters such as the initial compressive strength, the type of aggregate, the presence of additions, the W/C ratio, the moisture content and the way the mechanical test was performed is analysed. Sections 5.4 and 5.5 are focused on the mechanical behaviour at high temperature of high-performance concretes under a sustained mechanical load. Section 5.4 presents the experimental results obtained under a constant temperature, i.e. creep tests at high temperature. Section 5.5 presents experimental results obtained under increasing temperature. These results allow assessing the free thermal strain of concrete (when no mechanical load is applied) and the so-called "transient thermal strain". The influence of many parameters on that strain component is analysed. Finally, Sect. 5.6 collects and analyses the few results concerning the temperature-dependency of the tensile strength of high-performance concretes.

The authors underline the fact that fracture mechanics data has not been included in this report. Experimental and numerical materials on fracture mechanics will be further analysed in the RILEM Technical Committee SPF (Spalling of concrete due to Fire: testing and modelling).

5.2 Test Methods

Izabela Hager, Gérard Debicki and Pierre Pimienta

Many investigations of the evolution of concrete properties with temperature have been reported, however the comparison of results obtained by different researchers must be carefully performed due to the fact that concrete properties are closely related to the test conditions that are employed (i.e. the heating rate, presence of loading during heating, specimens tested hot or after cooling down). The test procedure should correspond to the service of the concrete element. In this chapter different testing regimes for determining mechanical concrete properties are briefly described and discussed.

Recently, an attempt to standardise mechanical properties tests was undertaken by RILEM Technical Committees (129-MHT, 2000 and 200-HTC) and testing pro-

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cedures have been established and published in the form of recommendations, Parts 1–11, see References. In those recommendations the details of specimen preparation, specimen conditioning and the testing procedures are described.

There are two main groups of high temperature tests: *steady state tests* and *transient tests*. The steady state tests are characterised by a specific heating period, after which the temperature in the cross section of the specimen is stabilised, and then the test is performed. In the transient tests the temperature varies while the mechanical test is performed. Most often the temperature increases linearly with time.

The choice of testing procedure should be suitable to the concrete element exploitation conditions. Following the classification that was proposed by RILEM, the exposition to the temperature and the specimens conditioning may correspond to the *service* or *accident conditions*. *Service conditions* commonly involve long-term exposure to test temperatures in the range 20–200 °C. *Accident conditions* usually involve short-term exposure to temperature in the range 20–750 °C or above.

Due to the fact that the moisture content of the specimens plays an important role in the material behaviour at high temperature, two specimen conditioning types (boundary conditions) are considered. As it is presented in Table 5.1, testing procedures may be performed with *sealed* or *unsealed* concrete specimens. Unsealed test specimens are more frequently used in investigations which are related to fire behaviour of concrete.

5.2.1 Steady State Tests

5.2.1.1 Steady State Tests of Mechanical Properties

In steady state test procedures the specimen is heated to the desired test temperature (T) with a constant heating rate expressed in degrees per minute. The applied heating rate should be slow enough to minimise the thermal gradient between surface and specimen core. In the literature investigations usually use rates in the range of 0.1–10 °C/min. Nevertheless, the heating rates recommended by RILEM and given for service and accident conditions depend on the specimen size. They are given in Table 5.2.

It is recommended that cylindrical concrete specimens with a length/diameter ratio (slenderness) between 3 and 5 are used. The minimum diameter of the specimen should be four times the maximum aggregate size for cored samples and five times for cast specimens. The recommended diameters of the test specimen are 150, 100, 80 and 60 mm.

When the test temperature T is reached it is maintained until a homogenous temperature distribution is achieved within the specimen. After temperature homogenisation, the specimen is subjected to a constant *rate of stress* or *constant rate of strain* in order to determine the failure load. RILEM recommends the loading and unloading to be performed at a rate of 0.5 ± 0.1 MPa/s in compression and 0.05 MPa/s in tension.

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Table 5.1 Different testing procedures

(a) Heating duration		
Service conditions	In this case material is continuously subjected to temperature from 20 to 200 °C	Nuclear plants
Accident conditions	In the case of an accident the temperature rises to 750 °C or above, however the exposition time is considered to be short	Case of fire
(b) Specimen boundary conditions		
Non sealed specimens, drying is possible (d: drying)	This type of specimen conditioning (d) refers to structures exposed to air and with maximum thickness of 400 mm or structures with no point located at a distance above 200 mm from the surface exposed to air	Case of fire
Sealed specimens, drying is prevented (nd: non drying)	This type of specimen conditioning (nd) applies to: – Sealed structures independent of their dimensions – Zones of structures with a distance >200 mm from the surface exposed to air – Structures under water	Nuclear plants, or massive structures where water evaporation possibilities are limited
(c) Mechanical properties determination		
Hot tested properties	Properties that are describing material performances in fire	
Properties tested after cooling down the specimen (residual test)	Those properties describe material performances after fire, they are suitable to assess the post-fire properties of concrete	

Table 5.2 RILEM's recommended heating rates R at the surface of the specimen for service and accident conditions (RILEM TC 129-MHT 2000a, b, c)

Specimen diameter (mm)	Service conditions R (°C/min)	Accident conditions R (°C/min)
150, 100	0.1	0.5
80, 60	0.1	2

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From the maximum load value the material strength can be determined. These results correspond to, so called, hot tested values of mechanical properties. In this category, two types of results may be distinguished: those determined for specimens that are *unloaded during heating* and those for specimens that are *loaded during heating.*

In many cases, in experimental investigations the material properties are determined after temperature exposure and cooling down to ambient temperature. Those data correspond to the *residual* mechanical properties of the material. The *residual strength tests* are most suitable for assessing the post-fire properties of concrete. Correspondingly, those test procedures require the application of similar heating procedures as described above. Specimens are heated to the target temperature with a constant heating rate. When the test temperature T is reached it is maintained until a homogenous temperature distribution within the specimen is achieved and afterwards the specimen is cooled down to the ambient temperature. Then the specimen is subjected to a constant *rate of stress* or *constant rate of strain* and the failure load is determined. Similarly to hot tested specimens, one can apply the procedure in which the specimen is *loaded during heating* or *unloaded during heating*.

For specimens that are *loaded during heating,* a preload up to the selected initial stress ratio α_i at ambient temperature, often in the range of 20–40% of the ultimate compressive strength at 20 °C, is applied to the concrete specimen prior to heating, and the load is sustained during the heating period. The results of these tests are most suitable for representing fire performance of concrete in a column or in the compression zone of a beam. When testing *unloaded specimens* (unstressed specimens) the specimen is heated, without preload, at a constant rate to the target temperature, which is maintained until a thermal steady state is reached within the specimen.

The RILEM recommendations for mechanical tests in steady state conditions test are given in RILEM TC 129 MHT (1995, 2000a, b, c, 2004, 2007a, b, c).

5.2.1.2 Steady State Creep Tests

The test specimen is heated to the target temperature T with the constant heating rate. When thermal equilibrium is reached a constant mechanical load is applied up to the selected initial stress ratio α_i, usually a percentage of the ultimate strength of the specimen measured at the ambient temperature, prior to heating.

The temperature T and the applied load are kept constant during the whole test period. During the test the strain is measured. Under sustained constant load creep deformation takes place. In the creep results shrinkage should be considered. The shrinkage values are determined separately by using an unloaded reference specimen. The results provide the relationship between strain and time, where time is extending over a long period. The RILEM recommendations for steady state creep and recovery are given in RILEM TC 129-MHT (2000a, b, c).

Recovery strain tests consist of measuring the strain at the end of the creep test, i.e. instantaneously after the unloading of the specimen and after a certain time.

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5.2.1.3 Steady State Relaxation Test

The test specimen is heated to the target temperature T with a constant heating rate. When thermal equilibrium is reached a constant strain is applied (ε = const). The temperature T and the strain that are applied are kept constant during the whole test period. The specimen is maintained at this initial strain during the duration of the test, and the stress change in time is recorded. The RILEM recommendations for relaxation are given in RILEM TC 200-HTC (2007a, b, c).

In creep and relaxation tests, performed in steady temperature conditions, the duration of the test exceeds the practical duration of fire. Thus, those test results have little relevance to real fire performance of concrete. However, those data are valuable when the long term behaviour of concrete elements at temperature is the focus.

5.2.2 Transient Tests

Transient temperature tests are characterised by a constant rate of temperature increase. Those test results are much more relevant to a real fire situation.

5.2.2.1 Thermal Strain Tests

In this type of test the specimen is heated to the target temperature T with a constant heating rate. The goal of the test is to measure strains in the heated concrete specimen induced by heating, without any mechanical load or restraint.

The RILEM recommendations on thermal strain determination procedures are given in RILEM (1997).

5.2.2.2 Thermal Strains Under Load

The specimen is subjected to a constant applied stress ratio α_i. Then the specimen is heated at a constant heating rate. The measurements result in a series of strain versus time curves corresponding to different applied stress ratios. Depending on the stress ratio α_i specimen rupture can take place during heating.

5.2.2.3 Transient Thermal Strains

The values of transient thermal strains are derived from the results of thermal strains under load and thermal strains without load. The transient thermal strain curves may be determined by subtraction.

This strain occurs in the conditions of a thermal strain test under load. This type of strain is also called transient creep. However, the term creep involves strain changes

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with the time. Tests carried out at different heating rates show that transient thermal strain is more related to temperature than time.

So the term of transient thermal strains is here considered to be more suitable. The testing procedure recommendations may be found in Part 7: Transient creep for service and accident conditions (RILEM 1998).

5.2.2.4 Transient Restraint Stress Test

Restraint stress in concrete is defined as the stress resulting from dimensional constraint in the axial direction of a specimen during first time heating at a constant rate, together with any initially applied stress. The restraint stress results from maintaining the length of a concrete specimen constant during the heating process. The RILEM recommendations for this test are given in RILEM TC 200-HTC (2005).

Unlike the *steady state creep* and *relaxation tests*, where the test durations exceed practical durations of building fires, the thermal strain, thermal strain under load and transient thermal strain tests simulate the transient conditions that concrete members might experience in real fire situations. Thus, data obtained from the transient tests are relevant to assess concrete structures in fire.

5.3 Stress-Strain-Relation, Compressive Strength and Modulus of Elasticity

Klaus Pistol, Sven Huismann, Ulrich Diederichs, Pierre Pimienta, Ulla-Maija Jumppanen and Jean-Christophe Mindeguia

5.3.1 Preliminary Remarks

The characteristic parameters of stress-strain-curves at high temperatures are the compressive strength ($f_{c,\theta}$), the modulus of elasticity ($E_{c,\theta}$) and the ultimate strain ($\varepsilon_{c,\theta,ult}$). There are RILEM recommendations for the evaluation of these material properties (RILEM TC 129 MHT 1995, 2004; RILEM TC 200 HTC 2007). However, if the stress-strain-relation of a concrete at high temperatures is known, the compressive strength and the modulus of elasticity can also be determined from the measured data.

Besides, compressive strength tests can be performed under stress-rate as well as strain-rate control. The advantage of strain-rate-controlled tests is that both the ascending descending branches of the stress-strain-curve can be recorded, whereas in stress-rate-controlled tests only the ascending branch can be determined. Since strain-controlled tests are more difficult to perform, stress-rate controlled tests have

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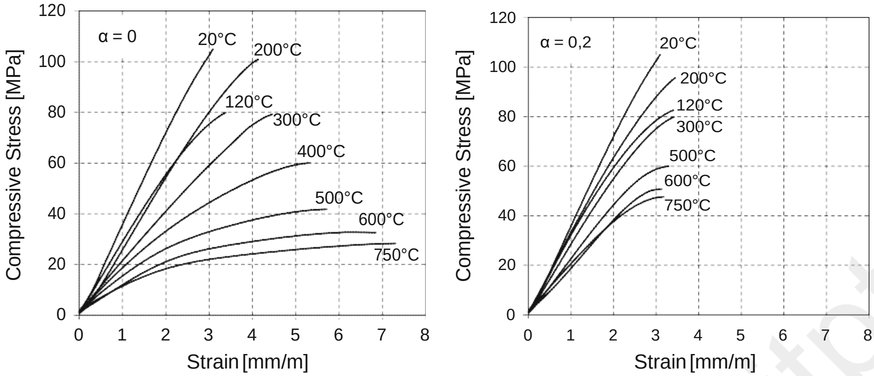

Fig. 5.1 Stress-strain curves of high strength concrete obtained in stress-rate-controlled tests (Huismann 2010)

been conducted in most cases and thus much more data of these tests are available in the literature.

Commonly, tests with unloaded specimens are reported in the literature. However, in practice structural elements are usually loaded and have to resist the fire load for a defined time. It is well known from the investigations of the mechanical behaviour of ordinary concrete (OC) that specimens which are loaded during heating have a higher strength (lower strength loss) at high temperatures (e.g. Schneider et al. 1977; Schneider 1985). If specimens are loaded in thermo-mechanical tests during heating, the stress level is characterised by $\alpha = \sigma_c/f_c$. Figure 5.1 shows results of stress-rate-controlled tests on high performance concrete (HPC).

In the left diagram, stress-strain curves of unloaded specimens are depicted, whereas the right diagram shows stress-strain curves of specimens which were loaded with a stress level of 20% ($\alpha = 0.2$) during heating to the target temperature and tested afterwards. By comparing these stress-strain curves it can also be seen that the compressive strength of HPC measured at high temperatures is higher if specimens are loaded during thermal exposure. This holds specifically for higher temperatures. Simultaneously, the ultimate strains are significantly reduced with loaded specimens. The different behaviour of loaded and unloaded specimens can be explained by different micro-cracking during heating, which is strongly influenced by the external stresses (see Sect. 5.3.9).

In the present chapter, a literature survey of stress-strain tests of HPC at high temperatures is given. Besides stress-strain relations the derived compressive strength and the modulus of elasticity are presented. Ultimate strains of HPC at high temperatures are not included in this report, because there are hardly any explicit data available. In the following “hot” properties and significant influences on them are presented and afterwards compared with “residual” properties.

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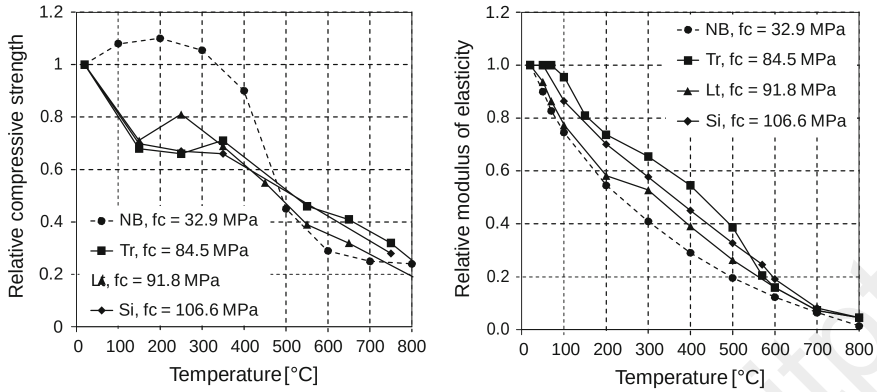

Fig. 5.2 Relative compressive strength (left) and modulus of elasticity (right) of normal and high strength concrete according to Diederichs et al. (1989)

5.3.2 Influence of the Initial Compressive Strength

The strength and the deformation capability of HPC at high temperatures depend naturally on the initial compressive strength measured at ambient temperatures (20 °C). According to the results reported by Diederichs et al. (1989) (Fig. 5.2) and by Pimienta and Hager (2002) (Fig. 5.3), HPC shows a higher strength loss than OC in the temperature range between 100 and 450 °C. Diederichs et al. explain this behaviour with the different content of paste in the various concretes. At ambient temperatures the internal stress response to an external load is uniformly distributed and the fine mortar matrix as well as the coarse aggregates are loaded with the same stress level, due to the similar mechanical properties (modulus of elasticity and strength). Thermal exposure leads to a weakening of the fine mortar structure due to physico-chemical reactions. In turn a redistribution of the internal stresses occurs so that the stress is now concentrated on the coarse aggregates alone and the strength of the whole body is significantly reduced. With OC the latter situation is already at hand at normal temperatures, because the modulus of elasticity and the strength of the fine mortar matrix are much lower than those of the aggregates. In these concretes a weakening of the fine mortar structure due to heating alters the internal stress distribution only slightly and consequently, only marginally reduces the strength of the composite material. However, the results by Phan and Carino (2001) (Fig. 5.4) do not show such a strong influence on the compressive strength. Phan's data show hardly any influence of the initial compressive strength on the modulus of elasticity at high temperatures, in contrast to Diederichs et al. (1989). However, the results from Diederichs et al. (1989) (Fig. 5.2) as well as from Pimienta and Hager (2002) (Fig. 5.3) reveal a slightly stronger loss in the modulus of elasticity at concrete with lower compressive strength.

