

Report on Early-Age Cracking: Causes, Measurement, and Mitigation

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Report on Early-Age Cracking: Causes, Measurement, and Mitigation

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Report on Early-Age Cracking: Causes, Measurement, and Mitigation

Reported by ACI Committee 231

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Early-age cracking is a challenge for the concrete industry. Materials selection, environmental conditions, and field practices all have considerable influence on the propensity for early-age cracking to occur. This document focuses on thermal- and moisture-related deformations; both are materials-related and contribute to early-age cracking. The document provides detailed reviews on the causes of deformation and cracking, test methods for assessing shrinkage and thermal deformation properties, and mitigation strategies for reducing early-age cracking.

Keywords: autogenous shrinkage; cracking; early-age; heat of hydration; measurement; microstructure; mitigation methods; shrinkage; shrinkage cracking; sustainability; thermal cracking; thermal properties.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

ACI Committee 231 defines “early age” as the period after final setting, during which properties are changing rapidly. For a typical Type I portland-cement concrete moist cured at room temperature, this period is approximately 7 days. This document, however, includes discussions of early-age effects beyond 7 days. It is important to understand how concrete properties change with time during early ages and how different properties are interrelated, which may not be the same as for mature concrete. It is also important to understand how these early-age changes influence the properties of concrete at later ages. The temperature history at early ages has a strong effect on whether concrete may develop its potential strength. Poor early-age curing has been demonstrated to detrimentally affect the strength, serviceability, and durability.

Concrete structures change volume due to the thermal- and moisture-related changes. This may be detrimental because substantial stresses may develop when the concrete is restrained from moving freely. This is particularly important at early ages while the concrete has a low tensile strength. Therefore, the assessment and control of early-age cracking should be based on several factors, such as age-dependent material properties, thermal- and moisture-related stresses and strains, material viscoelastic behavior, restraints, and environmental exposure.

Temperature control in concrete during the early stages of hydration is essential for achieving early strength as well as ultimate strength and to eliminate or minimize uncontrolled cracking due to excessive mean peak temperature rise and thermal gradients (ACI 207.1R and 207.2R). Of particular importance in determining the risk of early-age cracking of any concrete member is an assessment of the magnitude of the stresses generated in the concrete as a result of restraint to thermally induced movement. In general, there are two types of restraint: external and internal. External restraints are caused by support conditions, contact with adjacent sections, applied load, reinforcement, and base friction in the case of concrete slabs-on-ground. Internal restraint is a manifestation of the residual stresses that develop as a result of nonlinear thermal and moisture gradients within a cross section.

New methods were developed and older methods rediscovered for evaluating stress and strain development and assessing

cracking risk in concrete mixtures under realistic exposure conditions. Categories of evaluation methods discussed in this document include restrained and unrestrained volume change tests, coefficient of thermal expansion tests, and tools for assessing stress development and cracking potential. Some of these evaluation methods have been standardized.

Mitigation methods have focused mainly on reducing the autogenous (moisture-related) component of the early-age stresses or compensating for the early-age shrinkage by employing expansive cement. In the former case, both shrinkage-reducing admixtures (SRAs) and internal curing have been demonstrated to reduce the magnitude of the early-age shrinkage of specimens cured under sealed, isothermal conditions.

The prevention or mitigation of early-age cracking will improve the long-term durability of concrete structures and, therefore, enhance their sustainability by increasing the service life.

1.2—Scope

This document reviews the causes of early-age deformation and cracking. The test methods for quantifying the early-age stress development and hence the risk of cracking due to thermal and moisture conditions are described. Mitigation methods for stress reduction are also discussed.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

C	= cement factor (content) for concrete mixture, lb/yd ³ (kg/m ³)
C_f	= correction factor accounting for the change in length of the measurement apparatus with temperature, $0.56 \times 10^{-6}/^{\circ}\text{F}$ ($1 \times 10^{-6}/^{\circ}\text{C}$)
CS	= chemical shrinkage of cement (mass of water/mass of cement)
D	= moisture diffusion coefficient of concrete
dT/dt	= temperature change
$d\epsilon_{hygral}/dt$	= rate of nonthermal deformation due to autogenous shrinkage, drying shrinkage, or both
$E(t)$	= Young’s modulus at time t , psi (MPa)
E_c	= creep-adjusted modulus of elasticity of concrete, psi (MPa)
$E_c(t)$	= age-dependent elastic modulus of concrete
E_{CON}	= elastic modulus of concrete
E_{steel}	= modulus of elasticity of steel ring, psi (MPa)
$erfc$	= complementary error function
f	= geometry function (Moon and Weiss 2006)
h	= humidity (0 to 1)
G	= coefficient relating stress to steel ring strain 10.44×10^6 psi (72.2 GPa) for the ASTM ring)
$\hat{K}$	= stress amplification factor
K_r	= degree of restraint factor
L	= length
L_o	= measured length of specimen at room temperature, in. (mm)
M_{LWA}	= mass of (dry) fine lightweight aggregate needed per unit volume of concrete, lb/yd ³ (kg/m ³)
R	= degree of restraint

R'	= ideal gas constant [8.314 J/(mol·K)]
R_{IC}	= inner radius of concrete ring
R_{IS}	= inner radius of steel ring
R_{OC}	= outer radius of concrete ring
R_{OS}	= outer radius of steel ring
r	= radius
S	= degree of saturation of aggregate (0 to 1)
T_c	= average concrete temperature °F (°C)
T_{min}	= minimum concrete temperature on a cold night, °F (°C)
$T_{zero-stress}$	= concrete zero-stress temperature, °F (°C)
t_{CR}	= time to cracking
V_W	= molar volume of pore solution
α_{max}	= maximum expected degree of hydration of cement (0 to 1)
α_T	= coefficient of thermal expansion, strain/°F (strain/°C)
β	= coefficient relating shrinkage rate to shrinkage (0.056 day ⁻¹)
$\Delta\epsilon$	= strain increment from autogenous and drying shrinkage
ΔL_a	= actual length change of specimen during temperature change, in. (mm)
ΔL_m	= measured length change of specimen during temperature change, in. (mm) (increase = positive; decrease = negative)
ΔL_f	= length change of the measuring apparatus during temperature change, in. (mm)
ΔT	= measured temperature change (average of the four sensors) °F (°C) (increase = positive; decrease = negative)
$\Delta\sigma$	= stress increment, psi (MPa)
$\epsilon_c(t)$	= creep strain
$\epsilon_e(t)$	= elastic strain
$\epsilon_{sh}(t)$	= free shrinkage strain
ϵ_{hygral}	= nonthermal deformation due to autogenous or drying shrinkage
ϵ_{SH}	= free shrinkage at time t
$\epsilon_{SH-CONST}$	= shrinkage coefficient
ϵ_{ST}	= deformation of steel
ϕ_{LWA}	= desorption of lightweight aggregate from saturation down to 93% RH (mass water/mass dry lightweight aggregate)
$\phi(t)$	= creep coefficient at time t
γ	= surface tension
θ	= contact angle
ν	= Poisson's ratio
ν_c and ν_s	= Poisson's ratio of concrete and steel
σ	= stress, psi (MPa)
$\sigma_{Elastic-Max}$	= theoretical maximum elastic stress
$\psi(t)$	= relaxation coefficient at time t
$\Omega(t)$	= aging coefficient to account for the reduced creep coefficient due to a gradually increasing load in restrained specimen; 0.6 to 0.9 for ordinary hardened concrete, and 0.9 to 1.0 for young concrete

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, “ACI Concrete Terminology,” <http://terminology.concrete.org>. Definitions provided herein complement that resource.

early age—the period after final setting, during which properties are changing rapidly. For a typical Type I portland-cement concrete moist cured at room temperature, this period is approximately 7 days.