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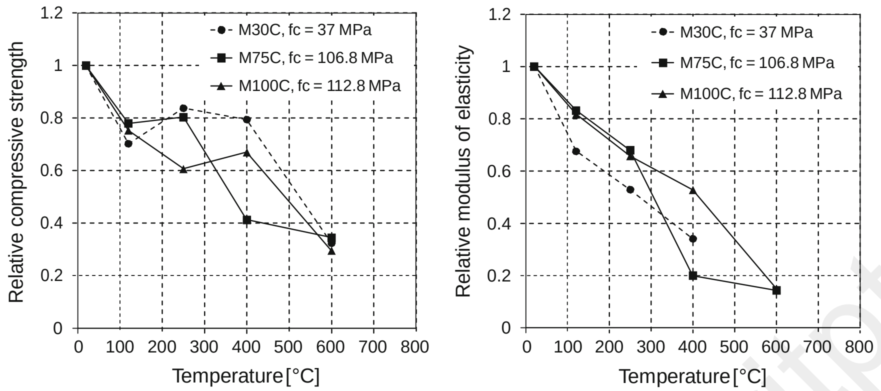

Fig. 5.3 Relative compressive strength (left) and modulus of elasticity (right) of normal and high strength concrete according to Pimienta and Hager (2002)

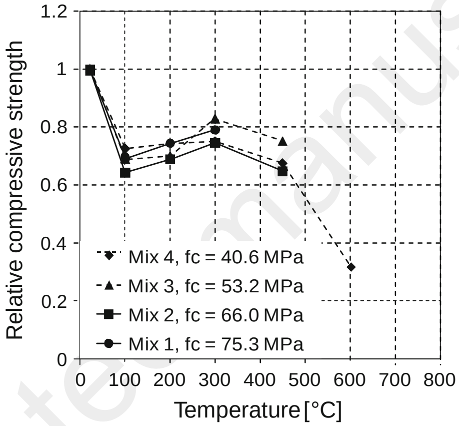

Fig. 5.4 Relative compressive strength of different concretes according to Phan and Carino (2001)

5.3.3 Influence of the Type of Aggregate

Above 400 °C, the compressive strength of HPC seems to be significantly affected by the aggregates. Thus, Pimienta and Hager (2002) (Fig. 5.5) found, that HPC with silico-calcareous aggregates containing a large fraction of flint have a higher loss of compressive strength than concrete made with calcareous aggregates. These observations coincide with those of Abrams (1971) gathered with OC. In the whole temperature range, the modulus of elasticity of concrete made with silico-calcareous aggregates is less than that of concrete with calcareous aggregates (Fig. 5.5).

In contrast, the results of Cheng et al. (2004) (Fig. 5.6) do not show any significant influence of the mineral type of aggregates on the compressive strength and modulus of elasticity of HPC. However, it has to be mentioned that these values have been obtained with specimens which were tested after cooling.

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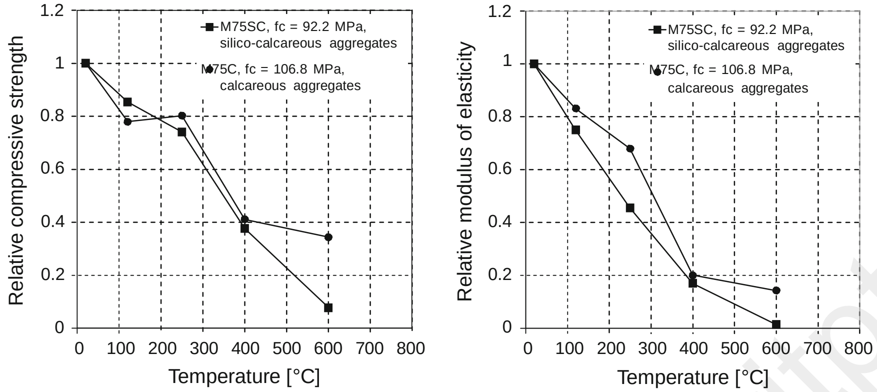

Fig. 5.5 Relative compressive strength (left) and relative modulus of elasticity (right) of two high strength concretes made with different aggregates according to Pimienta and Hager (2002) ("hot" tests)

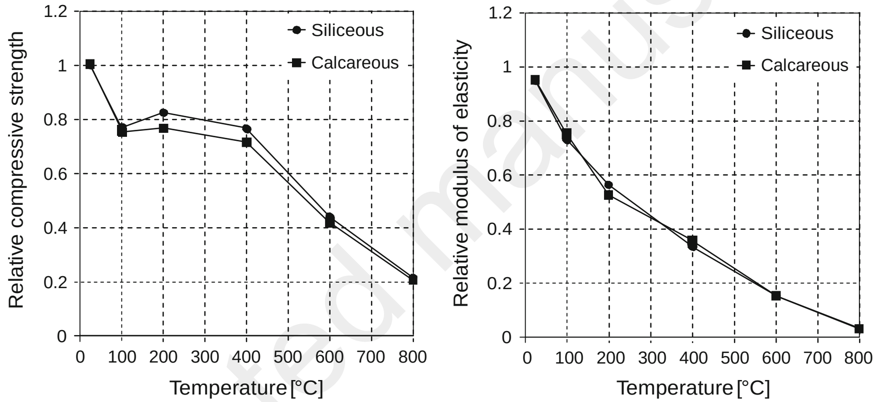

Fig. 5.6 Residual relative compressive strength (left) and relative modulus of elasticity (right) of two high strength concretes with aggregates according to Cheng et al. (2004) (residual tests)

5.3.4 *Influence of Additional Binders*

From the investigations of Diederichs et al. (1989) (Fig. 5.2) no significant differences between additional binders with respect to the compressive strength can be stated. Only the concrete containing fly ash has a higher compressive strength in the temperature region around 250 °C. The same tendency could be found in the relative modulus of elasticity. There seems to be a link between the porosity of the tested concrete and the temperature induced reduction of the stiffness.

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5.3.5 *Influence of PP-Fibres Additions*

Concerning the influence of PP-fibre additions on the compressive strength of HPC at high temperatures, different results are reported in the literature. Hager and Pimienta (2004a) (Fig. 5.7) found that even a small amount of PP fibres (0.9 kg/m^3) leads to an increase of the compressive strength at temperatures around 250 °C. However, the cement content of the HPC with PP-fibres was 10% higher than of that without PP-fibres addition. Horvath et al. (2004) (Fig. 5.8) found that a PP-fibres content higher than 2 kg/m^3 reduces the compressive strength at ambient temperatures significantly. However, the qualitative shape of the strength curves is nearly the same for all fibre contents.

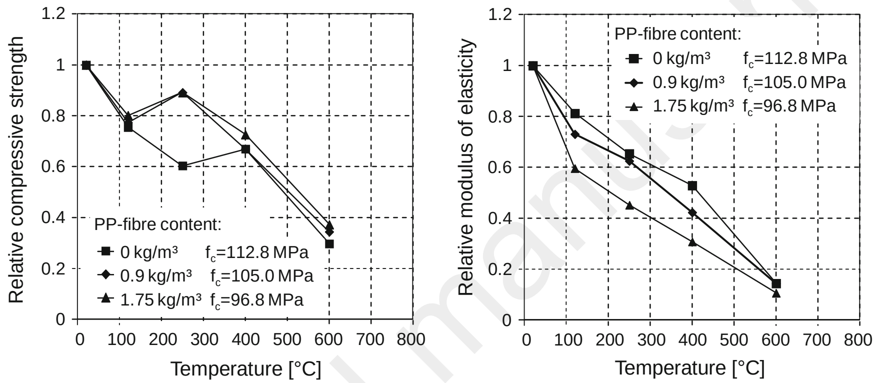

Fig. 5.7 Relative compressive strength (left) and modulus of elasticity (right) of high strength concrete made with different amounts of PP-fibres according to Hager and Pimienta (2004a)

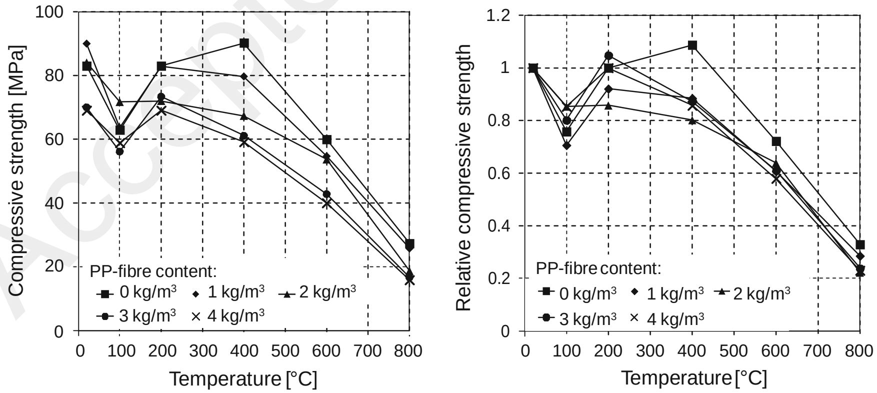

Fig. 5.8 Compressive strength (left) and relative compressive strength (right) of HPC made with different amounts of PP-fibres according to Horvath et al. (2004)

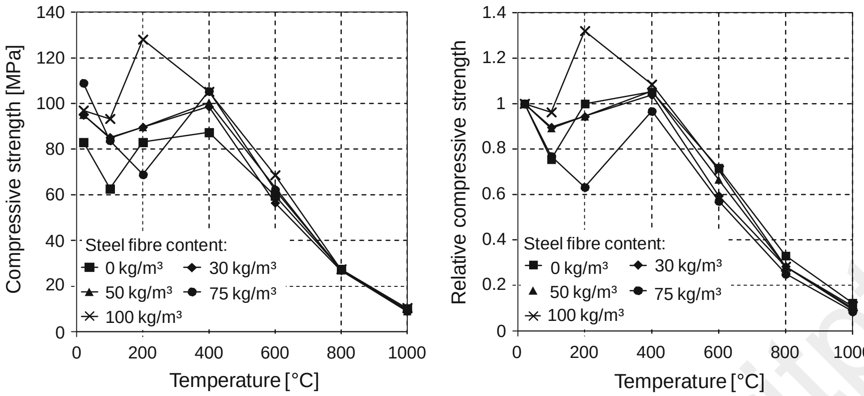

Fig. 5.9 Compressive strength (left) and relative compressive strength (right) of high strength concretes made with different amounts of steel fibres according to Horvath et al. (2004)

The highest temperature induced strength loss was observed in this study for a PP fibre content of 2 kg/m^3. The modulus of elasticity is also affected by PP fibre additives. The results of Hager and Pimienta (2004a) showed that the higher the PP-fibre content the higher is the temperature induced loss in the modulus of elasticity, especially in the temperature range between 120 and 400 °C.

5.3.6 Influence of the Steel Fibres

It is known, that steel fibres increase the compressive strength of HPC at ambient temperatures (Fig. 5.9) to a very little extent. At high temperatures different results have been found in the literature. Horvath et al. (2004) observed an influence mainly at temperatures around 200 °C while the results from Huismann et al. (2007) (Fig. 5.10) indicate an increase in the relative compressive strength at temperatures of 300 °C and higher. However, Cheng et al. (2004) (Fig. 5.11) did not find any significant influence of the steel fibre addition in either the modulus of elasticity or the compressive strength. For this reason, no general statement can be made concerning the influence of steel fibres on the compressive strength of HPC at high temperatures.

5.3.7 Influence of Moisture Content (Drying at 105 °C) and Type of Curing

Due to the high density of HPC the physically bound water evaporates very slowly out of the specimens. Huismann et al. (2010) found that up to temperatures of 300 °C, the physically bound water is not completely evaporated out of common HPC specimens.

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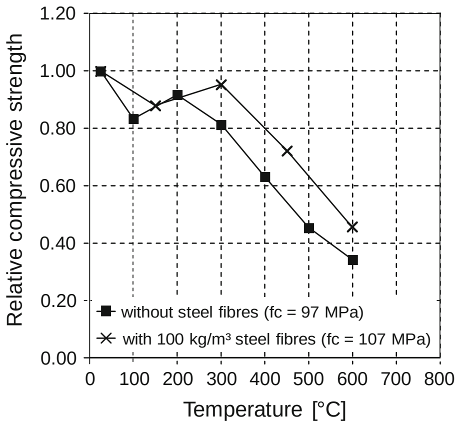

Fig. 5.10 Compressive strength of HPC with and without steel fibres according to Huismann et al. (2007)

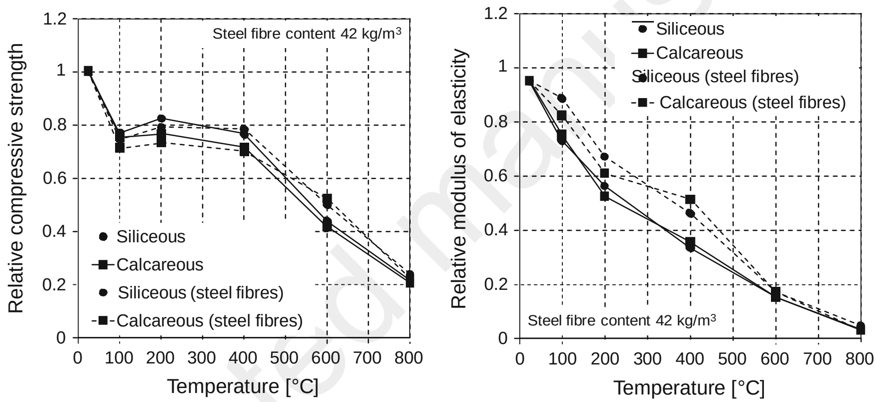

Fig. 5.11 Relative compressive strength and modulus of elasticity of HPC with and without steel fibres and different types of aggregates according to Cheng et al. (2004)

If PP-fibres are added the mass loss is slightly faster and the physically bound water is already liberated at about 250 °C. The consequence for lower temperatures is an increasing vapour pressure in the concrete specimen. Due to thermodynamic equilibrium the water moves into the finer gel pores and moistens the inner concrete surfaces. The result is a reduction of the surface energy and thus a reduction of stress levels, which led to failure. For this reason Hager (2004, c) (Fig. 5.12) found a lower strength loss if the specimens were preheated at 105 °C because the free and physically bound water already escaped prior to the test. This effect is also known for sealed concrete specimens made of OC.

In the case of sealing no water can evaporate out of the specimen, which leads to a much more dramatic strength loss, see e.g. Lankard et al. (1971).

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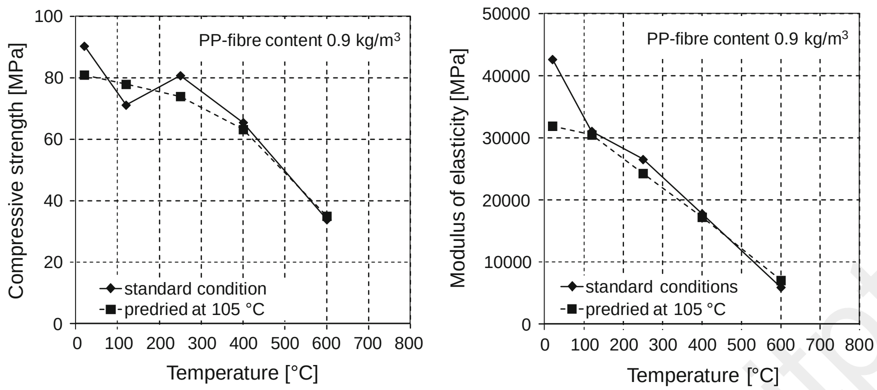

Fig. 5.12 Compressive strength (left) and modulus of elasticity (right) of high strength concrete with PP fibres for different preconditioning according to Hager (2004, c)

Preheating at 105 °C leads to drying and shrinking of the paste. This causes micro cracks, which are responsible for the dramatic drop of the modulus of elasticity from 43,000 to 32,000 MPa as shown in Hager (2004) (Fig. 5.12—right). Above the pre-drying temperature (105 °C), the run of the modulus of elasticity curve is the same for dried and non-dried specimens, because the moisture contents and losses are the same for both types of specimens.