CHAPTER 3—CAUSES OF EARLY-AGE DEFORMATION AND CRACKING

The two major driving forces for early-age volume change are the thermal deformation due to cement hydration and shrinkage deformation (autogenous shrinkage and drying shrinkage). The hydration reaction leads to a net reduction in the total volume of the hardened paste, which causes self-desiccation of pores and associated shrinkage. The incremental stress development in a hardening concrete structure with thermal and shrinkage deformations being considered is written as

$$\Delta\sigma = (\Delta T \cdot \alpha_T + \Delta\epsilon) \cdot E \cdot \psi \cdot R \quad (3-1)$$

3.1—Thermal deformation

Thermal deformation is a dimensional change resulting from a temperature change in concrete. Thermal deformation depends on the coefficient of thermal expansion (α_T), which is a function of the state of internal moisture. This was first experimentally identified in 1950 by Meyers (1950), later found by Zoldners (1971), and more recently validated by Bjontegaard (1999). Both Meyers and Zoldners found that, while the α_T at fully dried or saturated conditions was approximately the same, there was a dramatic increase in the α_T at intermediate relative humidities (RHs). Meyers hypothesized that in the intermediate RH range, there is additional dilation due to changes in the pore water pressure exerted on the solid skeleton. Powers and Brownard (1947) noted that with changes in temperature, water expands and undergoes a reduction in surface tension. As the surface tension of water goes down, the negative pressure exerted on the pore system goes down (the pressure pushing the solid nanostructure apart increases), causing additional expansion with increases in temperature.

The development of thermal stresses due to a temperature rise for concrete flatwork may be calculated by the simplified expression presented in Eq. (3-2). The magnitude of the thermal stress is directly proportional to the magnitude of the temperature change to which the flatwork is exposed. For an accurate estimate of the thermal stress, creep effects during early ages and throughout the member's life should be accounted for in Eq. (3-2) (Emborg 1989)

$$\text{thermal stress} = \sigma_T = \Delta T \alpha_T E_c K_r \quad (3-2)$$

It is clear from Eq. (3-2) that the α_T of mixtures is an important factor affecting early age thermal stress. During early ages, α_T changes very rapidly as the concrete gains

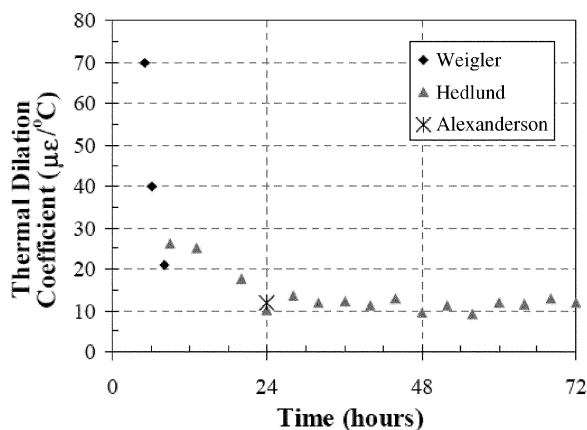


Fig. 3.1—Early-age thermal dilation coefficient for concrete (Hedlund 1996; Byfors 1980). (Note: 0.556 microstrain per °F is equivalent to one microstrain per °C.)

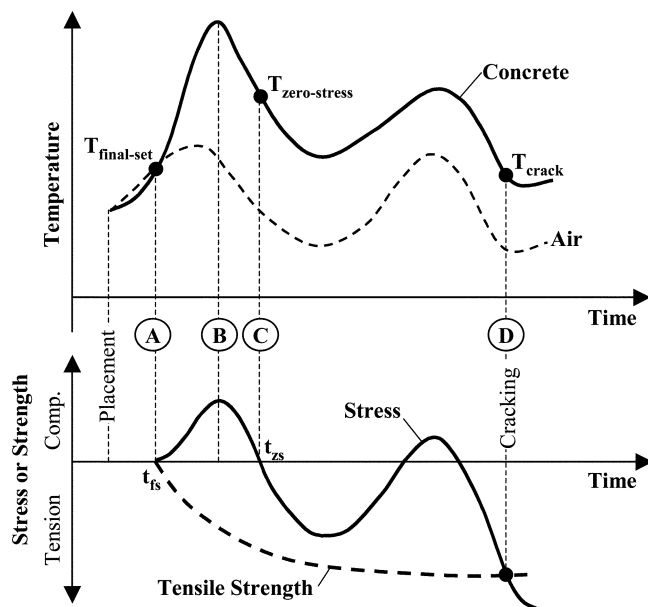


Fig. 3.2—Development of thermal stresses (Schindler and McCullough 2002).

strength. Research by Hedlund (1996) provided thermal dilation coefficients of concrete at early ages, as shown in Fig. 3.1. His test results are supplemented by those of Weigler and Alexanderson (Byfors 1980) that provide even higher values during the first hours. The thermal dilation coefficient reaches a plateau of approximately $6.7 \times 10^{-6}/^{\circ}\text{F}$ ($12 \times 10^{-6}/^{\circ}\text{C}$) after 24 hours. The α_T of hardened concrete depends both on the mixture proportions and its hygral state. There is a general agreement in the literature that the α_T of a given concrete depends strongly on the water content (Meyers 1950; Neville 1981). A fully saturated pore system and an empty (dried) pore system give lower α_T values, around 5.6 to $6.7 \times 10^{-6}/^{\circ}\text{F}$ (10 to $12 \times 10^{-6}/^{\circ}\text{C}$). A partially saturated pore system shows maximum values in the range of $13.9 \times 10^{-6}/^{\circ}\text{F}$ ($25 \times 10^{-6}/^{\circ}\text{C}$).

Figure 3.2 is a typical illustration of early-age temperature and stress development. For summer casting conditions, the

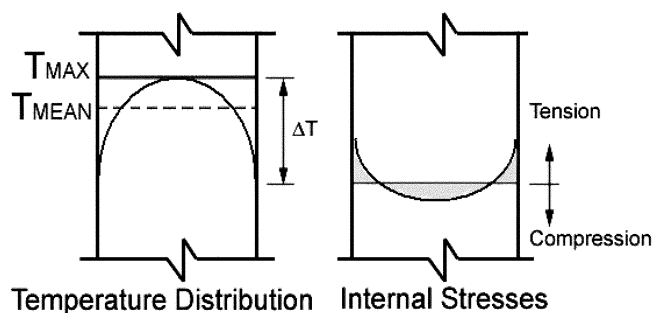


Fig. 3.3—Temperature profile and self-equilibrating stresses in symmetrical cross section at early ages (Emborg 1998).

concrete temperature increases beyond the setting temperature, Line (A). As the expansion of the concrete caused by this temperature rise is restrained, the concrete will be in compression when the peak temperature, Line (B), is reached. During this phase, the hydrating paste is still developing structure, the strength is low, and high amounts of early-age relaxation may occur when the concrete is subjected to high compressive forces (Emborg 1989). When a subsequent decline in concrete temperature starts to occur, the compressive stress will be relieved until the concrete temperature drops below the zero-stress temperature, where the stress condition changes from compression to tension (Line (C)). Due to the effects of relaxation, the zero-stress temperature may be significantly higher than the final-set temperature (Line (A)). If tensile stresses, caused by a further temperature drop, exceed the developing tensile strength of the concrete, cracking will occur (Line (D)).

The zero-stress temperature for the section placed under hot weather conditions (Fig. 3.2) is much higher than the section cast under normal paving temperatures. To improve the performance of concrete pavement placed throughout the year, sections constructed under high air temperature conditions require mitigation techniques to reduce heat development in the concrete (Schindler and McCullough 2002).

Large temperature gradients may develop if concrete surfaces are exposed to large thermal inputs that cannot dissipate quickly; hence, a rather large temperature difference may develop between the center and surface of the structure. Temperature gradients, typically a negative temperature gradient during the curing period, may produce cracks in the finished concrete structure due to the associated thermal strains. With the double-sided heating and cooling conditions seen in Fig. 3.3, the member should maintain a continuous and compatible displacement/strain field. This causes self-equilibrated internal stresses, known as residual stresses, to develop. If external restraint exists, forced thermal deformations create additional stresses.

The difference in surface and interior temperatures may be used to predict the risk of early-age thermal cracking. Based on a comprehensive experimental study, Freiesleben-Hansen and Pedersen (RILEM Committee 42-CEA 1981) suggest that the temperature difference between the middle and surface of a plain wall should be limited to approximately 68°F (20°C) to avoid thermally induced cracks. They found a systematic relationship between the magnitude of the

temperature difference and the occurrence of cracks. Similarly, in cases where a new concrete structure is restrained by a previously placed concrete section (external restraint), it has been suggested that the differentials between the average temperatures of the old and new placements should not exceed 54 to 59°F (12 to 15°C).

Temperature differences generally provide only a first estimate of the risk of thermal cracking. Pioneering work by Emborg (1998) demonstrated that the accuracy of thermal stress analysis depends not only on predicting the early-age temperature profiles, but also the development of concrete properties and degree of restraint. The effects of concrete mixture temperature and composition have a significant influence on thermal cracking.