5.3.8 Influence of Water to Cement Ratio

The w/c-ratio affects the compressive strength and modulus of elasticity at ambient temperatures. The influence of w/c-ratio at high temperatures has been investigated by Hager and Pimienta (2004b) (Fig. 5.13). Their results show that the effect of the w/c ratio on the compressive strength is limited to temperatures of max. 120 °C, but the respective results do not exceed the usual scattering of strength values. The relative modulus of elasticity is only marginally affected by the w/c ratio.

5.3.9 Influence of the Mechanical Load During Heating

The mechanical load of a specimen during heating is often given as the load level α, which is the ratio of the compressive stress during heating ($\sigma_{c,\theta}$) and the initial compressive strength (f_c). As already mentioned in the preliminary remarks, the temperature induced development of the compressive strength is strongly dependent on the load level during heating. This has been shown by Huismann et al. (2009) (Fig. 5.14) and by Diederichs et al. (2009) (Fig. 5.15).

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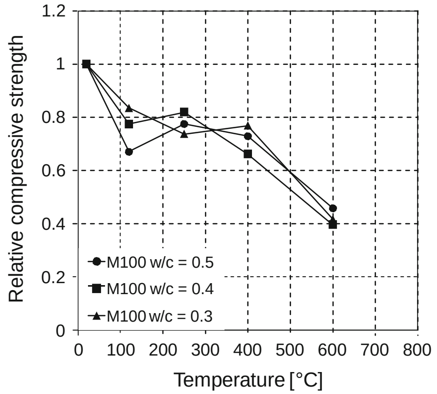

Fig. 5.13 Compressive strength (left) and modulus of elasticity (right) of HPC made with different w/c-ratios (but same mix composition) according to Hager and Pimienta (2004b)

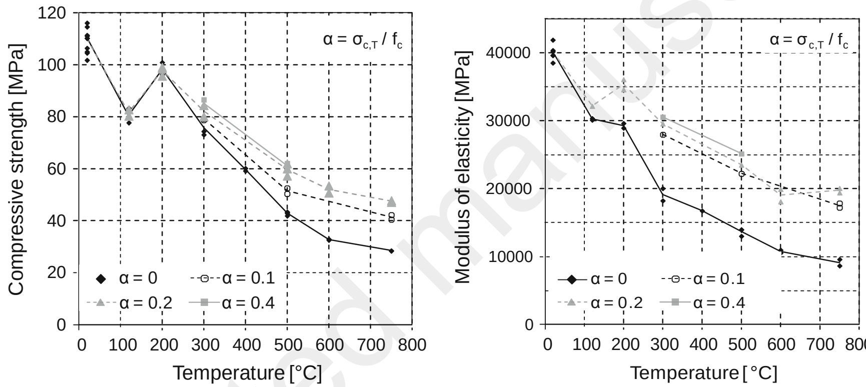

Fig. 5.14 Compressive strength (left) and modulus of elasticity (right) of one high strength concrete with different load levels during heating according to Huismann et al. (2009)

The underlying mechanism was investigated and verified e.g. by Huismann (2010). It seems to be as follows: the mechanical load prevents the cracking perpendicular to the applied load, therefore the cracks are mainly orientated parallel to the external load. This means that the number of slip planes is reduced, which leads to a strength loss. For this reason, the effect becomes significant as soon as cracking becomes more pronounced. This starts at about 250 °C. A further consequence is that the strength is only affected up to a load level between 0.2 and 0.3. If the load level is higher (above 0.4) an additional damage due to the mechanical load occurs during heating and leads to failure of the specimen, see e.g. Huismann (2010), Schneider and Diederichs (1981a, b). For OC, the influence of the respective mechanisms and results have already been reported by Abrams (1971) and Schneider et al. (1977).

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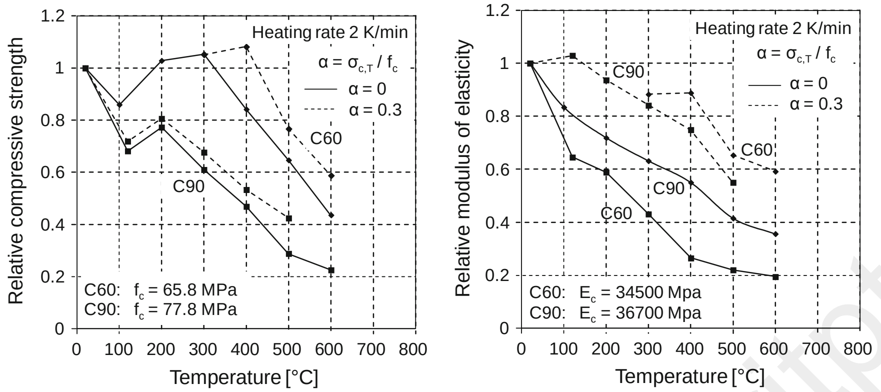

Fig. 5.15 Relative compressive strength (left) and relative modulus of elasticity (right) of two high strength concretes with two different load levels during heating according to Diederichs et al. (2009)

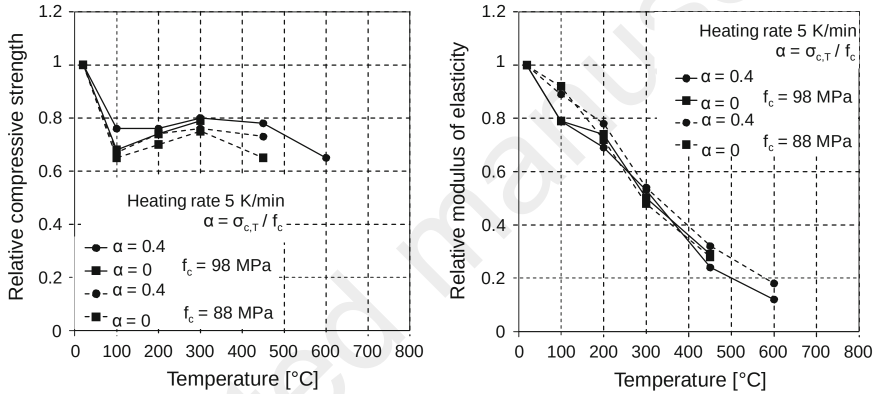

Fig. 5.16 Relative compressive strength (left) and relative modulus of elasticity (right) of two HPC with different load levels during heating according to Phan and Carino (2001)

The results of Phan and Carino (2001) (Fig. 5.16) and Khoury (1999) (Fig. 5.17) do not show such effects of the load level on the temperature induced strength loss.

The modulus of elasticity is also strongly influenced by the load during heating. However, the load already reduces the loss in the modulus of elasticity at about 100 °C. It is caused by the densification of the concretes microstructure as well as the reduced crack formation perpendicular to the stress direction. At temperatures below 250 °C the dehydration plays a major role whereas at higher temperatures the reduction of cracking perpendicular to the external load becomes more important, see Huismann (2010).

However, only Phan and Carino (2001) (Fig. 5.16) did not find any significant effect of the load during heating on the temperature dependence of the compressive strength or the modulus of elasticity.

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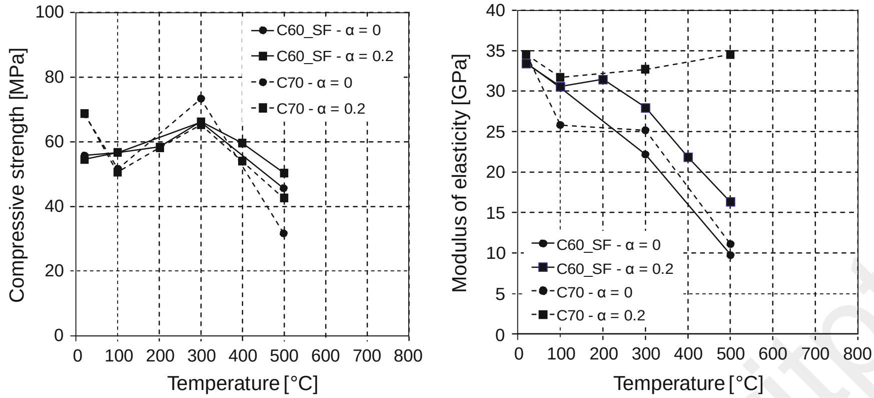

Fig. 5.17 Compressive strength (left) and modulus of elasticity (right) of two HPC with two different load levels during heating according to Khoury (1999)

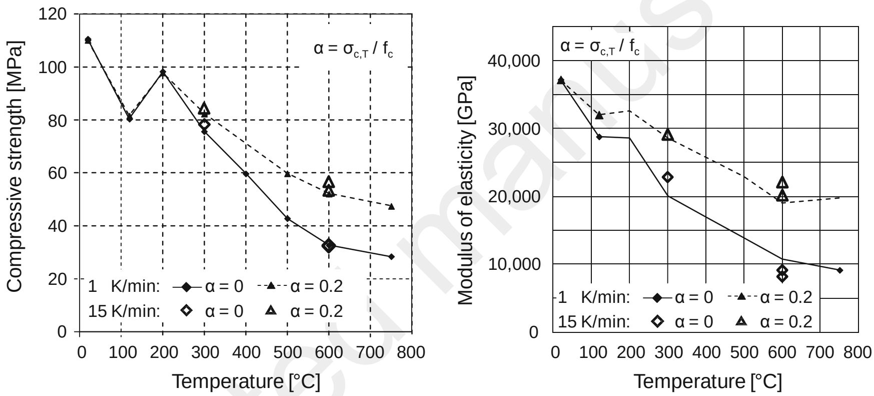

Fig. 5.18 Compressive strength (left) and modulus of elasticity (right) of HPC with two different load levels and heating rates according to Huismann (2010)

5.3.10 Influence of the Heating Rate

In the RILEM Recommendations (RILEM) the heating rate is limited to avoid unacceptable temperature gradients over the cross-section of heated specimens. High internal stresses should be avoided by these means, although it was proven by Huismann (2010) that a heating rate of 15 °K/min causes additional cracking. But the influence on the compressive strength and the modulus of elasticity at high temperatures is not significant (Fig. 5.18). This result is only based on one single study and cannot be generalised presently.

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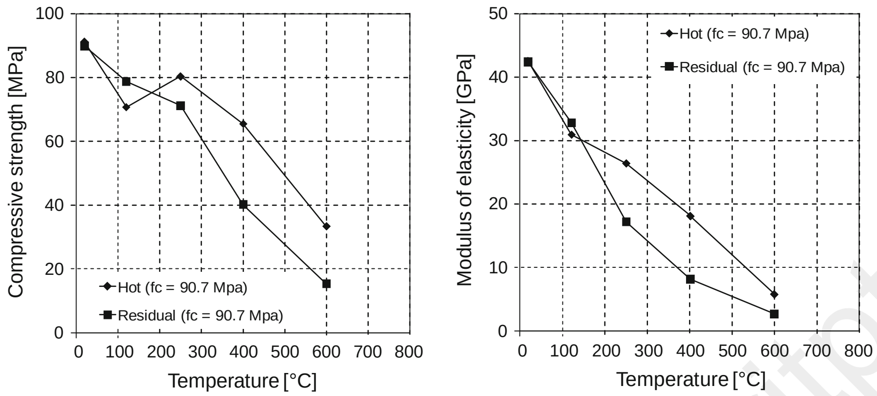
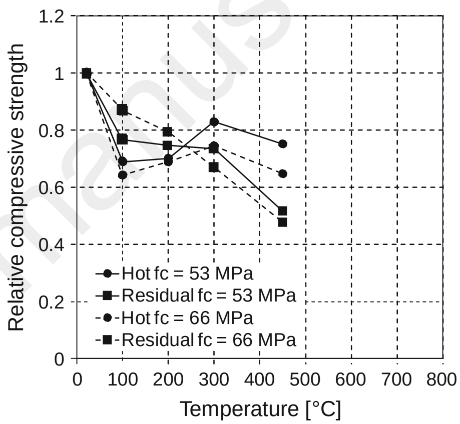


Fig. 5.19 Comparison of the "hot" and "residual" compressive strength (left) and modulus of elasticity (right) of high strength concrete according to Hager and Pimienta (2004a, b)

Fig. 5.20 Comparison of the "hot" and "residual" compressive strength (left) and modulus of elasticity (right) of HPC according to Phan and Carino (2001)



5.3.11 Comparison of "Hot" Versus "Residual" Properties

According to the results of Hager and Pimienta (2004a, b) (Fig. 5.19) it can be stated that at each temperature below 250 °C the compressive strength is lower at "hot" state than after cooling. For temperatures of 250 °C and higher the opposite holds. This means the "hot" compressive strength is higher than the "residual" compressive strength. These observations were also gathered by Phan and Carino (2001) (Fig. 5.20) and many others (see e.g. Huismann 2008; Khoury 1999).

The influence of steel fibres has been investigated by Horvath et al. (2004) (Fig. 5.21). The results indicate that in the temperature region above 300 °C the "hot" strength of steel fibres reinforced concrete remains a little bit higher than the residual strength. HPC made with PP-fibres shows a slight reduction of strength (5–15%) at 100 °C. At higher temperature (200 °C) some kind of strength recovery

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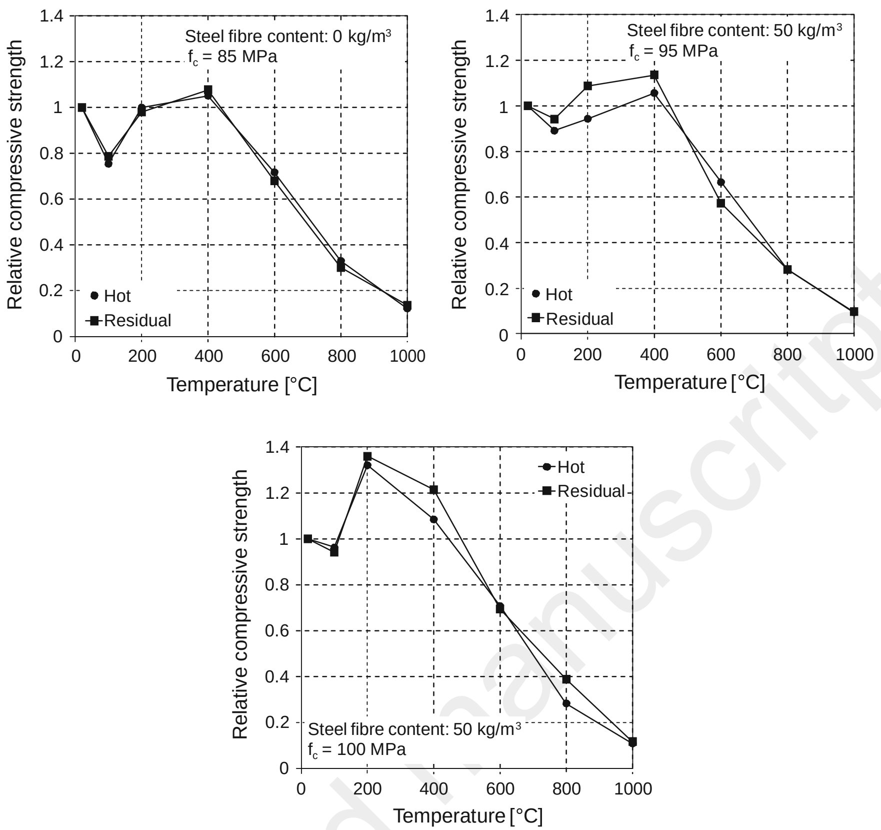

Fig. 5.21 Comparison of the "hot" and "residual" relative compressive strength of HPC with different amounts of steel fibres according to Horvath et al. (2004)

(15–25%) occurs (Fig. 5.22). The physical mechanisms behind this behaviour are still under discussion.

From the results of Khoury (1999) (Fig. 5.23) it can be stated that at temperatures above 300 °C the residual compressive strength is higher compared to the hot strength when the specimens are loaded during heating and cooling. For this behaviour, the same mechanisms are acting as described in Sect. 5.3.9.

In the whole temperature range the residual modulus of elasticity is higher with specimens which are loaded during heating than those, which are heated without load.