3.1.1 Influence of placement temperature—As demonstrated previously, the cracking susceptibility of a concrete mixture is highly sensitive to the temperature of fresh concrete. The effect of fresh concrete temperature on cracking sensitivity is shown in Fig. 3.4 and, as expected, lowering the fresh concrete temperature allows the concrete to cool to a lower temperature before cracking initiates, thereby lowering the cracking temperature. These tests were conducted in a room with a temperature that matched the placement temperature using the measurement method described in detail in Chapter 4. Springenschmid and Breitenbücher (1998) found that lowering the fresh concrete temperature from 77 to 54°F (25 to 12°C) decreased the cracking temperature by 21.6 to 34.2°F (12 to 19°C). This vast reduction in cracking temperature may be attributed to a lower concrete setting temperature and an increase in the tensile strength of the hardening concrete (Springenschmid and Breitenbücher 1998).

Low placement temperatures reduce the heat of hydration, thereby reducing the zero-stress temperature. Lowering the zero-stress temperature decreases the subsequent temperature drop during cooling, which in turn reduces tensile stresses (Breitenbücher and Mangold 1994). Springenschmid and Breitenbücher (1998) stated that, as a general rule, increasing the fresh concrete temperature by 18°F (10°C) increases the cracking temperature by 23.4 to 27°F (13 to 15°C).

The increased tensile strength of the hardening concrete may be explained by the bonds formed between the hydration products (Springenschmid and Breitenbücher 1998). At low curing temperatures, the bond formed between the calcium silicate hydrate (C-S-H) gel is stronger than the bond formed at higher temperatures, thus resulting in an increase in tensile strength (Verbeck and Helmuth 1968).

3.1.2 Influence of concrete mixture compositions—To assess the cracking susceptibility of a concrete mixture, the various constituents used to produce the concrete should be investigated on an individual basis. To achieve this, Springenschmid and Breitenbücher (1998) employed an approach that used a standardized concrete mixture. Based on experience, RILEM Technical Committee 119 (1998) specified an appropriate mixture composition to use for a standard:

- Maximum aggregate size: 1.25 in. (32 mm);
- Cement content*: 573 lb/yd³ (340 kg/m³)

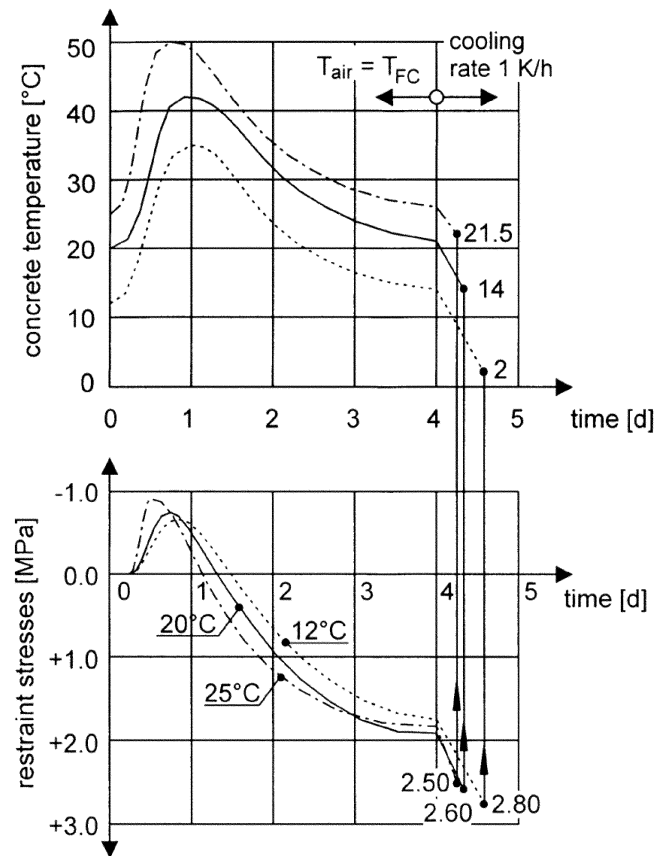


Fig. 3.4—Influence of fresh concrete temperature on cracking sensitivity (Springenschmid and Breitenbücher 1998). (Note: °F = 1.8 × °C + 32; 145 psi = 1 MPa.)

- Water content: 273 lb/yd³ (162 kg/m³)

*The cement content should be appropriately reduced when pozzolanic admixtures are used.

3.1.2.1 Aggregates—Concrete aggregates, especially coarse aggregates, have a substantial effect on cracking tendency. To determine an appropriate coarse aggregate for a given application, consideration should be given to the α_T as well as the aggregate roughness and size. Thermal deformations may be drastically reduced by using aggregates with a low α_T . Breitenbücher and Mangold (1994) reported that concretes composed of quartzite aggregates, which have a 50% higher α_T than limestone, generated approximately 50% higher stresses in the rigid cracking frame (refer to test description in Chapter 4). Although the concretes tested had almost identical temperature profiles, those produced with quartzite aggregates had a cracking temperature 10.8 to 16.2°F (6 to 9°C) higher than those produced with limestone aggregates (Breitenbücher and Mangold 1994).

In addition to their α_T , other thermophysical properties of the aggregates, such as density, heat capacity, and thermal conductivity also influence heat transfer within and through a concrete element and thermal stresses. Literature values indicate that the thermal conductivity of siliceous aggregates (Kim et al. 2003; Horai 1971) is in the range of 2.9 to 4.6 Btu/(ft·hr·°F) (5 to 8 W/(m·K)), approximately double that of the value of 1.7 Btu/(ft·hr·°F) (3 W/(m·K)) for limestone (Kim et al. 2003; Vosteen and Schmidt 2003). Conversely, the