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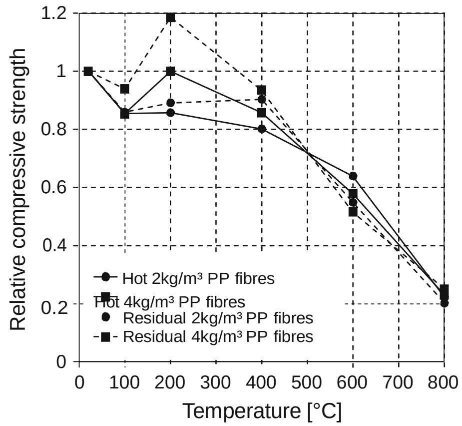

Fig. 5.22 Comparison of the "hot" and "residual" relative compressive strength of HPC with different amounts of PP-fibres according to Horvath et al. (2004)

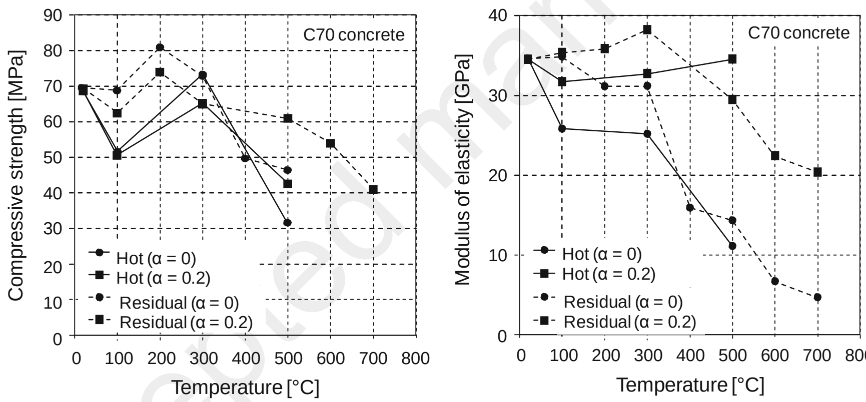

Fig. 5.23 Comparison of the "hot" and "residual" compressive strength (left) and modulus of elasticity (right) of HPC with and without a load during heating according to Khoury (1999)

5.4 Steady State Creep and Creep Recovery

Pierre Pimienta, Izabela Hager, Ulrich Diederichs and Jean-Christophe Mindeguia

This paragraph deals with the strains of high performance concrete under thermal steady states. It is worth noting that studies on the behaviour of concretes under thermal steady states were mostly done when considering concrete structures of

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nuclear reactors. For this reason, the majority of the tests were made at temperatures lower than 200 °C (Schneider 1982). It should be emphasised that a high number of thermal steady state tests have been performed on ordinary concretes, but very few were performed on high performance concrete.

In a fire situation, the thermal state of concrete is not steady: initially loaded elements are gradually heated and physico-chemical reactions (water departure, CSH dehydration, portlandite decomposition) take place, which may induce strain modifications. Mechanical behaviour of concrete under transient heating is presented in Sect. 5.5.

However, carrying out tests in thermal steady state also contributes to the understanding of the mechanisms of deformation at high temperature, and the determination of their respective contributions. For this reason, the results of tests in thermal steady state may also have an interest for the study of the concrete fire behaviour.

Steady state creep is tested in steady state temperature conditions. The specimen is heated to the target temperature with a constant heating rate, slow enough to reduce temperature gradients within the specimen (RILEM TC 129-MHT 2000a, b, c). Once the target temperature is reached, temperature is maintained until a homogenous temperature distribution in the whole sample is achieved. Another assumption made when those tests are performed is that all the physico-chemical reactions that occur at given temperature have been completed. Strain measurement starts after this temperature stabilisation phase.

An important aspect when carrying out thermal steady state tests, is the sealing of the sample. As it is described in Sect. 5.2, drying is prevented for a sealed sample but possible for a non-sealed sample.

For a sealed specimen, when a sustained mechanical load is applied (initial stress ratio α_i) after the temperature stabilisation phase, the measured strain corresponds to the *basic creep*.

For a non-sealed specimen, when a sustained mechanical load is applied (initial stress ratio α_i) after the temperature stabilisation phase, the measured strain corresponds to the *drying creep*.

The *recovery strain* tests consist of measuring the strain at the end of creep test, i.e. instantly after the unloading of the specimen and after a certain time.

If no mechanical load is applied during a thermal steady state test on a non-sealed sample, the measured strain corresponds to the *drying shrinkage*.

5.4.1 Basic Creep and Drying Creep

In the research presented in Schneider (1982) *basic creep* and *drying creep* tests were performed up to a temperature of 120 °C. Sample sealing was obtained by the application of saturated air conditions during the test. According to Acker and Ulm (2001), Benboudjema and Torrenti (2008) these conditions correspond to those occurring in massive concrete reactor vessels walls. The general trend observed (Fig. 5.24) is that the creep values increase with the increase of temperature (Schneider 1982). It can

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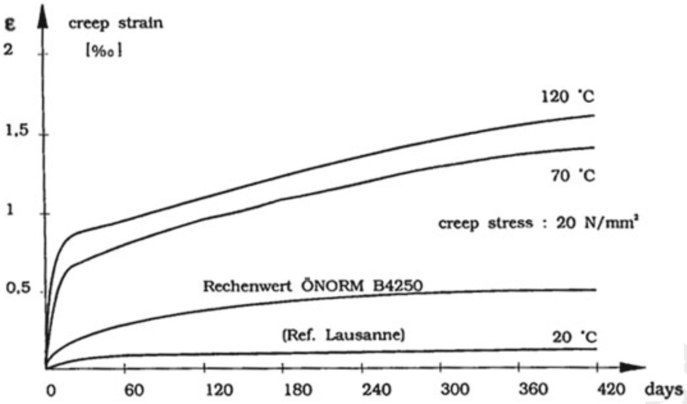

Fig. 5.24 Basic creep of high performance basalt concrete; samples tested in autoclave (concrete tested at age of 90 days) (Schneider 1982)

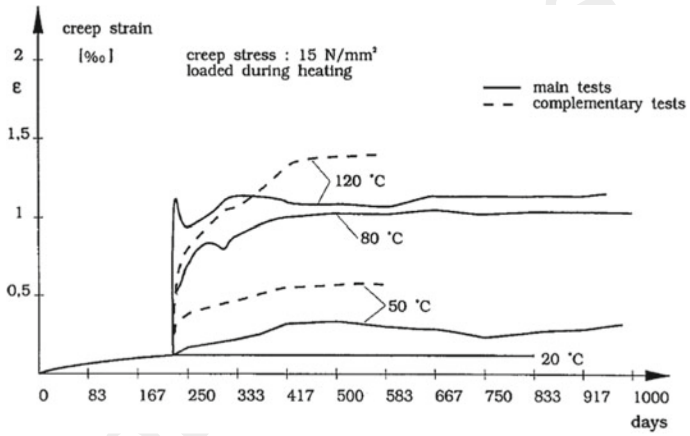

Fig. 5.25 Drying creep of HP basalt concrete, non-sealed samples (concrete tested at age of 3.5 years, and 90 days—complementary test) (Schneider 1982)

be also observed that basic creep at 120 °C is higher than drying creep: 0.24% against 0.14% respectively. The author also indicates that the age of concrete at the moment of testing influences the creep development. The drying creep measured at 120 °C on 90 days old concrete samples reached the value of 0.14% while creep of 3.5 year old concrete reached the value of 0.10%.

Basic creep tests at higher temperatures are quite rare, because of problems in the implementation of such tests (i.e. the difficulty in sealing the specimen to prevent moisture loss during the test). Figs. 5.25 and 5.26 illustrate the different behaviours observed between the two types of tests.

In the research program made by Wu et al. (2010), a high performance concrete (f_{c28} = 84 MPa) with addition of polypropylene fibres (2 kg/m^3) was tested. Three

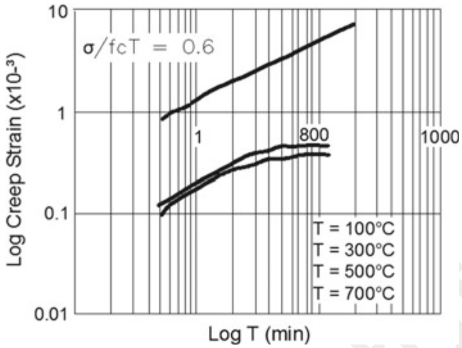

Fig. 5.26 Markedly different response of drying creep of HPC (84 MPa) with PP fibres at different temperatures. Load level of 0.6. The higher curve coresponds to 700 °C (Wu et al. 2010)

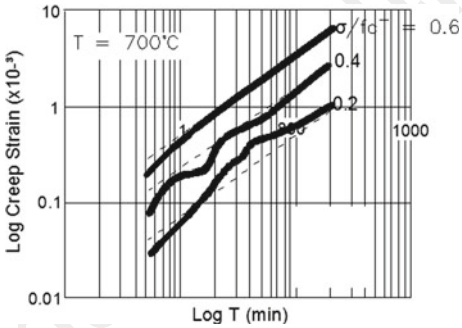

Fig. 5.27 Effect of stress ratio on drying creep of HPC (84 MPa) with PP fibres at 700 °C. Load levels of 0.2, 0.4, and 0.6 (Wu et al. 2010)

load levels, 0.2, 0.4 and 0.6 of initial strength respectively were applied after the temperature equilibrium in the specimens was reached. In this study, drying creep was measured on cylindrical specimens, diameter Ø75 mm, length 150 mm, heated to the temperatures 100, 300, 500 and 700 °C. As reported by the authors an increase in temperature and the increase in stress ratios are the main factors that increase the creep values. The creep was observed to be activated during the first 60 min after the load was applied. Afterwards the creep slows down. However, at this first stage of testing almost 80% of the total creep strain value was reached. Values of creep at 700 °C were markedly higher than those observed for lower temperature levels 100, 300, and 500 °C. At the same stress ratio creep at 700 °C was 10 times greater than creep that developed at 500 °C (Fig. 5.26). This finding suggests that the temperature of portlandite decomposition is the limit of structural usefulness of Portland cement based concrete. Moreover, it was reported that with the increase of the stress ratio the creep strain rate abruptly increases (Fig. 5.27). The authors have compared their creep results at stress level 0.4 with the results found in the literature concerning OC. One of the authors' conclusions was that creep of HPC is lower than for ordinary concrete at the temperatures up to 500 °C. For the temperature of 700 °C creep of HPC with PP fibres is higher than for OC.

In the research study by Mindeguia et al. (2006) and (Mindeguia 2009) tests were performed on moderately high-strength concretes (calcareous aggregates, w/c =

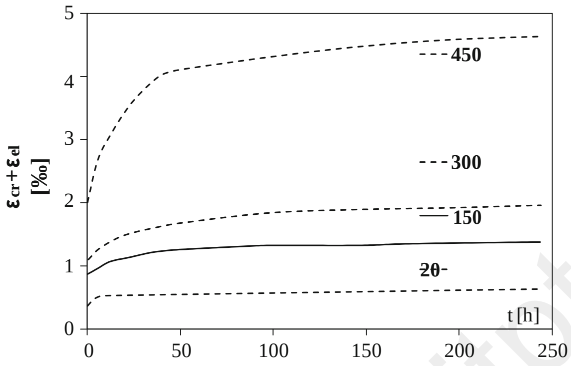

Fig. 5.28 Drying creep behaviour of concrete with quartz aggregates at 20, 150, 300 and 450 °C and load level of 0.3 by Schneider (1982)

0.54) and 28 days compressive strength of 45.4 MPa. Drying creep tests were performed on non-sealed cylindrical samples. The drying creep was measured both in longitudinal and radial directions. The results show that the drying creep does not occur in the radial direction. It was found that in the longitudinal direction creep was activated from the very first hour of the loading. The rapid creep initiation was explained by relatively high initial stress ratio of 40% of f_c. It was noted that with higher temperature, the creep activation was faster. The creep values measured at 400 °C were eight fold higher than those observed at ambient temperature. Another interesting observation was that the creep values at the temperature of 120 °C were higher than those measured at 250 °C. A similar observation of greater values of creep at 121 °C (250 °F) than those observed at 232 °C (450 °F) was reported by Nasser and Lohtia (1971) for ordinary concrete. However, the generally observed trend is that the higher the temperature is the more important creep values are, except around 250 °C. This tendency can be found also in the research study made on moderately high-strength concretes (39.8 MPa) by Schneider (1982). The results of the investigations on the drying creep behaviour of concrete with quartz aggregates are represented in Fig. 5.28. The stress ratio of 0.3 for three levels of temperature 150, 300 and 450 °C was applied on concrete specimens at the age of 1189 days.

Anderberg and Thelandersson (1976) performed drying creep tests at constant temperature levels. The material tested was moderately high-strength concretes (quartzite aggregates, w/c = 0.6) with compressive strength of 53 MPa, and the tests were performed at two stress ratios 22.5 and 45% (Figs. 5.29 and 5.30). For each temperature level unloaded specimen was tested in order to determine shrinkage. However, it was reported that no deformation of those specimens, independently of the applied temperature, was measured during the time period corresponding to the creep tests (3 h). According to the authors, creep values in the time span of 3 h could be neglected up to 400 °C. Above this temperature creep deformation is more significant. It was observed that creep values are proportional to the ratio between the applied mechanical stress and the concrete strength at each test temperature, at least up to 400 °C (see Fig. 5.31) .

Nasser and Lohita (1971) suggest that concrete creep for moderate temperatures originates in the cement paste. The adjacent CSH gel laminar particles get closer,

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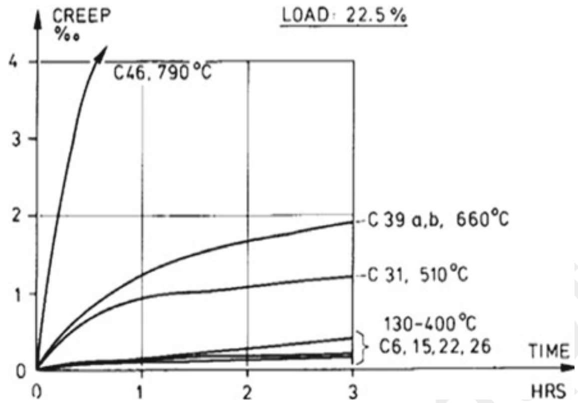

Fig. 5.29 Drying creep of concrete load level 22.5% (Anderberg and Thelandersson 1976)

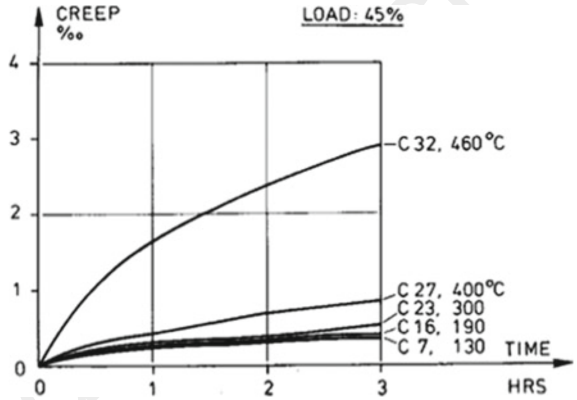

Fig. 5.30 Drying creep of concrete load level 45% (Anderberg and Thelandersson 1976)

which is facilitated by the presence of water in gaps between the particles where a mechanism similar to dislocation takes place (Ulm 1999). Higher temperatures cause acceleration of the diffusion of the water along gaps between particles (CSH sheets). At moderate temperature, below 100 °C it is also possible that further hydration of non-hydrated cement particles takes place. Above 100 °C drying of the concrete is very rapid, which is associated to the increase in the creep rate until a stable moisture condition within specimen is reached. So the size of the specimen or concrete member may be important when creep is analysed in a way that its affects the time and rate of moisture movement.

Seeing that creep of concrete originated in cement paste, drying creep tests on high performance cement paste were performed by Dias et al. (1987) and Khoury et al. (1986), see Fig. 5.32. It was observed that for higher temperatures a larger proportion of the creep takes place at an early stage (Khoury et al. 1986). This statement is valid for concrete and cement paste as can be seen in Fig. 5.32. According to Khoury et al. (1986) the creep-time relationship at high temperature can be approximated by a power law function.