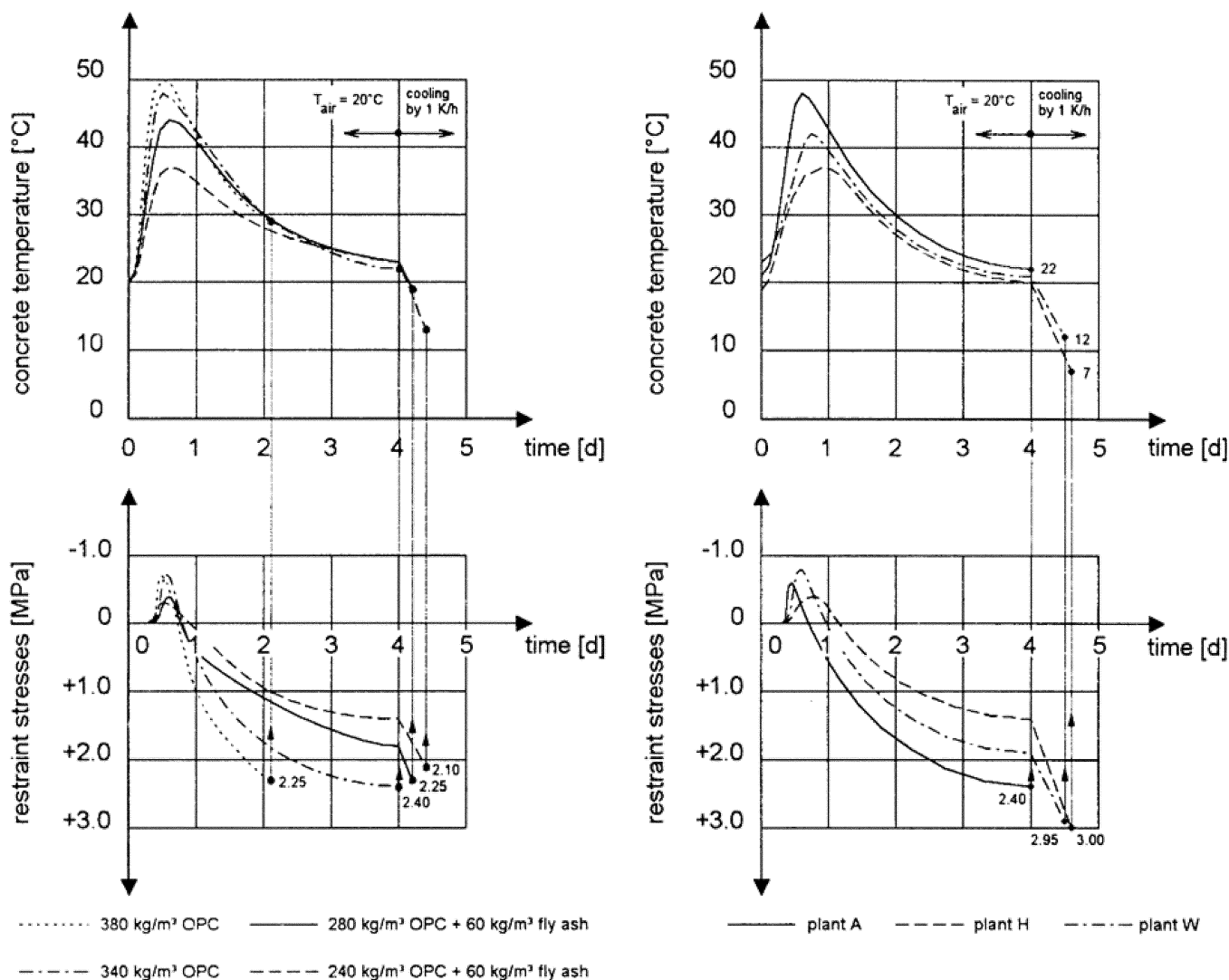


Fig. 3.5—Influence of cement content and cement sources on cracking sensitivity (Breitenbücher and Mangold 1994). (Note: $^{\circ}\text{F} = 1.8 \times ^{\circ}\text{C} + 32$; $145 \text{ psi} = 1 \text{ MPa}$; $1.686 \text{ lb/yd}^3 = 1 \text{ kg/m}^3$.)

heat capacities of the two most common types of aggregates are similar, with limestone having a value of $0.2 \text{ Btu}/(\text{lb} \cdot ^{\circ}\text{F})$ ($0.84 \text{ J}/(\text{g} \cdot \text{K})$), and siliceous aggregates having a value of $0.18 \text{ Btu}/(\text{lb} \cdot ^{\circ}\text{F})$ ($0.75 \text{ J}/(\text{g} \cdot \text{K})$) (Waller et al. 1996).

Crushed aggregates with a rough surface are usually advantageous because the tensile strength of the concrete is increased. Furthermore, the use of larger aggregates tends to reduce the cracking susceptibility of a concrete mixture and reduces the amount of cement paste necessary for sufficient workability; lowering the amount of cement paste results in a lower heat of hydration. Larger coarse aggregates, however, tend to reduce the tensile strength of the concrete (Springenschmid and Breitenbücher 1998).

3.1.3 Cement types and sources—Portland cements obtained from different plants displayed very different cracking sensitivities in the rigid cracking frame, as shown in Fig. 3.5. The cements were all of the same specification type; however, each had different chemical compositions, fineness, or both (Springenschmid and Breitenbücher 1998). Tests on 17 concretes made with cements acquired from

different plants yielded cracking temperatures ranging from 43 to 72°F (6 to 22°C). Although temperature profiles of these concretes were different, the large range of cracking temperatures may only be partly attributed to the various temperature histories (Breitenbücher and Mangold 1994).

Springenschmid and Breitenbücher (1998) found that portland cements with a low alkali content (K_2O , Na_2O), a high sulfate content in relation to the C_3A content, and that are not too fine, lead to low cracking temperatures. These results indicate that higher sulfate contents (that is, higher additions of gypsum) could decrease the cracking temperature; however, this is true only for cements with low alkali content. In high-alkali cements, an increased sulfate content does not improve the cracking sensitivity (Breitenbücher and Mangold 1994).

3.1.3.1 Cement content and fly ash—The effect of cement content and fly ash on the cracking temperature is also shown in Fig. 3.5. In the normal range of water-cementitious material ratios ($0.4 \leq w/cm \leq 0.7$), cracking temperatures decrease as the cement content decreases.

Breitenbücher and Mangold (1994) found that once the w/cm exceeds 0.7, the tensile strength of the concrete lowers, resulting in increased cracking temperatures.

To maintain sufficient workability, Breitenbücher and Mangold (1994) added fly ash when the cement content of the concrete was lower than 573 lb/yd³ (340 kg/m³). Breitenbücher and Mangold (1994) stated that during the testing period in the first 4 to 5 days, the fly ash itself did not significantly influence the cracking tendency of the concrete, and could be considered as an inert material. As shown in Fig. 3.5, the addition of fly ash as a replacement of portland cement may reduce the cracking temperature.

3.1.3.2 Slag cement—Slag cements are normally used as low-heat cements for mass concrete applications because they are effective in reducing the rate of heat development in the concrete (Breitenbücher 1990). In some concretes with slag cements, however, a higher cracking temperature was found than in concretes using ordinary portland cement (OPC) from a plant with the same clinker (Springenschmid et al. 1987). The increased cracking temperatures are related to the slow strength development of the slag concrete. Although slag cements are effective in reducing thermal deformations, the benefits are offset by the lack of tensile strength at early ages (Thomas and Mukherjee 1994).

3.1.3.3 Air-entraining admixtures—Air-entrained concrete has two important properties that affect cracking susceptibility. First, the ultimate tensile strain capacity of air-entrained concrete is approximately 20% higher than normal concrete (Springenschmid and Breitenbücher 1998). Second, air-entrained concretes have a lower modulus of elasticity, which in turn leads to lower restraint stresses. Breitenbücher and Mangold (1994) found that adding air-entraining agents (air content of approximately 3 to 6% by volume) reduced the cracking temperature by approximately 9°F (5°C).

3.2—Autogenous shrinkage

The internal volume reduction associated with the hydration reactions in a cementitious material is typically 1.66 to 1.94 in.³/lb (6 to 7 mL/100 g) of fully hydrated cement (Jensen and Hansen 1996). The basic reactions of cement clinker are well understood and generally defined in terms of four ideal clinker phases, namely, C_3S , C_2S , C_3A , and C_4AF . Each of these clinker minerals requires water for reaction, and results in a decreased volume of the reaction products. Chemical shrinkage is like a molecular-level volume change, and creates the underlying driving force for the occurrence of autogenous shrinkage that is the macroscopic bulk deformation of a closed, isothermal, cementitious material system not subjected to external forces.

Autogenous shrinkage has been shown to be equal to chemical shrinkage as long as the paste is liquid. Around the time of setting, however, a solid skeleton is formed that allows empty pores to form, and the resulting autogenous shrinkage becomes much smaller than the underlying chemical shrinkage. Figure 3.6 shows such behavior for a $w/cm = 0.3$ portland-cement paste mixture. The chemical and autogenous shrinkage are initially identical within the first 4 hours when

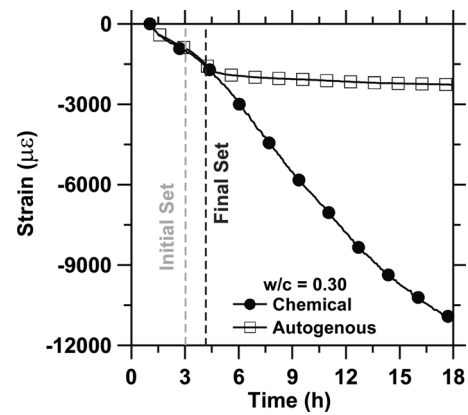
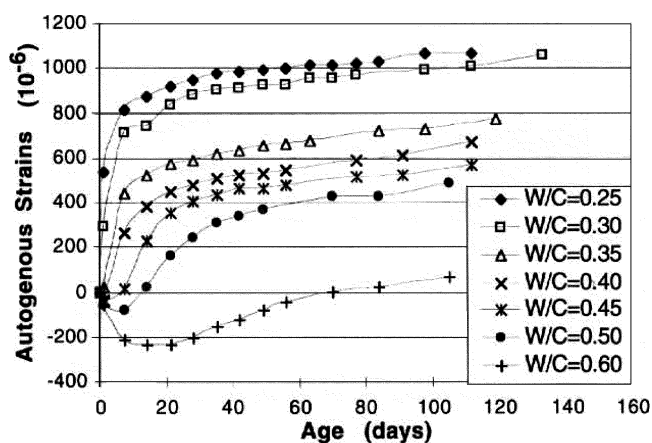


Fig. 3.6—Development of chemical shrinkage and autogenous shrinkage, $w/cm = 0.3$ (Sant et al. 2009).