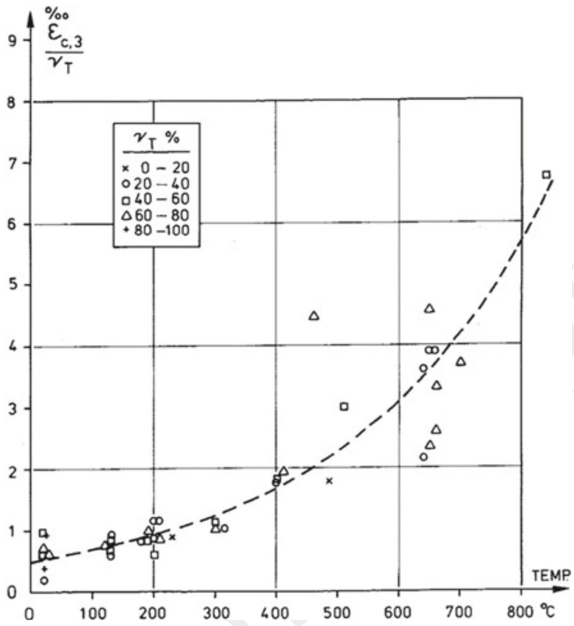

Fig. 5.31 The relationship between $\varepsilon_{c,3}/\nu_T$ and creep testing temperature, where: $\varepsilon_{c,3}$—creep at 3 h; ν_T—ratio between applied stress and concrete strength at high temperature T (Anderberg and Thelandersson 1976)

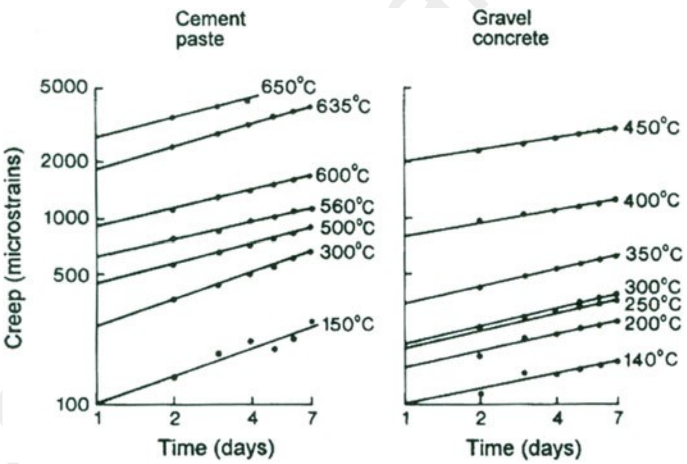

Fig. 5.32 Drying creep of high performance cement paste and gravel concrete (Khoury et al. 1986)

From the state of the art on the high temperature steady state creep, the different observations can be summarised as follows:

- steady state creep increases with the increase of temperature with the exception of temperatures close to 250 °C where lower values of creep were observed;

- for higher temperature levels, above the temperature of portlandite decomposition (500 °C), an increase of creep may be observed;
- creep strain increases proportionally to the stress level;
- for higher temperatures a larger proportion of creep takes place at early stages of testing (first few hours).

5.4.2 *Creep Recovery*

Mindeguia (2009) measured the recovery strain after unloading samples, under sustained temperature. Schneider (1982) measured the recovery strain after unloading and after cooling samples to ambient temperature. In Mindeguia (2009) the recovery strain measurements in the longitudinal and radial direction were analysed. The recovery strain was measured at 120, 250 and 400 °C during 5 h. As it was described above, in the radial direction, creep was practically not activated however radial recovery exceeded the values of longitudinal recovery, see Figs. 5.33 and 5.34. The recovery tests on HPC were also performed by Schneider (1982) but the strain was measured after unloading and the specimens had cooled down to ambient temperature, see Fig. 5.35.

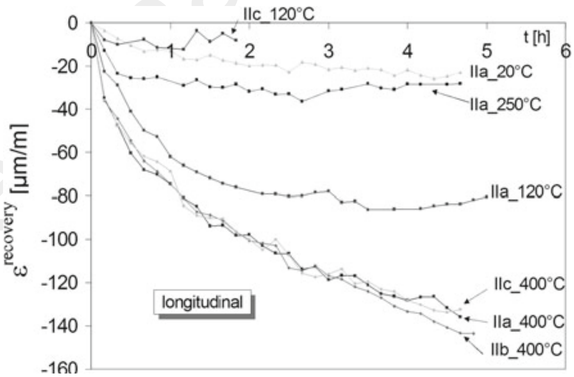

Fig. 5.33 Recovery of longitudinal creep strains (Mindeguia 2009)

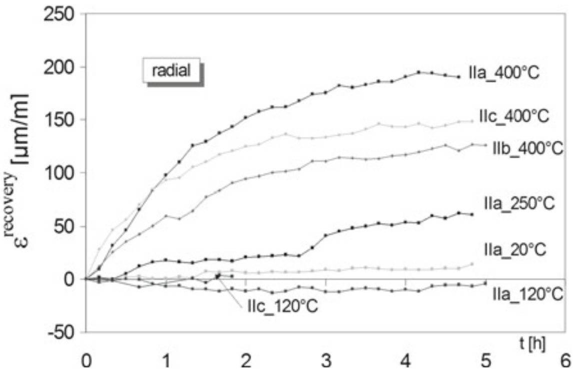

Fig. 5.34 Recovery of creep strains in radial direction (Mindeguia 2009)

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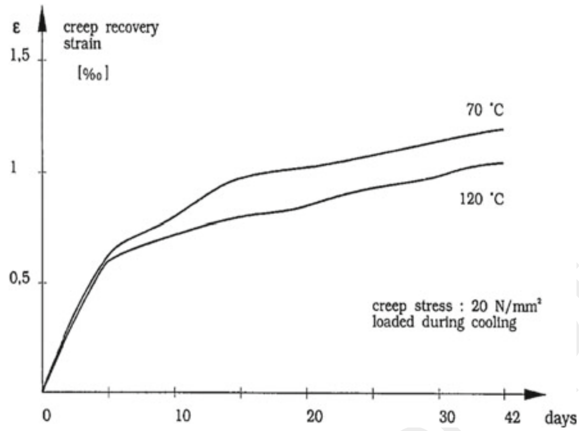

Fig. 5.35 Creep recovery strain of sealed basalt concrete samples at 20 °C (Schneider 1982)

5.5 Thermal Strain, Transient Thermal Strain and Restraint Forces

Izabela Hager, Pierre Pimienta, Ulrich Diederichs, Ulla-Maija Jumppanen, Jean-Christophe Mindeguia, Fekri Meftah and Sven Huismann

5.5.1 Thermal Strain of Concrete

Concrete, as with most materials, expands during heating. Aggregates occupy 70–80% of the volume of concrete and thus heavily influence its thermal behaviour. The main factor dominating the extent of concrete expansion is the mineralogical character of the aggregates and cement paste type. Examples of thermal expansion of aggregates can be found in Khoury et al. (1985a, b), Bažant and Kaplan (1996) (Figs. 5.36 and 5.37).

As opposed to aggregates during heating cement paste endures an important shrinkage when heated. This behaviour initiates when the temperature exceeds 150–200 °C. Thermal strain of cement paste is influenced by its mineral composition and the type of additives used in its production. The thermal strains of cement pastes modified with addition of blast furnace slag (F), silica fume (Si), fly ash (Lt) and Portland cement (OPC) are represented in Figs. 5.38 and 5.39.

The thermal expansion of the cement paste is non-linear. This non-linearity is mainly a result of thermal strain incompatibility between the shrinking cement paste and the expanding aggregates. As a result, the cement paste-aggregate bond is the weakest point in heated cementitious material.

Thermal strain of high performance concretes is a result of cement paste and aggregate behaviour during heating. Therefore, thermal strain of concrete depends

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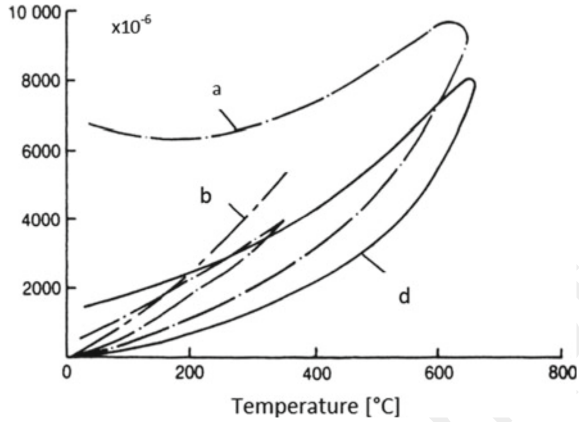


Fig. 5.36 Thermal strain of aggregates from Khoury et al. (1985a), a limestone b-gravel and d-basalt

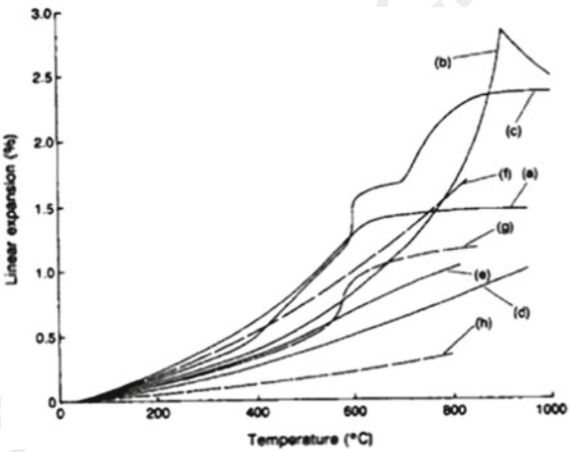
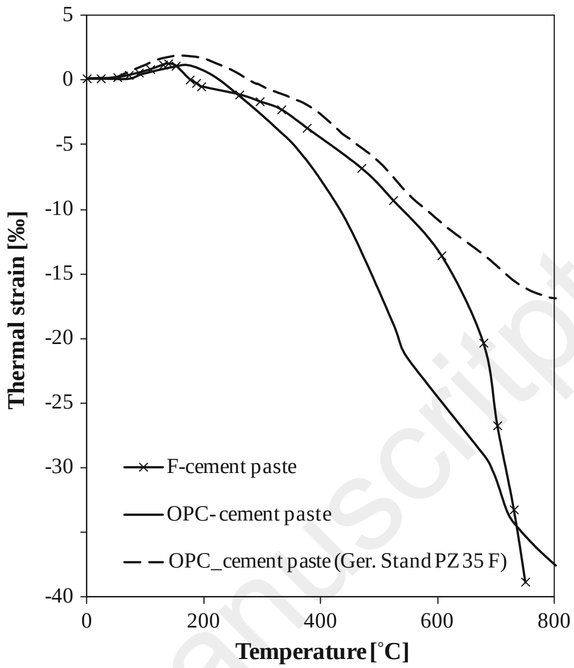


Fig. 5.37 Thermal strain of aggregates; a and g—sandstone, b and f—calcareous, c—granite, d—anorthosite, e—basalt and h—pumice (Bažant and Kaplan 1996)

largely on the aggregate type. As an example a comparison of strains in HPC made with silico–calcareous and calcareous aggregates is hereby presented (Hager 2004) in Fig. 5.40. Thermal strains of concrete with silico—calcareous aggregates significantly exceed those obtained for concrete made with calcareous aggregate (see Fig. 5.40). At a temperature of 600 °C, the strain value is twice as high compared to those measured for calcareous aggregates reaching up to 20 mm/m. This fact is associated with the higher thermal expansion coefficient of siliceous aggregates. In addition, the phase transformation of quartz, which occurs at 573 °C, is accompanied by a significant increase in aggregate volume. Higher expansion, as well as significantly higher residual strain after cooling down, was observed for concrete with siliceous aggregates, which was the result of an important number of cracks visible on the surface of samples (Fig. 5.41).

The determinant character of the aggregates controlling the thermal strain of concrete was also observed when thermal strains of different types of concretes made

Fig. 5.38 Thermal strains of Portland cement pastes (OPC) and cement paste with blast furnace slag (F) (Jumppanen et al. 1986)



with the same type of aggregates were compared. The thermal strains measured on ordinary concrete (OC) and two HPCs made with the same type and content of aggregate (69% in the case of OC and 71% in the case of two HPCs) were almost identical, despite differences in their composition, i.e. different water cement ratios (w/c), different types of cement used (Hager 2004; Mindeguia et al. 2006).

5.5.2 Transient Thermal Strain of Concrete

The strains of high performance concretes (HPC) under load during non-stationary heating at a constant heating rate combines thermal strain (ε_{th}) and the transient thermal strain (ε_{tr}). Transient thermal strain strongly influences the global structural behaviour under fire, especially for structural elements in compression (columns and walls) (Ali et al. 2001). It appears during the non-stationary heating of concrete under mechanical load (Anderberg and Thelandersson 1976; Diederichs et al. 1989; Huismann et al. 2012; Khoury 1985b; Khoury 2006; Schneider 1976; Schneider 1988). It was shown that the presence of this phenomenon could relax the compressive stresses in a column subjected to temperature (Huismann 2010; Tenchev and Purnell 2005). This strain component can reach a high magnitude. Extensive studies of this phenomenon have been conducted for ordinary concretes by Anderberg and

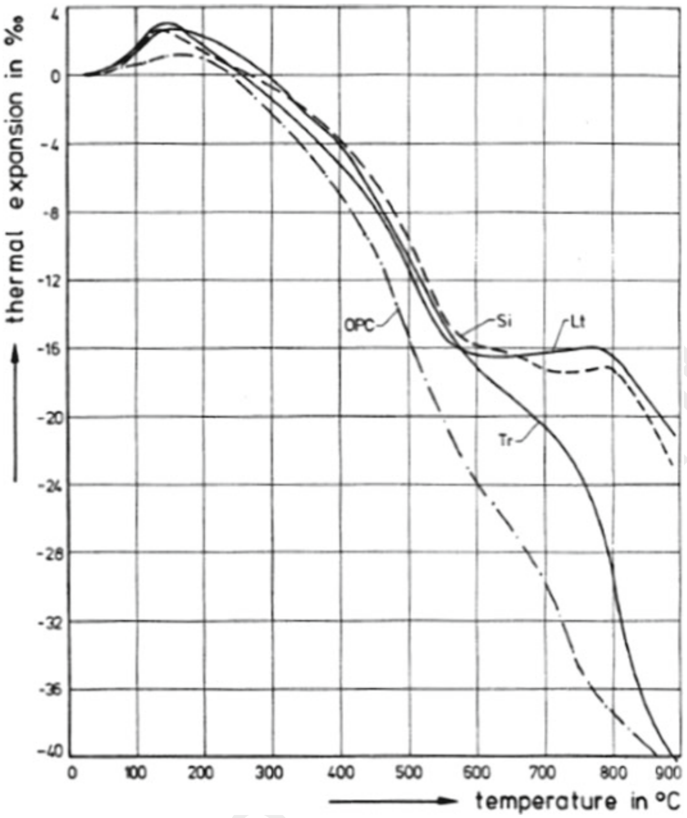


Fig. 5.39 Thermal strains of cement pastes (Diederichs et al. 1989). Silica fume (Si), fly ash (Lt), Portland cement (OPC) and blast furnace slag (Tr) cement pastes (Diederichs et al. 1989)

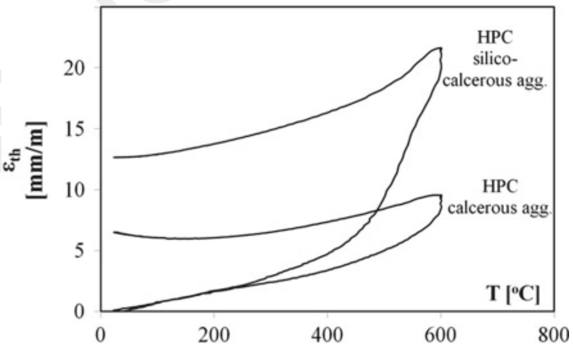


Fig. 5.40 Thermal strains of two HPCs containing silico-calcareous and calcareous aggregates. Samples heated to 600 °C and cooled down to the room temperature

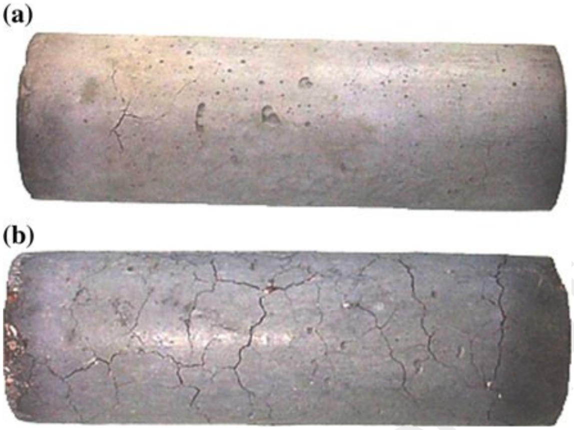


Fig. 5.41 Visible cracks on the surface of specimens after heating to 600 °C, **a** HPC with calcareous, and **b** HPC with silico-calcareous aggregates

Thelandersson (1976), Diederichs et al. (1989), Khoury et al. (1985a, b), Schneider (1976). More recent studies have been carried out, principally for high performance concretes (Hager 2004; Hager and Pimienta 2005a, b; Huismann 2010; Mindeguia 2009; Mindeguia et al. 2006; Mindeguia et al. 2013; Pimienta 2001; Pimienta and Hager 2002), in order to assess and understand contributing mechanisms of transient thermal strain activation.

It is generally agreed that the total strain of heated concrete subjected to mechanical load (ε) is equal to:

$$\varepsilon = \varepsilon_{\sigma} + \varepsilon_{th} + \varepsilon_{creep} + \varepsilon_{tr} \tag{5.5.1}$$

where:

ε_{σ} is the instantaneous stress-dependent strain,
ε_{th} is the thermal strain,
ε_{creep} is the creep strain,
ε_{tr} is the transient thermal strain, which is the topic of this chapter.

This assumption is in agreement with (EN 1992-1-2 Eurocode 2) standard recommendations.

Assuming that classical material creep ε_{creep} is negligible due to the relatively short duration of the experiment (around 10 h), Eq. (5.5.1) can be written as below:

$$\varepsilon_{tr} = \varepsilon - \varepsilon_{th} - \varepsilon_{\sigma} \tag{5.5.2}$$

The expression given by (5.5.2) allows the ε_{tr} component to be simply assessed by measuring:

- the thermal strain (ε_{th}) of the sample,
- the thermal strain under load (ε) of the sample

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Thermal strains of concrete are measured when concrete is heated with no applied load. Thermal strain of concrete is a non-linear function of temperature, strongly affected by the aggregate nature and its volume content in tested concrete (Khoury et al. 1985a, b; Schneider 1976). HPC strain during non-stationary heating depends on the aggregate type as it was explained in the previous chapter. The thermal strain measurements made on unsealed specimens during non-stationary heating include shrinkage which is unavoidable.

Thermal strains under load are measured when concrete is heated under a constant compressive load applied before a thermal load is applied. The strain measurements start when the heating begins (initial elastic strain subtraction). The load level applied ($\alpha = \sigma/\sigma_{ult}$) is usually expressed as a percentage of the compressive strength of the tested concrete at room temperature. This load level is maintained constant during heating.

Transient thermal strain is calculated by finding the difference between thermal strains and thermal strains of specimens loaded in compression. It contains all mechanically induced strain components.

The experimental parametrical studies of transient thermal strain on high performance concrete (French national project "BHP 2000" 2005; Hager 2004; Hager and Pimienta 2005a, b; Huismann 2010; Mindeguia 2009; Mindeguia et al. 2006; Mindeguia et al. 2013; Pimienta 2001; Pimienta and Hager 2002) examined the influences of several parameters: load level, original compressive strength of concrete, the nature of aggregate, fibres, the heating scenario and the initial moisture content. As the results indicate, water in cement paste plays a substantial role in the development of transient thermal strains.

The influence of the aforementioned parameters is discussed below:

Load Level

As highlighted in Diederichs et al. (1989), Khoury (1985b), Schneider (1976) and (1988), the transient thermal strains of concrete are clearly proportional to applied loads up to 30% of the initial compressive strength. A similar conclusion can be drawn from Fig. 5.43, which presents the thermal strain under load and the transient thermal strains for high strength concrete with the addition of polypropylene (PP) fibres obtained by Huismann (2010). The author uses the terms "total strain" and "mechanically induced strains" respectively for the thermal strain under load and transient thermal strains. With the increase of α, the rupture temperature decreases. For the specimen loaded to 70% of its initial compressive strength and heated with the heating rate of 1 K/min the rupture occurs at 120 °C (See Figs. 5.42 and 5.43).

Original Concrete Strength (HPC vs. OC)

Thermal strain, thermal strains under load and transient thermal strains for HPC (120.7 MPa, calcareous aggregates) and OC (39.3 MPa, calcareous aggregates) are presented and compared in Fig. 5.44 (results from Mindeguia et al. (2006), described in detail in Hager (2004), Hager and Pimienta (2005a). In the lower graphs, transient thermal strain curves are presented as a result of subtraction of thermal strain values

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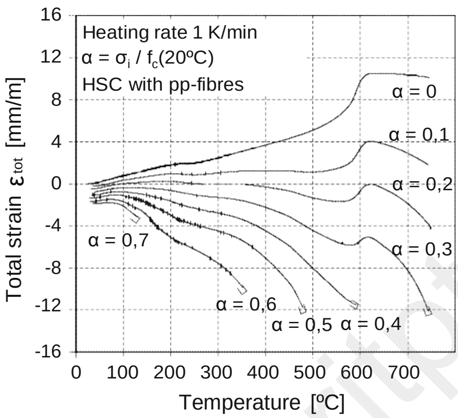

Fig. 5.42 Thermal strain under load of high strength concrete with PP fibres (Huismann 2010)

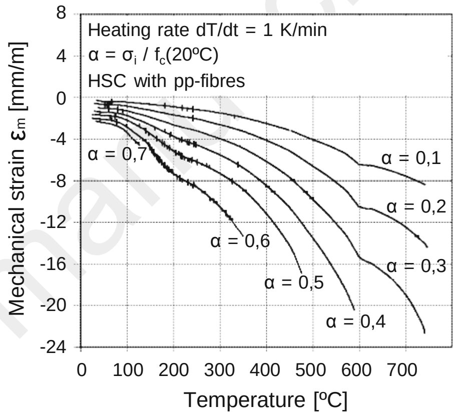

Fig. 5.43 Transient thermal strain under load of high strength concrete with PP fibres (Huismann 2010)

from the thermal strains under load. It can be observed, that thermal strains of those two concretes are close due to the type of aggregates used for both concretes. From 20 °C to about 400 °C, the thermal strain curves are almost linear. Above 400 °C strains increase more rapidly due to crack development.

It can be observed that, when the applied load level reaches 20% of the ultimate compressive strength, transient thermal strains are more important for HPC concrete. Thermal strain under load determined on OC is close to zero when α = 40% is applied (15.7 MPa). It should be noted that a similar result was obtained when load level α = 20% was applied to HPC concrete (24.1 MPa).

A comparison of transient thermal strain results obtained for two concretes differentiated by the w/c ratio under the same load levels is necessary. This would highlight the issue of the influence of material strength (or water cement ratio) on the evolution of the phenomenon in question. As noted above, the analogous results were obtained for both OC and BHP concretes for relatively similar load levels of 15.7 and 24.1 MPa (Fig. 5.44).

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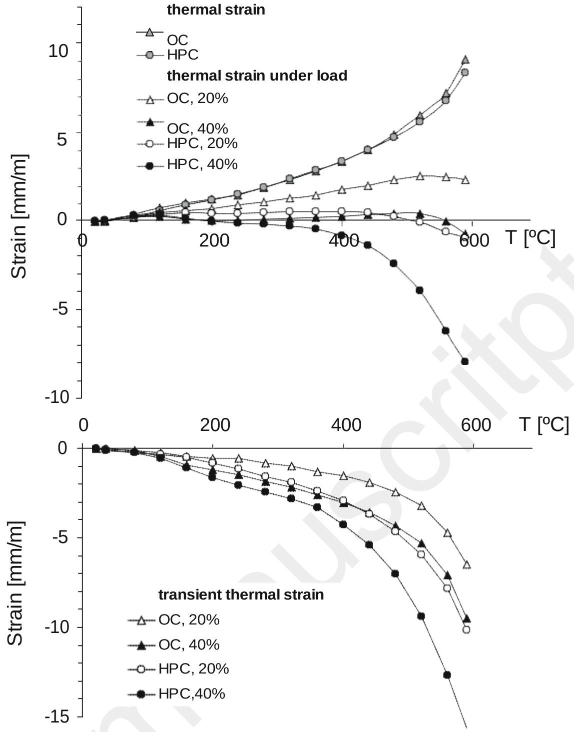


Fig. 5.44 Thermal strains, thermal strains under load and transient thermal strains for HPC and OC [results from Hager (2004), Hager and Pimienta (2005a, b)]

Influence of the Nature of the Aggregates

According to Schneider (1988) (results presented in Fig. 5.46), transient thermal strains of different types of ordinary concretes are quasi-identical up to about 450 °C. Similar conclusions can be drawn from the research done by Khoury (2006), Fig. 5.47. The transient thermal strains (LITS in his nomenclature) are similar for concretes made with different type of aggregates. The ε_{tr} curves for five concretes are alike up to 450 °C. The tested concretes had different types of aggregates (L—limestone, BI, BII—basalt, LW—lightweight, G—gravel) but with similar aggregate content; preconditioning; and most important, original compressive strength. A similar value of compressive strength induces similar values of compressive load level for all tested concretes. This led to the achievement of similarly shaped transient thermal strain curves at temperatures up to 450 °C. Above this temperature, the observed differences are due to the development of cracks (Khoury 2006; Schneider 1988).

The transient thermal strains of HP concretes with different types of aggregates as tested by Hager (2004), Hager and Pimienta (2005a, b), have similar shape, up to 300 °C. The main results obtained for HPC limestone and HPC siliceous concretes are presented in Fig. 5.45. It can be noted that transient thermal strain of HPC siliceous is approximately equal to that of the HPC calcareous up to 300 °C. For temperatures

Fig. 5.45 Thermal strains, thermal strains under load and transient thermal strains for HPC with limestone aggregate and HPC with silico-calcareous aggregate [results from Hager (2004) and Hager and Pimienta (2005a)]

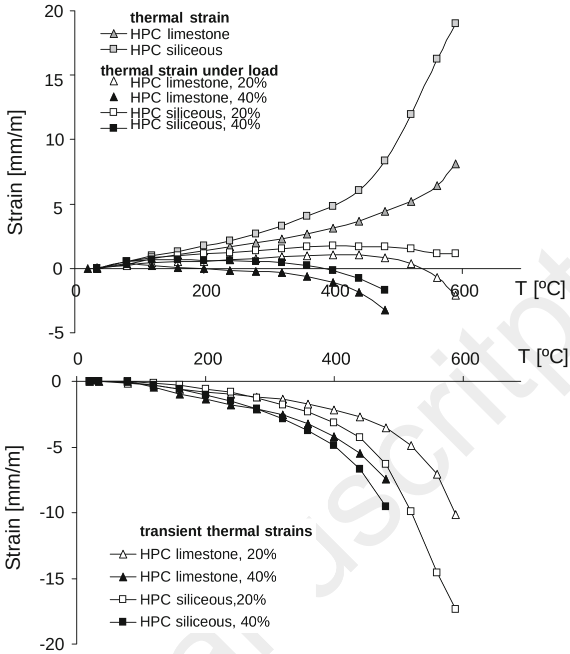
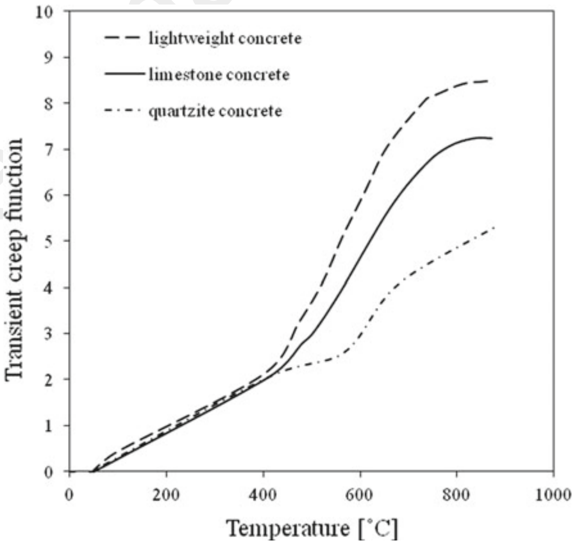


Fig. 5.46 Transient thermal strains (transient creep—Schneider's nomenclature) of concretes with different type of aggregates (Schneider 1988)



exceeding this level, it increases more rapidly for the concrete with siliceous type of aggregates (Seine gravel).

All sources cited agree that, above a definite temperature, differences in transient thermal strain evolution for different aggregate types may occur. The discrepancies

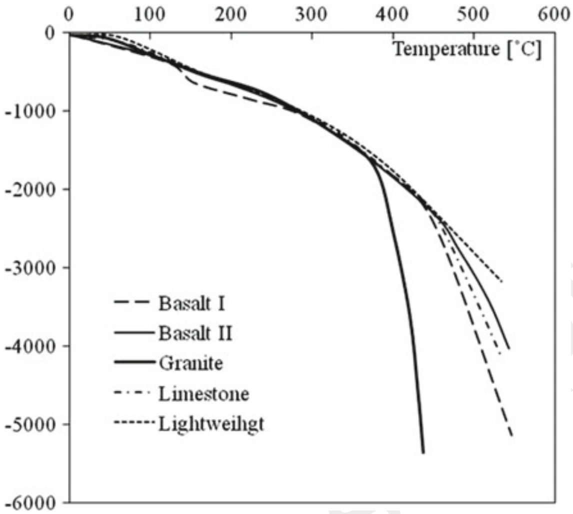


Fig. 5.47 Transient thermal strains (LITS in Khoury's nomenclature) of five concretes with different type of aggregates, measured during first heating (1 °C/min) (Khoury 2006)

in the thermal strains between expanding aggregates and shrinking cement paste are the reason for the development of cracks (Hager 2004; Hager and Pimienta 2005a, b; Khoury 2006; Schneider 1988). As a result the transient thermal strains values change.

Influence of Different Binders/Cement Paste Composition

The results presented in this section concern the comparison of transient thermal strains of concretes with different cement paste compositions (Diederichs et al. 1989). As mineral additive silica fume (Si), fly ash (Lt) and blast furnace slag (Tr) were used and compared with concrete made with Portland cement (OPC) (See Fig. 5.48). In this research concretes with mineral additives presented mechanical properties much higher than those obtained for OPC so they can be considered as high performance cementitious matrices (respectively: Si—91.8 MPa, Lt—87.3 MPa; Tr—91.4 MPa; OPC—32.9 MPa). However the comparison between those four concretes is difficult due to the differences in the mix compositions. Different aggregate types were used (HPC—granite based—sand and diabase, OC—greywacke, sandstones, quartz and quartzites) as well as different coarse aggregate and binder contents. The constant parameter for all HPC concretes was the water/binder ratio respectively equal to 0.27 compared with 0.45 for OC.

Thermal strains of all three concretes with mineral additions are very similar and reached 8‰. Ordinary concrete with Portland cement was characterised by higher values of thermal strains recorded during heating, which was attributed to the nature of aggregates used for this concrete (high quartz content). The thermal strains under higher load levels led to rupture in some cases described as violent destruction.