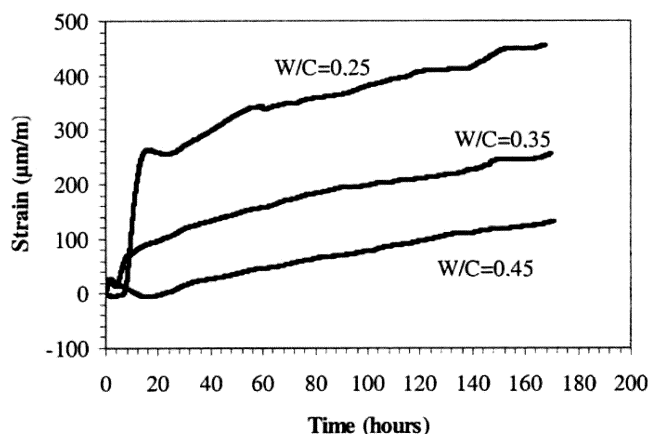
the paste is in a liquid condition; thereafter, the autogenous shrinkage is substantially reduced due to the formation of a solid skeleton. The part of the autogenous shrinkage that occurs after setting is self-desiccation shrinkage. Autogenous shrinkage due to self-desiccation typically increases with decreasing w/cm . When external curing water is not readily available, self-desiccation will occur, and a set of empty pores will be created within the microstructure. This will produce capillary stresses that result in a measurable physical shrinkage of the material.

There is a general agreement on the existence of a relationship between autogenous shrinkage and relative humidity changes in the pores of the hardening cement paste (Lura 2003). Autogenous shrinkage may be thought of as a special class of drying shrinkage where the moisture is removed from the pore structure through internal chemical and physical reactions rather than external drying. The hydration reaction that occurs when portland cement comes in contact with water leads to an overall volume reduction such that the volume of the products is less than the volume of the reactants. Once percolation of the solid phase (initial setting) occurs, the volume reduction leads to the formation of vapor-filled porosity as water is further consumed in the hydration reaction. Moisture will tend to be consumed from larger pores first, followed by successively smaller pores. Unlike later-age drying that may occur from the outside of the specimen inward, internal drying occurs uniformly throughout the concrete microstructure.

3.2.1 Influence of water-cementitious material ratio—High-performance concrete (HPC) (relative to conventional concrete) and concrete with a low water-cementitious material ratio (w/cm) are expected to have increased autogenous shrinkage due to higher cement content, and the pozzolanic mineral admixtures that consume additional water in a reaction to form secondary C-S-H (Jensen and Hansen 1996). From linear measurements of autogenous shrinkage, a low w/cm produces higher autogenous shrinkage for both cement paste and concrete, as shown in Fig. 3.7. For concrete, autogenous shrinkage differences are pronounced within the first days, and a low w/cm gives high autogenous shrinkage. At the same time, autogenous expansions are observed in concrete paste with w/cm above 0.4, resulting in lower net tensile stresses



(a)



(b)

Fig. 3.7—Effect of w/cm on autogenous deformation (+ is shrinkage) in: (a) cement paste (Baroghel-Bouny 1996); and (b) concrete (Toma et al. 1999) (both with ordinary portland cement) during 68°F (20°C) isothermal tests. The measurements were started at the time of initial set in (a), and from $t = 0$ in. (b). (Note: 1×10^{-6} in./in. = $1 \mu\text{m}/\text{m}$.)

The correlation between w/cm and autogenous shrinkage may be explained by the self-desiccation process that dominates the autogenous shrinkage after the formation of a solid skeleton of the cement paste. The self-desiccation process is assumed to be based on a capillary tension mechanism such that a finer initial pore structure will generate a lower-equilibrium, internal RH and will create higher tension in the pore water. This is in agreement with experimental observations that have indicated autogenous shrinkage is increased by a reduction in w/cm , an increase in the fineness of the cement (Bentz et al. 2001a), or an addition of silica fume (Jensen and Hansen 1996).

3.2.2 Influence of aggregate—An increase in aggregate volume fraction (reduction in paste content) or an increase in maximum aggregate size will generally reduce autogenous shrinkage. Holt's experimental results show that paste has approximately 1.7 times greater shrinkage as compared with mortar (Holt 2002). Test results reported by Tazawa et al. (1995) also show that an increase in the aggregate content leads to a decrease of autogenous shrinkage.

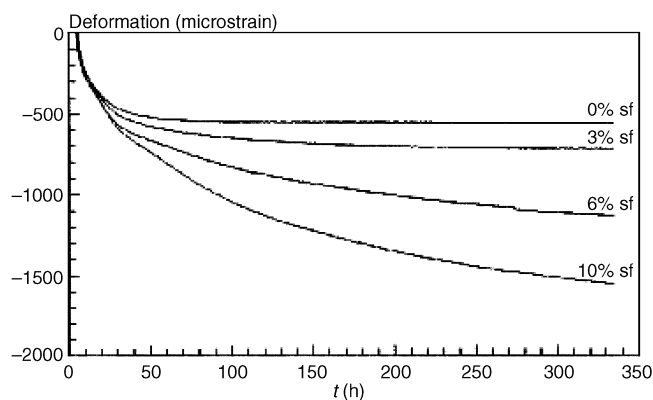


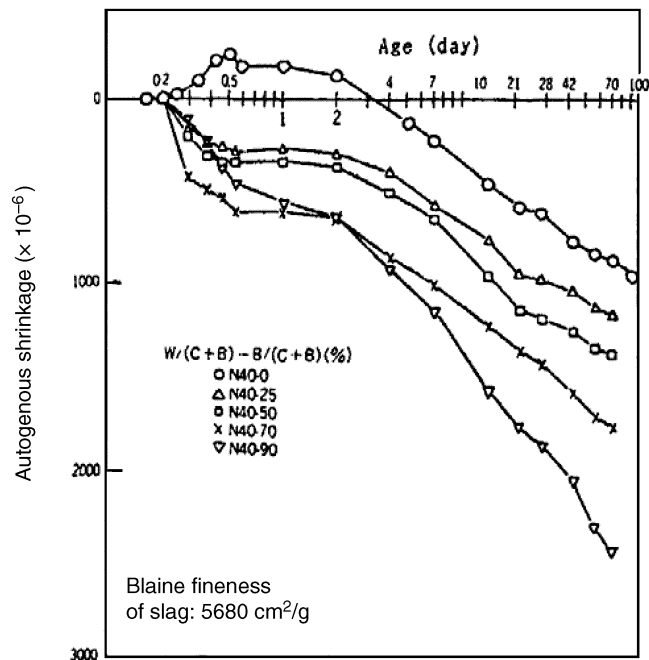
Fig. 3.8—Autogenous deformation after setting of cement pastes with $w/cm = 0.35$ and 0 to 10% silica fume addition: silica fume addition marked on curves; time measured from water addition; deformation fixed to 0 at time of setting, temperature 86°F (30°C) (Jensen and Hansen 1996).

3.2.3 Influence of supplementary cementitious materials—

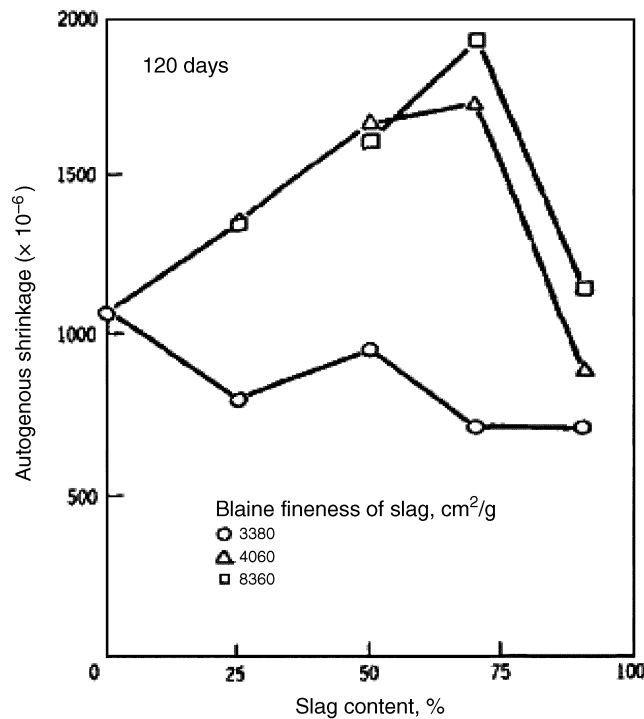
Supplementary cementing materials, such as fly ash, silica fume, and slag cement, are increasingly used for the production of low w/cm mixtures.

Jensen and Hansen (1996) measured the linear autogenous shrinkage of cement pastes containing 0 to 10% silica fume. All mixtures had a w/cm of 0.35. Their experiments indicated that the silica fume dosage has a dominating influence, as shown in Fig. 3.8. A 10% silica fume addition could increase the autogenous deformation after 2 weeks by 1000 microstrains. By way of comparison, a change of the w/cm from 0.40 to 0.25 for cement pastes containing 10% silica fume only increased the autogenous deformation after 2 weeks by approximately 200 microstrains. A mechanism was proposed by Jensen and Hansen (1996) to explain the considerable increase of autogenous shrinkage by the addition of silica fume. It is because of the disappearance of restraining components (silica fume, calcium hydroxide, or both), as the pozzolanic reaction of the silica fume involves a consumption of calcium hydroxide. The addition of silica fume may also lead to a finer pore structure of the cement paste, thereby increasing self-desiccation. Furthermore, the pozzolanic reaction increases the chemical shrinkage and, thus, the autogenous shrinkage of the blended cement. In concrete practice, silica fume addition has frequently been linked with cracking during hardening. This might be a consequence of the highly increased autogenous deformation due to silica fume additions.

Autogenous shrinkage depends both on the slag cement content and its fineness (Tazawa and Miyazawa 1995) (Fig. 3.9). Fly ash was found to have a variable influence in the literature. An investigation by Setter and Roy (1978) showed that fly ash has no significant effects on the autogenous shrinkage. In a study by Justnes et al. (1998), however, it was found that the addition of Class F fly ash significantly increases the autogenous shrinkage of cement paste. Not until this research was based on the measurement of chemical shrinkage.



(a)



(b)

Fig. 3.9—(a) Influence of slag cement on autogenous shrinkage; and (b) influence of Blaine fineness of slag cement (Tazawa and Miyazawa 1995).

3.3—Drying shrinkage

Drying shrinkage may produce cracks in concrete if the shrinkage is restrained internally, externally, or both. As drying progresses from the concrete surface inward, pore humidity and the corresponding free drying shrinkage are generally distributed nonuniformly along the structure's

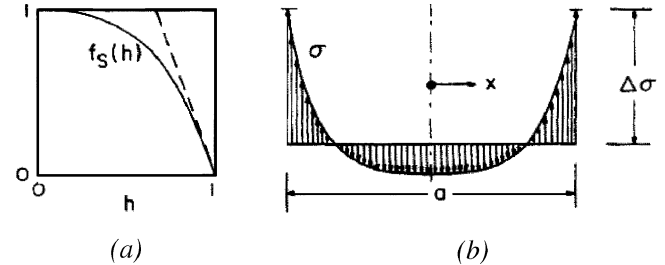


Fig. 3.10—(a) Variation of free shrinkage with RH h in pores; and (b) typical distribution of stresses throughout a wall (Bažant and Raftishol 1982).

thickness, with the largest strain at surface and decreasing as the distance from the free surface increases. Bažant and Panula (1978) established a function for determining the free drying shrinkage of a small element of concrete from the pore RH, as shown in Fig. 3.10(a). For the case of drying from both sides, the drying shrinkage stress is obtained through the consideration of internal restraint, and the strain should be uniform over the cross section. Bažant and Panula (1978) calculated the drying shrinkage stress by multiplying free drying shrinkage with an effective modulus that is Young's modulus E reduced to consider creep effects. Calculated shrinkage stress is illustrated in Fig. 3.10(b), which shows maximum tensile stress is reached at the drying surfaces. Many researchers found that peak tensile stress in the surface layer occurs in the beginning or during the first day of drying, and surface microcracking was observed in concrete and cement paste (Hwang and Young 1984; Higgins and Bailey 1976). Higgins and Bailey (1976) even observed microcracking in cement paste within 1 minute after exposure to drying.

The structural responses to drying may be different from other types of shrinkage deformations, such as autogenous shrinkage and thermal contraction, because drying often occurs differentially throughout the member thickness. For the case of a concrete slab resting on the ground, drying proceeds from the top surface only (humidity at the bottom is fairly constant due to contact with the ground) will cause a nonlinear distributed shrinkage profile along slab depth. Two visible deformations develop as a result of the differential shrinkage: 1) the axial movement, which is conventionally treated as drying shrinkage; and 2) warping. If restrained, these two deformations may cause axial stress and bending stress, respectively, as shown in Fig. 3.11. Another stress component is also generated during the drying, and is described as the residual stress that is due to the internal restraint from the surrounding small elements to satisfy translational symmetry in concrete cross sections.

3.4—Creep and stress relaxation from deformation restraint

Concrete undergoes time-dependent deformations during the application of internal or external stresses. Aggregate in concrete serves to restrain these deformations, which are a function of the volume of cement paste. The mechanisms of creep are not completely understood, as they depend on the composition and microstructure of cement hydration products.

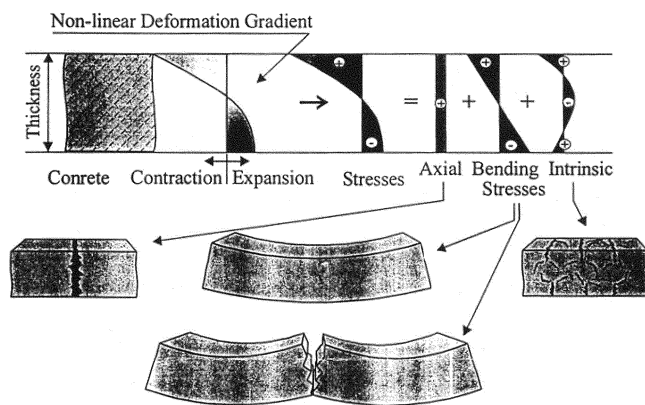


Fig. 3.11—Deformations and stresses caused by a nonlinear drying-shrinkage gradient (Springenschmid and Plannerer 2001).

Creep in concrete is the viscoelastic deformation of the porous cement hydration products due to applied stress. Several mechanisms are thought to be responsible for time-dependent deformation (ACI 209R; Acker 2004; Altoubat and Lange 2003):

- Redistribution of capillary water or interlayer water movement;
- Sliding or shearing of gel particles lubricated by adsorbed water (viscous flow);
- Sliding or shearing movement of the crystalline lattice (plastic flow);
- Diffusion of solid material; and
- Permanent plastic deformation caused by micro-cracking, recrystallization, or the formation of new physical bonds in the hydration products.

At early ages, creep may be viewed as a beneficial relaxation mechanism because it reduces tensile stress development that leads to cracking. In some instances, a reduced creep capacity could have a negative impact on a restrained structure in terms of cracking and durability. Creep deformation, however, leads to larger deflections in structural elements and loss of prestressing force, so it may be unfavorable for structural serviceability requirements.

Many of the same concrete properties that affect shrinkage also affect creep, such as internal moisture content, drying rate, and temperature. In fact, shrinkage of concrete may be analyzed as a viscoelastic response to internal stresses (Acker 2004; Grasley et al. 2006b; Lura et al. 2005). Creep is often divided into separate mechanisms for drying creep and basic creep. Basic creep occurs in sealed specimens; however, deformations at an early age may also be due to autogenous shrinkage (Altoubat 2000). Drying creep was first observed by Pickett, and is often referred to as the Pickett effect (Pickett 1942). Although several mechanisms for drying creep have been proposed, none are universally accepted (Neville 1981). The two components have been separated by experimental methods using superposition analysis (Kovler 1994; Altoubat 2000). Pickett observed that the creep deformation of a specimen loaded in compression during drying was greater than creep measured in a sealed specimen and shrinkage added together from separate

specimens. The Pickett effect, also referred to as stress-induced shrinkage, describes the additional creep not explained with superposition.

3.5—Mitigation of shrinkage

The simplest way of mitigating shrinkage is to reduce the cement paste content in concrete, because cement paste undergoes autogenous and drying shrinkage. Research by Bisschop (2002) indicates that the amount of drying shrinkage is approximately six to eight times smaller for concrete compared with cement paste. This mitigating method, however, may not be feasible as in cases where specific concrete properties are to be maintained. The advance of technology, therefore, has resulted in alternatives for mitigating shrinkage, such as adding chemical admixtures (SRAs) and incorporating internal curing agents (internal curing). These topics are covered in more detail in Chapter 5.