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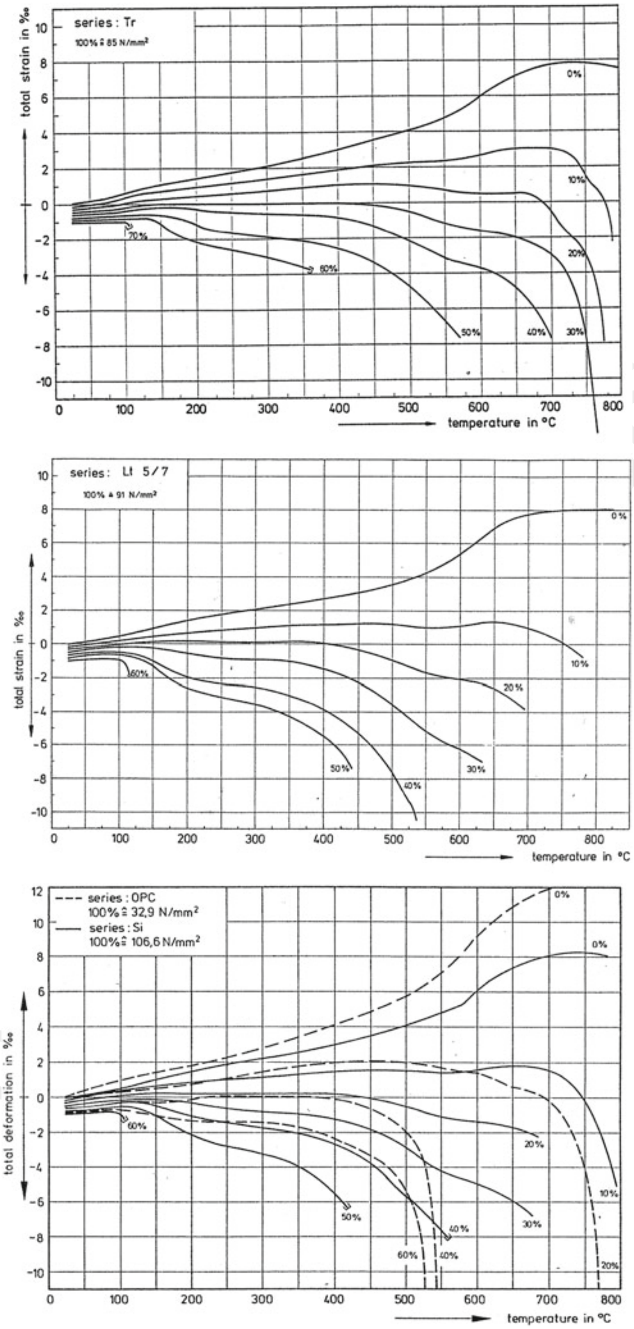
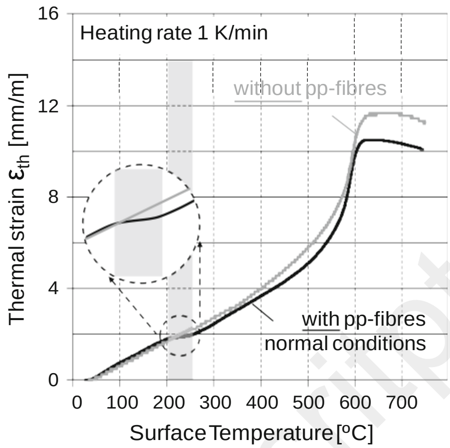


Fig. 5.48 Thermal strains and thermal strains under load of concretes made with different cement paste compositions (Diederichs et al. 1989). Fly ash (Lt), blast furnace slag (Tr), silica fume (Si), compared with ordinary Portland cement concrete

Fig. 5.49 Thermal strain of HP concrete with and without addition of PP fibres (Huismann 2010)



Influence of Polypropylene Fibres

The influence of PP fibres on thermal strains and transient thermal strain behaviour was investigated by Huismann (2010), Huismann et al. (2012). For high performance concretes with PP fibres two characteristic temperature ranges on the thermal strain curves were observed (Fig. 5.49). The first characteristic point, represented as a plateau on the thermal strain curve, for HPC containing PP fibres occurs between 200 and 250 °C. This plateau was attributed to an elevated shrinkage due to an accelerated moisture loss (RILEM Recommendations 1998). Moreover, it was observed that thermal strains of HPC were lower for HPC containing PP fibres for temperatures above 300 °C. (Huismann et al. 2012) explained this phenomenon by the appearance of homogeneous micro-cracking in the concrete with PP fibres. A homogenous porosity and micro-cracking of the cement paste surrounding the fibre beds results in a lower thermal strain of those concretes.

The influence of PP fibres on the transient thermal strain (mechanical induced strain in Huismann nomenclature (Huismann 2010) was represented in Fig. 5.50. It is important to note that, for load levels greater than 0.3 (10% of the room temperature compressive strength), spalling of specimens was observed. The spalling phenomenon occurred for the specimens with no fibres when the surface temperature reached 325 °C. By adding 2 kg/m^3 of PP fibres spalling was successfully limited even for higher load levels applied during heating. It was also observed that for HPC with PP fibres the values of transient thermal strains are higher. However, this influence becomes more pronounced when the temperature exceeds 300 °C, which was mainly explained by the appearance of cracks.

Cooling Phase

It was observed (Hager 2004; Hager and Pimienta 2005b) that transient thermal strain does not occur during the cooling phase (Fig. 5.51) and only appears during the first heating cycle. In the Fig. 5.52 the curves of thermal strain under load determined

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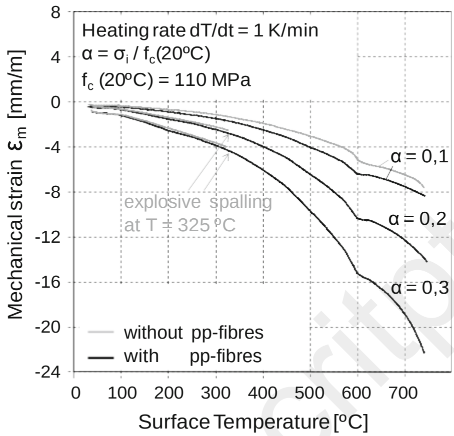

Fig. 5.50 Transient thermal strain (mechanical induced strain) of HP concrete with and without addition of PP fibres (Huismann 2010)

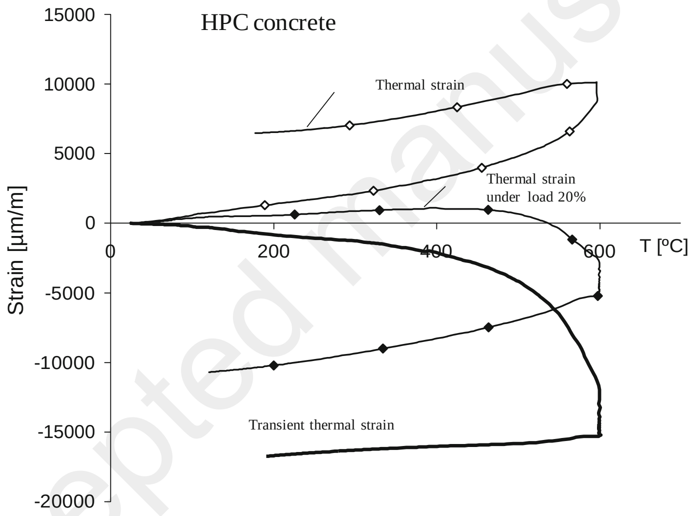

Fig. 5.51 Thermal strain, thermal strain under load and transient thermal strains of HPC sample during heating and cooling phase (Hager 2004)

on an HPC sample exposed to two heating/cooling cycles were presented. It can be observed that the transient thermal strain only appears during first heating cycle. No phenomenon is present during the cooling phase and the next heating cycle. The strains that were measured during the second cycle correspond to thermal strains. This is represented as a part of the thermal strain under load curve parallel to the thermal strain curve (Fig. 5.52). These results are in agreement with the research and findings of other researchers made on ordinary concretes (Khoury 1985b; Schneider 1976; Schneider 1988).

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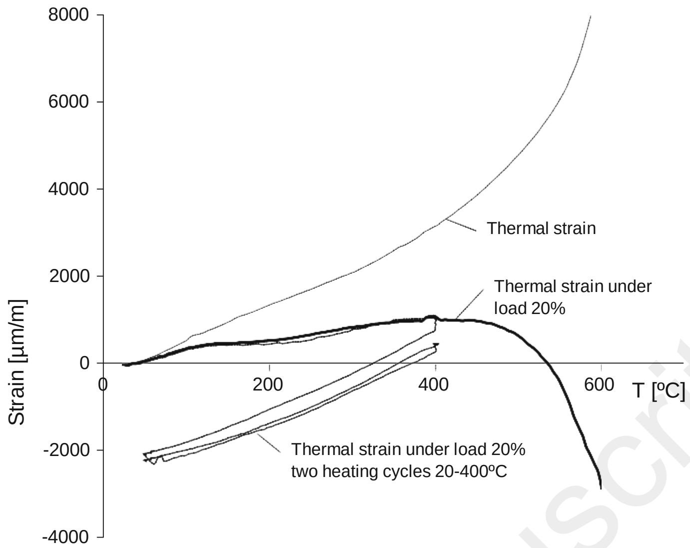

Fig. 5.52 Thermal strain under load curves of HPC sample after two cycles of heating at 400 °C (Hager 2004)

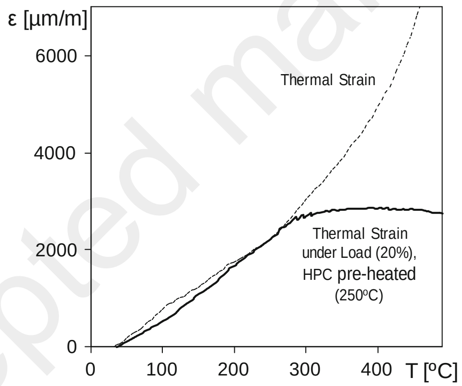

Fig. 5.53 Thermal strain and thermal strain under load of HPC in a sample pre-heated up to 250 °C, transient thermal strain is re-activated when T > 250 °C (Hager 2004)

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Influence of Heating Scenarios

It is accepted that transient thermal strain occurs in non-dried concrete subjected to heating under compressive loads (Diederichs et al. 1989; Khoury 2006; Schneider 1976, 1988). It was observed that its activation coincides with water migration and cement paste dehydration. As a consequence, the pre-heating of concrete samples at various temperatures (80, 250, 300 °C) prior to the tests significantly alters transient thermal strain development. This fact is illustrated in Figs. 5.53 and 5.54.

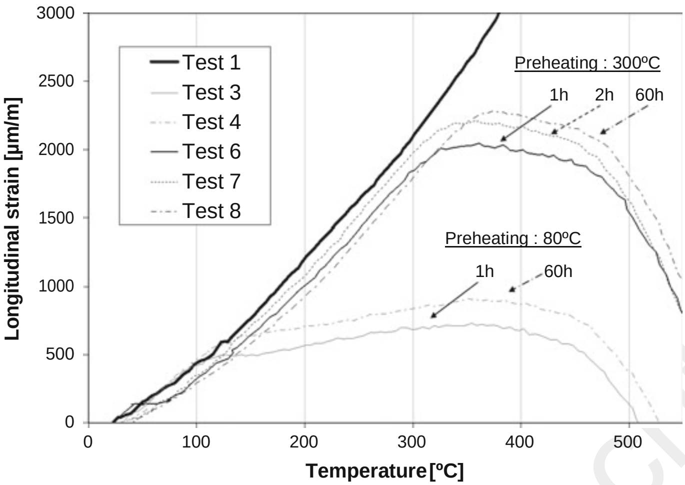

Fig. 5.54 Thermal strain and thermal strain under load of HPC samples with different states of dehydration. Influence of the pre-heating duration (Mindeguia 2009; Mindeguia et al. 2013)

The thermal strain under load of pre-heated concrete samples follows the thermal strain curve up to the pre-heating temperature (Hager 2004; Mindeguia 2009; Mindeguia et al. 2013). When analysing thermal strains under load of pre-heated HPC concretes (Figs. 5.53 and 5.54), it is observed that the curves begin to separate when the heating temperature exceeds the temperature of pre-heating. This means that the transient thermal strain of pre-heated concrete samples appears only for temperature higher than the maximum pre-heating temperature. The experimental results above suggest that transient thermal strain is generated by a thermo-hygral process (at least below 400 °C) and could be strongly linked with the dehydration processes of cement paste (Mindeguia 2009).

For the pre-heating temperatures of 80 and 300 °C, it was observed that the longer the pre-heating duration, the higher the temperature at which the thermal strain under load curves separates from the free thermal strain curve. This fact can be explained by the kinetics of C-S-H dehydration. A more significant level of dehydration is achieved when samples are pre-heated longer. The duration of the pre-heating slightly influences transient thermal strain activation for times ranging up to 60 h (Mindeguia 2009; Mindeguia et al. 2013).

It was reported that the drying of concrete samples at the conventional drying temperature of 105 °C (time of drying: approximately 120 days) may significantly influence the evolution and extent of transient thermal strains, even up to 400 °C (Hager 2004; Hager and Pimienta 2005a). These results confirm the strong link between concrete water content, dehydration and transient thermal strain development.

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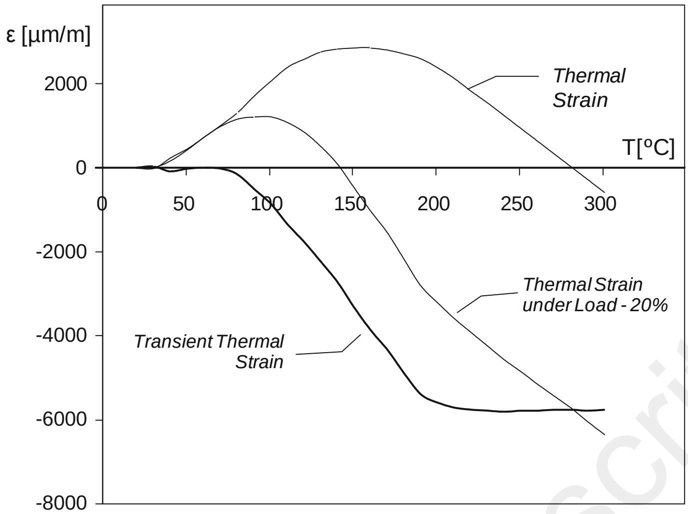

Fig. 5.55 Thermal strain, thermal strain under load and transient thermal strain for HPC cement paste (Hager 2004)

Transient Thermal Strain of Cement Paste

As previously indicated, water content in concrete strongly influences the development of the transient thermal strain. In fact, water in concrete is mainly present in the cement paste (Mindeguia et al. 2013). In order to verify that this phenomenon originates in the cement paste (C-S-H gel), tests on a pure high performance cement paste sample were carried out (Hager 2004). The cement paste mix composition was the same as the one used for HPC concrete studies carried out by Hager (2004). Figure 5.55 presents the thermal strain, thermal strains under compressive load (α = 20%) and transient thermal strains. In the first stage, the cement paste expands until T = 150 °C, then starts to shrink. The phenomenon occurs between 80 and 200 °C. Transient thermal strain values of cement paste are important and may reach the value of 6 mm/m. Such high value suggests that transient thermal strain of concrete derives from the cement paste.

Transient Thermal Strain in the Radial Direction

Research has also been carried out to elucidate the transient thermal strain under compressive presence in the radial direction It has been observed in Mindeguia (2009), Mindeguia et al. (2006), Mindeguia et al. (2007) and Mindeguia et al. (2013) that the transient thermal strain is not detectable in the radial direction. Neither the different pre-heating temperatures, nor their durations have an influence on ε_{tr} in the radial direction. Thermal strain under load curves are quasi-identical with the curve representing thermal strain up to 400 °C (Mindeguia 2009; Mindeguia et al. 2007; Mindeguia et al. 2013).

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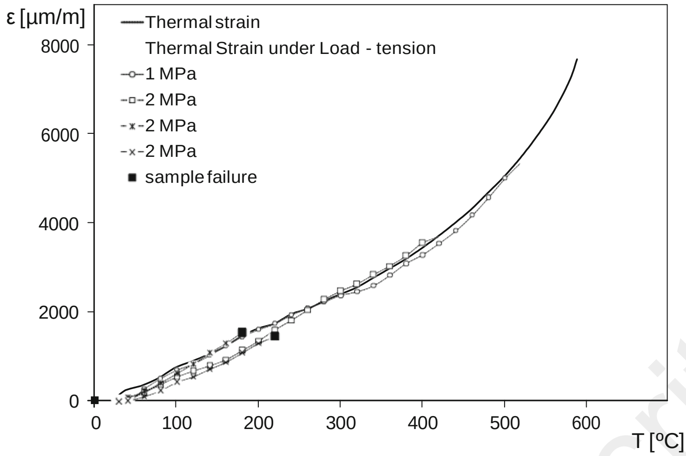

Fig. 5.56 Thermal strains and thermal strains under tensile load of HPC—now observable or absence of the transient thermal strains in tension (Mindeguia et al. 2013)

Transient Thermal Strain in Tension

The research program verifying the existence of transient thermal strains in tension for HPC was described in Mindeguia et al. (2013). Measuring the strains under direct tensile load during heating required a particular testing method. The equipment and testing procedure were described in detail in Hager (2004). The thermal strain and thermal strain under direct tensile load measurements were performed. A tensile load of 1 and 2 MPa was applied on HPC specimens heated with the constant heating rate of 1 °C/min. Figure 5.56 presents a comparison of thermal strains with thermal strains under load curves of the HPC mix. Thermal strains under tensile load are superimposed on the thermal strain curve which suggests that transient thermals strains cannot be seen in tension. Transient thermal strain under tensile load appears then to be either non-detectable or non-existent. The first case (non-detectable) could be related to the low load level (1 and 2 MPa) that can be applied without exceeding the rupture load.

Transient Thermal Strain Mechanism and Modelling

It has been shown (Diederichs et al. 1989) that transient thermal strains cannot be explained by strains resulting from the change in the modulus of elasticity of heated material. Indeed, transient thermal strains are largely higher.

Two main mechanisms are commonly presented in the literature and were summarised in Mindeguia (2009) and Mindeguia et al. (2013). Transient thermal strain is a result of two simultaneous processes acting in concrete while heated:

- **Thermo-hygral process**: at 20 °C and under sustained load, the humidity movement into the macropores and micropores of the medium accelerates the shear deformation of the C-S-H layers and involves an additional delayed strain, called

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"drying creep". This strain is also known as the "Pickett effect" (Acker and Ulm 2001; Bažant et al. 2004; Ulm 1999). It is important to note that this particular creep could appear both during the drying of concrete and during the rehumidification of the material. Drying creep is then directly influenced by water transfer whatever the direction of the movement. By heating concrete, many physico-chemical transformations take place in the cement paste, and in particular for the C-S-H, dehydration for temperatures between 105 and 300 °C (Bažant and Kaplan 1996; DeJong and Ulm 2007; Khoury 1992; Schneider and Diederichs 1981a, b). This dehydration will involve a movement of water from the C-S-H layers (initially bonded water) to the porous media. As for the case of drying creep, this water movement under sustained load will accelerate the sliding of the solid microstructure involving in this way an additional high temperature delayed strain, called "dehydration creep" in Meftah and Sabeur (2006), Sabeur and Colina (1985), Sabeur and Meftah and (2008). This dehydration creep, added to the drying creep, could be responsible for the activation of transient thermal strain.

- **Thermomechanical process**: for T > 400 °C: transient thermal strain seems to be caused by the damage induced by thermal strain mismatch between paste and aggregates. This thermo-mechanical contribution mainly depends on the nature of the aggregates and the cement paste properties (mechanical properties and thermal expansion). Due to a transient heating, damage can appear in concrete in different forms and at different scales (Schrefler et al. 2002): thermal gradient induced stresses, cement paste damage caused by dehydration, thermal strains due to mismatches between cement paste and aggregates.

By coupling these different damages, some authors succeed in simulating transient thermal strain by using macroscale models (Nechnech 2000) or mesoscale models (Mounajed et al. 2005; Menou et al. 2004; La Borderie et al. 2007). More recently in De Sa (2007), it has been shown, thanks to mesoscale simulations that thermal strain mismatch between paste and aggregates had no effect on transient thermal strain below 400 °C.

5.5.3 Restraint Forces

When heated locally, like during a fire, concrete elements expand but this movement is often restricted due to the rigidity of the surrounding elements. In this case important restraint forces may appear. Experimental investigations of restraint force evolution during heating were conducted by Diederichs et al. (1989), Schneider (1976).

Restraint forces increase rapidly after the heating starts. Depending on the load level, they increase approaching values higher than 50% of ultimate strength at 100 °C. For higher temperatures restraint forces decrease and the further evolution of restraint forces seems to be less influenced by the loading ratio. The restraint forces decrease with the increase of temperature up to 550 °C. At this temperature portlandite decomposition takes place. As it was emphasised by authors (Diederichs et al. 1989) during tests at this temperature most of the samples failed (Fig. 5.57).

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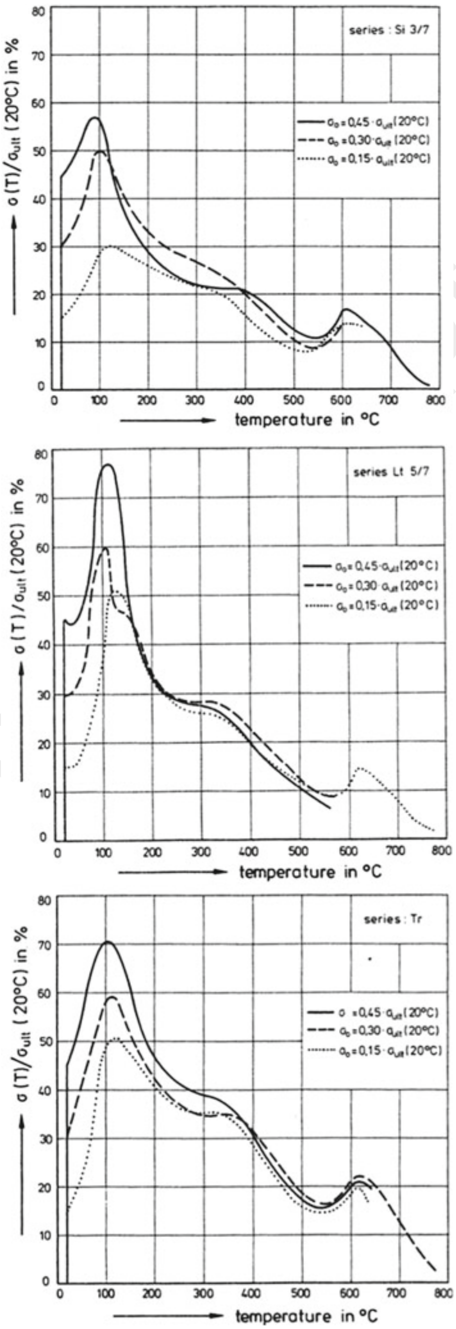

Fig. 5.57 Restrain forces of concretes made with different mineral additives to cement paste silica fume (Si), Fly ash (Lt), blast furnace slag (Tr) (Diederichs et al. 1989)

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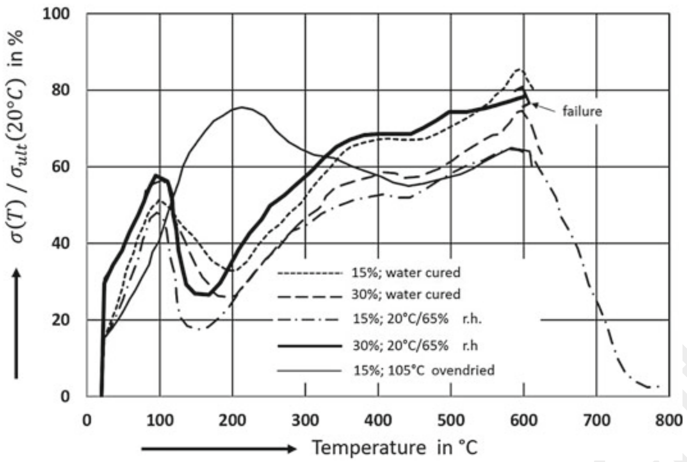

Fig. 5.58 Restrain forces of concretes made with Finnish aggregates and F-cement containing blast furnace slag (F) (Jumppanen et al. 1986)

According to Jumppanen et al. (1986) restraint forces and transient thermal strains should be considered complementary phenomena like creep and relaxation at constant temperature.

The restraint forces measured for concretes water cured, oven dried at 105 °C and stored in constant conditions of temperature and humidity (20 °C and RH = 65%) (Jumppanen et al. 1986) have revealed that restraint forces are highly sensitive to the moisture content of concrete specimens, as can be seen in Fig. 5.58.

5.6 Tensile Strength

Izabela Hager and Katarzyna Mróz

5.6.1 Introduction

The results of investigations of tensile strength (f_t) changing with temperature T are not as often presented as the results concerning compressive strength. This is mainly due to the technical difficulties encountered when testing concrete in tension. An additional difficulty is to carry out tests at high temperature (hot tested tensile strength f_t^T). Hence, most results applying to the tensile strength are obtained after cooling down the specimen to ambient temperature (residual tensile strength $f_{t,res}^T$).

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5.6.2 Testing Methods

In order to determine the changes in tensile strength resulting from heating, the RILEM recommendations were established (RILEM TC 129-MHT 2000a). In part 4 of those recommendations the “standard” tensile test was described. A direct tensile test is considered as the standard method of tensile strength determination. Cylindrical specimens should be used and the load in the longitudinal direction is applied by gluing them to the end plates of the testing machine or by clamping. When a clamping technique is used the concrete specimens are shaped in the form of dumbbells in order to allow the load application.

Another testing option is to include a notch in the middle of the specimen to initiate the crack propagation. However, (RILEM TC 129-MHT 2000a) considers this option as a “non-standard” test condition. On the other hand, if the absence of a notch causes problems and the crack develops outside the cylindrical part of dumbbell shaped specimen, then it may be desirable to include a notch. Notched specimens are commonly used for fracture mechanics parameter determination and were employed by Felicetti and Gambarova (1998), Felicetti et al. (2000) for tensile strength, fracture energy and characteristic length determination of heated HPC (Felicetti and Gambarova 1998; Felicetti et al. 2000).

5.6.2.1 Direct Tension

In the literature the number of the results concerning high performance concrete tests in tension and its mechanical property evolution with temperature is limited. Direct tension tests on HPC on glued or clamped specimens, were performed on hot specimens by Felicetti et al. (2000) as well as by Hager (2004). The direct test results obtained after cooling down the specimens to ambient temperature may be found in Noumowé et al. (1996), Felicetti and Gambarova (1998) and Felicetti et al. (2000). In Noumowé et al. (1996) specimens on HPC and OC concretes with square 10 × 10 cm cross sections were used enabling the comparison of the behaviour of those two concretes in tension. The results of direct tension presented in Felicetti and Gambarova (1998), Felicetti et al. (2000) consisted of testing clamped specimens with circular cross sections (Ø100 × 300 mm), while (Hager 2004) presents the tests investigated using cylindrical specimens (Ø100 × 600 m). In Fig. 5.59. the temperature effects on relative tensile strength values, obtained using direct methods is presented. Additionally, in Fig. 5.59. a curve showing tensile strength changes as a function of temperature, recommended by (EUROCODE 2) is shown. Starting from a temperature of 100 °C, (EUROCODE 2) expects a drop in tensile strength reaching 20% per 100 °C.

According to Noumowé et al. (1996) HPC behaves better in tension than OC because the average rates of decrease of concrete tensile strength were lower for HPC. The rate of decrease of tensile strength were 8.4%/100°C and 10.5%/100°C for HPC and OC respectively. The results obtained by Hager (2004) indicate the increase

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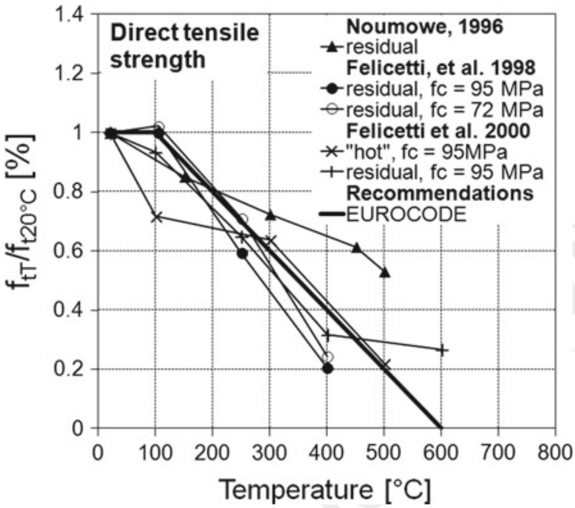

Fig. 5.59 Relative direct tensile strength test results

of the tensile strength with temperature in the range 20–400 °C. Such behaviour has been explained by the drying processes that increase the strength of concrete. From the results presenting direct hot and residual results (Felicetti and Gambarova 1998), it can also be concluded that values obtained for hot specimens are lower than those obtained after cooling them down (Felicetti et al. 2000).

5.6.2.2 Indirect Tension

In the literature several studies on the temperature effect on tensile strength are presented employing indirect tension test methods. Felicetti and Gambarova (1998) presented tests based on the bending of HPC concrete prisms (beams), with and without notches, both hot and after cooling specimens down. The tensile strength was also investigated using a splitting test method by Noumowé et al. (1996), Zhang et al. (2000) and Hager et al. (2014).

In the research presented in Zhang et al. (2000), the tensile strength of HPC heated with three different heating rates 1, 3 and 10 °C/min was compared. The main conclusion issuing from this research was that practically no influence on residual tensile strength was observed when those three different heating rate were employed. The rate of decrease of concrete tensile strength tested by Zhang et al. (2000) was about 10%/100 °C.

In Hager et al. (2014) the results of residual splitting tests performed on HPC concretes with different type of aggregates were presented. In all concretes the cement paste volume was the same and its composition was identical in all series. The only difference was the aggregate type: granite, dolomite, basalt and riverbed gravel.

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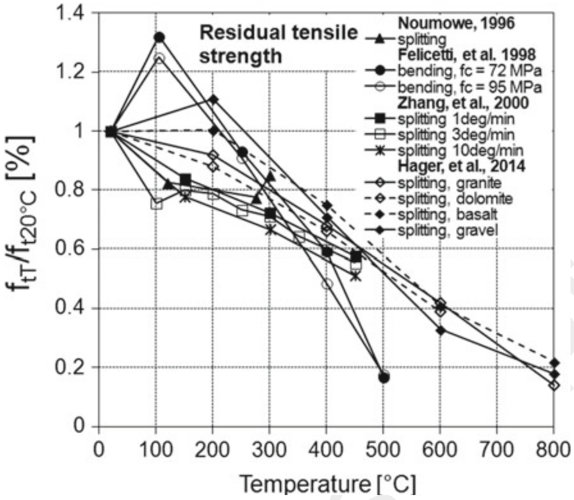

Fig. 5.60 Relative indirect tensile strength test results

According to Hager et al. (2014), the type of aggregate affects tensile strength only for the temperature of 200 °C. Above that temperature, the reductions of tensile strength are quasi-linear and similar to each other. Up to the temperature of 200 °C in all tested cases, the splitting residual tensile strength slightly increases for basalt and gravel concretes and slightly decreases for granite and dolomite concretes. Above 400 °C, there is no significant influence of aggregate type on tensile strength of concrete. In the results presented by Hager et al. (2014), the slope of the tensile strength decrease was higher than in case of other researchers' results (about 14%/100 °C).

Figure 5.60 presents the effects of temperature on relative tensile strength, obtained using indirect methods: bending and splitting tests (Noumowé et al. 1996; Felicetti and Gambarova 1998; Zhang et al. 2000; Hager et al. 2014)

5.6.2.3 Comparison of Testing Methods

Since there are several methods available that allow assessment of concrete tensile strength (direct method, splitting, bending, etc.), obtaining different results is expected. The results presented in Noumowé et al. (1996), Felicetti and Gambarova (1998), allow the comparison of residual tensile strength test results obtained on the same mixtures of HPC using direct methods (clamping) and indirect methods (splitting or bending). The results confirmed that the method significantly influences the obtained results, see Fig. 5.61. Simultaneously, it is clear that the residual tensile strengths obtained by the use of direct methods are always lower values than those obtained in indirect methods. The differences are considerable: 12% in the case of the splitting method (Noumowé et al. 1996) and about 20% in case of the 3-point bending method (Felicetti and Gambarova 1998).

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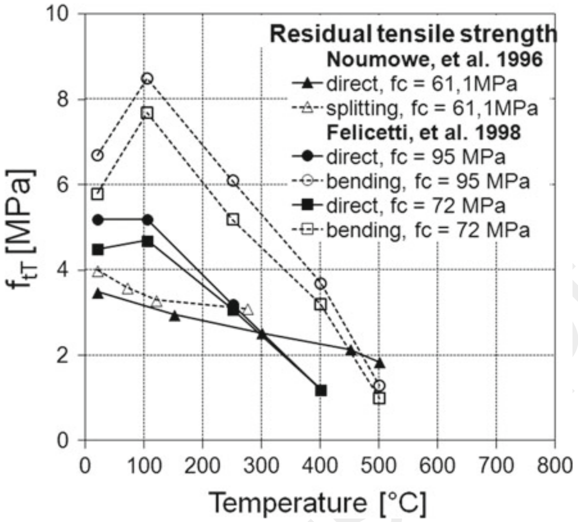
Fig. 5.61 The results of tensile strength obtained with different test methods



5.6.2.4 Additional Remarks

In Felicetti and Gambarova (1998) and Chan et al. (1996) it is also highlighted that the drop in the concrete tensile strength with temperature is more significant than the drop in compressive strength. Tensile strength in concrete seems therefore to be a more sensitive property to high temperature than compressive strength. It is explained by the creation of micro- and macro-cracks in the specimens subjected to high temperature, which affects more the tensile strength than the compressive strength of concrete.

Tensile strength evolution as a function of temperature seems to be important from the point of view of understanding the spalling phenomenon. This type of instability may be related to internal stresses generated by steam pressure and thermal strains and according to some researchers (Khoury 2008) the spalling phenomenon occurs when those stresses exceed the value of concrete tensile strength.

5.6.3 Conclusions

The very few cited results indicate that there is only a little attention paid to High Performance Concrete behaviour in tension, whether it is direct or indirect tension (in bending or splitting). It is also worth noting that there is only one study available on tensile strength tests investigated on hot samples (Felicetti et al. 2000; Hager 2004). A research program concerning the effect of high temperature on the tensile strength of HPC is therefore highly desired.

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Moreover, it should be noted that in the light of the RILEM recommendations (RILEM TC 129-MHT 2000a) all implemented testing methods presented in this State-of-the-Art report should be considered as "non-standard".

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