CHAPTER 4—TEST METHODS AND ASSESSMENT

4.1—Introduction

Often, it is the superposition of shrinkage (autogenous plus drying shrinkage) and thermal stresses that, combined, lead to early-age cracking. As the two effects are difficult to separate experimentally for realistic size concrete replicas, their combined influence on a measurement is instead assessed in Chapter 4.

Several test methods have been proposed to assess the shrinkage cracking potential of concrete mixtures (Grzybowski and Shah 1990; Weiss and Shah 1997) including ring specimens (Carlson 1942; Shah et al. 1992; Bentur 2003; AASHTO 2004; ASTM C1581) and linear specimens (Paillere et al. 1989; Springenschmid et al. 1985; Kovler et al. 1993; Toma et al. 1999; Altoubat and Lange 2002). Ring tests evaluate the performance of concrete or mortar mixtures when they are restrained from shrinking freely. In the ring test, a concrete (or cement paste) annulus is cast around a steel ring. If unrestrained (that is, no steel ring), the concrete would shrink freely; however, the steel ring prevents (restrains) this movement, resulting in the development of tensile stresses. Due to its simplicity and economy, the ring test has been developed into both AASHTO (2004) and ASTM C1581. Linear specimens have the advantage of a relatively straightforward data interpretation; however, these test methods are generally not used for quality-control procedures, partially due to difficulties associated with providing sufficient end restraint (Weiss and Shah 1997; Altoubat and Lange 2002). Due to the complexity of a concrete member, finite element analysis becomes a useful tool for assessing the restraint stress development. A few commercially available finite element programs and their features are introduced in this chapter.

4.2—Shrinkage measurements

4.2.1 Drying shrinkage—Drying shrinkage is an unavoidable phenomenon when concrete members are exposed to the environment. As mentioned previously, uniform shrinkage and warping may be caused by a drying process in concrete.

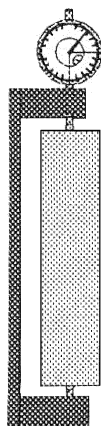


Fig. 4.1—Conventional shrinkage test (Springenschmid and Plannerer 2001).

Typically, only uniaxial drying shrinkage is measured to assess the potential effects of drying shrinkage.

The uniaxial drying shrinkage is a time-dependent longitudinal contraction of a prismatic or cylindrical specimen under drying conditions (Springenschmid and Plannerer 2001) as shown in Fig. 4.1. The magnitude of uniaxial drying shrinkage depends on the size of concrete members because the drying process is limited to a surface region of a few centimeters thickness. A part of the drying shrinkage is irreversible. As concrete dries and then wet again, it will not fully resume its original dimensions (Neville 1981). The shrinkage strain of concrete that is dried for the first time is usually significantly greater than if the concrete is rewetted and dried again.

4.2.2 Autogenous shrinkage—As a result of cement hydration, chemical shrinkage is quantified through measurements of the volume contraction of paste in a glass beaker connected to a water-filled tube (LeChatelier 1904). There have been two methods used to directly measure chemical shrinkage: dilatometry and weighing reduced buoyancy. Both of these methods include an external water source in contact with the cement paste sample. The dilatometry test procedure follows LeChatelier's principle where an Erlenmeyer flask or small glass vial containing a cement paste sample is connected to a pipette in which the dropping water level is monitored (Fig. 4.2). It is a simple and commonly used test procedure (Tazawa 1999), but requires manual reading of the pipette. Typically a cement paste with a w/cm in the range of 0.4 is used. The technique was standardized as ASTM C1608. There is always the risk of faulty readings from the fit of the pipette through a rubber stopper in the top of the vial or flask.

The mass method (reduced buoyancy) is easier in the sense that it may be continuously measured and logged by a computer. It was developed based on Archimedes principle that a water-submerged sample will register a volume reduction by a mass increase. The mass of the submerged cement paste or mortar sample increases as the cement hydration proceeds. This method has been applied by many researchers, and is shown in Fig. 4.2 (Boivin et al. 1999; Geiker and Knudsen 1982; Paulini 1992). An excellent

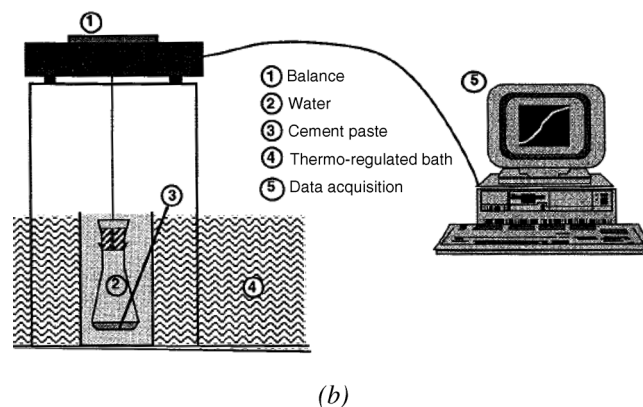
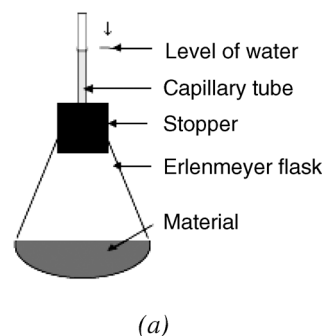


Fig. 4.2—(a) Dilatometry test of chemical shrinkage; and (b) reduced buoyancy test arrangement.

correlation between the two chemical shrinkage measuring methods was shown in tests by Boivin et al. (1999).

The major difference between the environments used for chemical shrinkage and autogenous shrinkage measurements is that in chemical shrinkage, an external water source provides the additional water absorbed by the cementitious material. In an autogenous shrinkage test, the sample is sealed, and there is no moisture exchange between the sample and the environment.

Autogenous shrinkage may be measured in terms of either volume change or length change. Volumetric measurements are done by taking the mass of a submerged sample sealed in a thin rubber membrane, as shown in Fig. 4.3 (Bjontegaard 1999). The procedure consists of filling the rubber membrane with fresh cement paste or mortar and placing it in a rigid tube. This tube is then immersed in a temperature-controlled water bath and continuously rotated to avoid bleeding. The creation of empty spaces leads to a modification of the buoyancy, making it possible to obtain variations in volume of the sample by simple mass measurements (Justnes et al. 1996).

Reabsorption of the bleeding water, however, may be an important factor affecting the autogenous shrinkage measured using the volumetric method. The autogenous shrinkage of a static sample is always smaller than that of a rotated sample. This is caused by the reabsorption of the bleeding water, which significantly reduces the self-desiccation in the static sample. This effect increases with the increase of w/cm , as higher w/cm samples are more susceptible to bleeding. Even more importantly, it has been found that impermeable membranes commonly used are actually quite

permeable to water over the course of the testing period (Lura and Jensen 2005).

Linear measurements are usually carried out by placing the sample in a rigid mold with low friction. The length change of the cement paste is recorded by a displacement transducer at the end of the specimen (Fig. 4.4). There are, however, a few limitations in the linear measurements. For example, the geometry of the sample, the friction between the sample and mold, and the measurement might not be carried out before the sample has set.

Jensen and Hansen (1995) developed a method for measuring the autogenous shrinkage from the linear length change of the sample, as shown in Fig. 4.5. Cement paste or mortar may be cast into the sealed soft plastic tubes that are corrugated to minimize restraint on the paste. The tubes are 11.8 in. (300 mm) in length, and 1.18 in. (30 mm) in diameter. The techniques allow measurements to start 30 minutes after casting, and a transducer monitors the change in tube length over time. This corrugated mold transforms volumetric deformations into linear deformations when the paste is in a fluid state; the ratio between the length and volume change was found to be 2.15 in./fl oz (1.85 mm/mL). This measurement technique is being standardized within the ASTM C09-68 subcommittee on volume change.

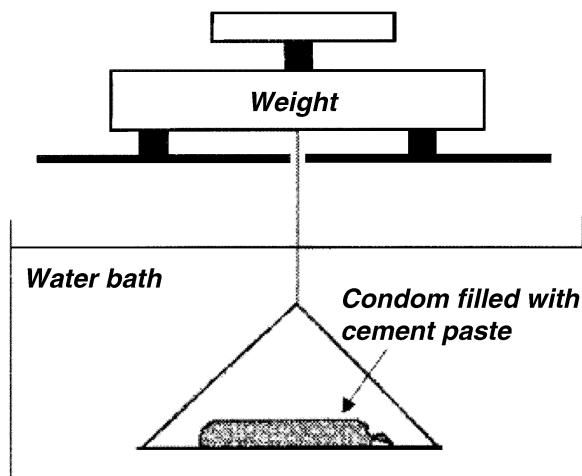


Fig. 4.3—Rubber membrane test for measuring autogenous shrinkage (Bjontegaard 1999).

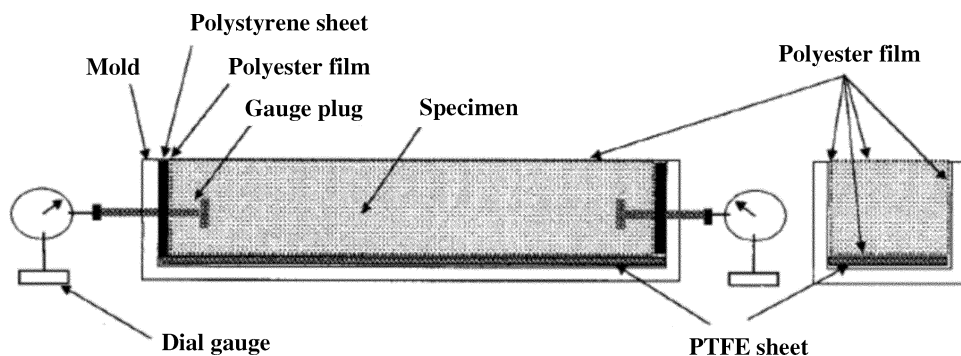


Fig. 4.4—Linear measurement of autogenous shrinkage (Tazawa 1999).

4.3—Ring test

The ring test is a practical tool for assessing time-of-cracking and cracking sensitivity in cementitious materials. The following sections provide an overview of the qualitative use of the restrained ring test, the use of the ring for quantitative analysis of stress development, and emerging developments that may account for materials that expand at early ages.

4.3.1 Quantitative restrained ring tests—The ring test has been used by many researchers over the last century. The most common use of the ring test is where the inner (steel) ring provides passive restraint (through its stiffness) to the concrete ring specimen. For example, Carlson (1942, 1988) and Coutinho (1959) used the ring test to assess the cracking of restrained concrete elements. Brewer and Burrows (1951) used a smaller ring geometry to assess cracking in the cement paste. Swamy and Stavarides (1979) and Krenchel and Shah (1987) used the ring test for fiber-reinforced materials. The restrained ring test has also been used to examine the influence of new materials (for example, SRAs) (Shah et al. 1992, 1998; Folliard and Berke 1997) and mixture proportions on the cracking potential of concrete (Krauss et al. 1996).

Grzybowski and Shah (1989, 1990) used a model to calculate the stresses that could be expected to develop in a ring specimen (assuming the tangential stress is uniform along the radius) and predicted the cracking potential using a damage-based model. Weiss (1999) and Weiss et al. (2000) computed the nonlinear stress distribution that develops in the ring and used a nonlinear fracture mechanics model to predict cracking for various ring geometries. Other researchers used the ring geometry with a more active inner core. Malhotra (1970) used a pressurized ring to determine tensile strength of concrete, Kovler et al. (1993) used an inner ring made of poly(methyl) methacrylate that expanded on heating, and Weiss (1999) used a pressurized inner ring to monitor creep. Figure 4.6 is a schematic illustration of the restraint stress distribution in typical ring tests for ring specimens that experience drying from top and sides, respectively.

4.3.2 Stress development—Over the last few decades, ring tests have been instrumented with strain gauges and used to quantify the stress development inside concrete. Figure 4.7 illustrates the geometry of the ring specimens, with the inner radius of the steel ring given as R_{IS} , the outer radius of the steel and the inner radius of the concrete ring given as $R_{OS} = R_{IC}$, and the outer radius of the concrete ring given as R_{OC} .

In general, strain is measured using four strain gauges mounted on the inner surface of the steel ring (that is, at R_{IS}) at midheight along the z-direction (however, a greater or lesser number of strain gauges may be used). The average strain measured on the inner surface of the steel ring, ϵ_{ST} , is used in the stress calculations. Figure 4.7(b) shows the



Fig. 4.5—Dilatometer with corrugated modulus for measurement of linear autogenous shrinkage of cement paste (Jensen and Hansen 1995).

geometry and dimensions of the ASTM C1581 ring setup. In this setup, two strain gauges are mounted on diametrically opposite inner surfaces of the steel ring. The specimens are dried from the outer circumference of the ring.

Swamy and Stavarides (1979) used the strain gauges to obtain an indication of the magnitude of the stress (average stress) that develops in the concrete (E_{Steel} is the elastic modulus of the steel ring)

$$\sigma_{Ave} = -\epsilon_{ST} \cdot E_{Steel} \cdot \frac{(R_{IC} - R_{IS})}{2(R_{OC} - R_{IC})} \left(1 + \frac{R_{IC}^2 - R_{OC}^2}{2R_{OC}} \right) \quad (4-1)$$

Equation (4-1) was derived assuming a linear stress distribution throughout the steel ring. Equilibrium concepts were then applied, setting the compressive force in the steel ring equal to the tensile force in the concrete ring. This enabled the determination of the average tensile stress in the concrete ring.

Attigbe et al. (2001), See et al. (2003), and Weiss and Ferguson (2001) independently proposed solutions that use the strain measured in the steel ring to compute the maximum stresses in the concrete ring. Attigbe et al. (2001) and See et al. (2003) considered a thin-wall approximation to

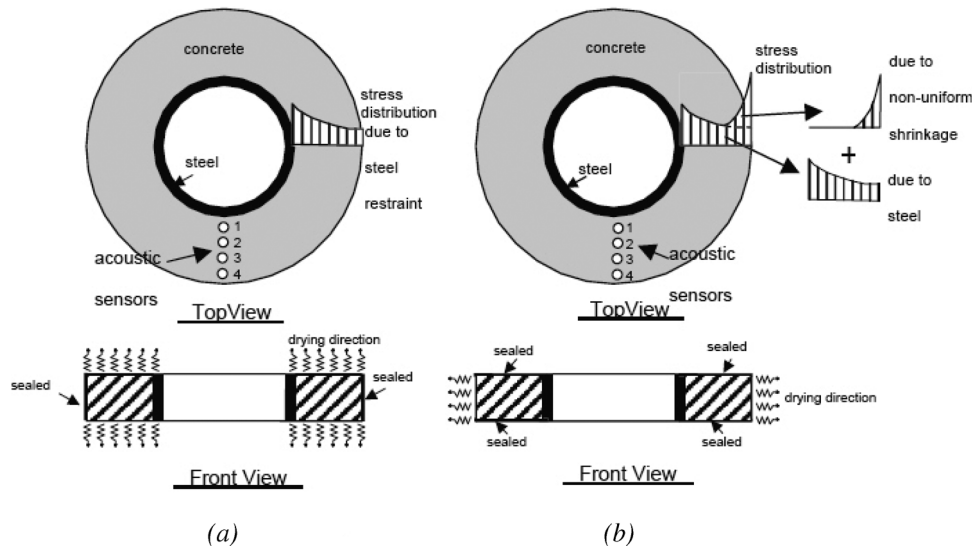


Fig. 4.6—Schematic illustration of restraint stresses generated in ring test (Weiss et al. 2000).

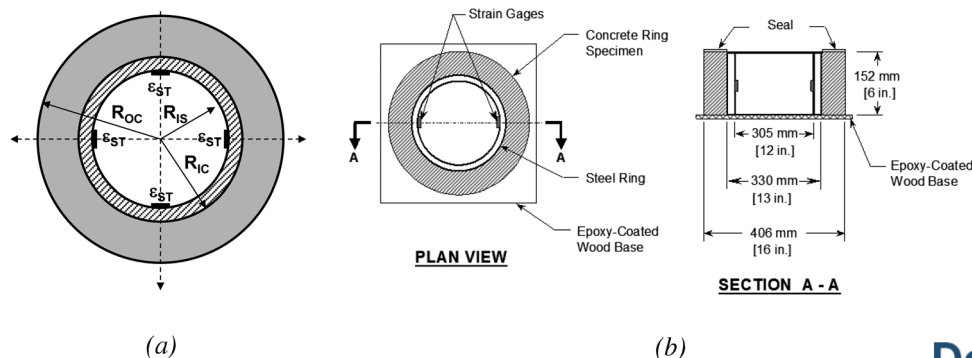


Fig. 4.7—(a) Ring geometry; and (b) ASTM C1581 ring setup.