Control of Cracking in Concrete Structures

Reported by ACI Committee 224

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The principal causes of cracking and recommended crack-control procedures are presented. The current state of knowledge in microcracking and fracture of concrete is reviewed. The control of cracking due to drying shrinkage and crack control in flexural members, overlays, and mass concrete construction are covered in detail. Long-term effects on cracking are considered and crack-control procedures used in construction are presented. Information is presented to assist in the development of practical and effective crack-control programs for concrete structures. Extensive references are provided.

Keywords: aggregates; anchorage (structural); bridge decks; cement-aggregate reactions; concrete construction; concrete pavements; concrete slabs; cooling; corrosion; crack propagation; cracking (fracturing); crack width and spacing; drying shrinkage; shrinkage-compensating concrete; heat of hydration; mass concrete; microcracking; polymer-modified concrete; prestressed concrete; reinforced concrete; restraint; shrinkage; temperature; tensile stresses; thermal expansion; volume change.

CONTENTS

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CHAPTER 1—INTRODUCTION

Cracks in concrete structures can indicate major structural problems and detract from the appearance of monolithic construction. There are many specific causes of cracking.

This report presents the principal causes of cracking and a detailed discussion of crack-control procedures. The report consists of eight chapters designed to help the engineer and the contractor in developing crack-control measures.

This report is an update of previous committee reports (ACI Committee 224 1972, 1980, 1990). ACI Bibliography No. 9 supplemented the original ACI 224R (1971). The Committee has also prepared reports on the causes, evaluation, and repair of cracking, ACI 224.1R; cracking of concrete in direct tension, ACI 224.2R; and joints in concrete construction, ACI 224.3R.

In this revision of the report, Chapter 2 on crack mechanisms has been revised extensively to reflect the interest and attention given to aspects of fracture mechanics of concrete during the 1980s. Chapter 3 on drying shrinkage has been rewritten. Chapter 4 has been revised to include updated information on crack-width predictive equations, cracking in partially prestressed members, anchorage zone cracking, and flexural cracking in deep flexural members. Chapter 6 on concrete overlays has been reorganized and revised in modest detail to account for updated information on fiber reinforcement and on polymer-modified concrete. Chapter 7 on mass concrete has been revised to consider structural consequences more extensively.

CHAPTER 2—CRACK MECHANISMS IN CONCRETE

2.1—Introduction

Cracking plays an important role in concrete's response to load in both tension and compression. The earliest studies of the microscopic behavior of concrete involved the response of concrete to compressive stress. That early work showed that the stress-strain response of concrete is closely associated with the formation of microcracks, that is, cracks that form at coarse-aggregate boundaries (bond cracks) and propagate through the surrounding mortar (mortar cracks) (Hsu, Slate, Sturman, and Winter 1963; Shah and Winter 1966; Slate and Matheus 1967; Shah and Chandra 1970; Shah and Slate 1968; Meyers, Slate, and Winter 1969; Darwin and Slate 1970), as shown in Fig. 2.1.

During early microcracking studies, concrete was considered to be made up of two linear, elastic brittle materials; cement paste and aggregate; and microcracks were considered to be the major cause of concrete's nonlinear stress-strain behavior in compression (Hsu, Slate, Sturman, and Winter 1963; Shah and Winter 1966). This picture began to change in the 1970s. Cement paste is a nonlinear softening material, as is the mortar constituent of concrete. The compressive nonlinearity of concrete is highly dependent upon the response of these two materials (Spooner 1972; Spooner and Dougill 1975; Spooner, Pomeroy, and Dougill 1976; Maher and Darwin 1977; Cook and Chindaprasirt 1980; Maher and Darwin 1982) and less dependent upon bond and mortar microcracking than originally thought. Research indicates, however, that a significant portion of the nonlinear deformation of cement paste and mortar results from the formation of microcracks that are several orders of magnitude smaller than those observed in the original studies (Attiogbe and Darwin 1987, 1988). These smaller microcracks have a surface density that is two to three orders of magnitude higher than the density of bond and mortar microcracks in concrete at the same compressive strain, and their discovery represents a significant step towards understanding the behavior of concrete and its constituent materials in compression.

The effect of macroscopic cracks on the performance and failure characteristics of concrete has also received considerable attention. For many years, concrete has been considered a brittle material in tension. Many attempts have been made to use principles of fracture mechanics to model the fracture of concrete containing macroscopic cracks.

The field of fracture mechanics was developed by Griffith (1920) to explain the failure of brittle materials. Linear elastic fracture mechanics (LEFM) predicts the rapid propagation of a microcrack through a homogeneous, isotropic, linear-elastic material. The theory uses the stress-intensity factor K that

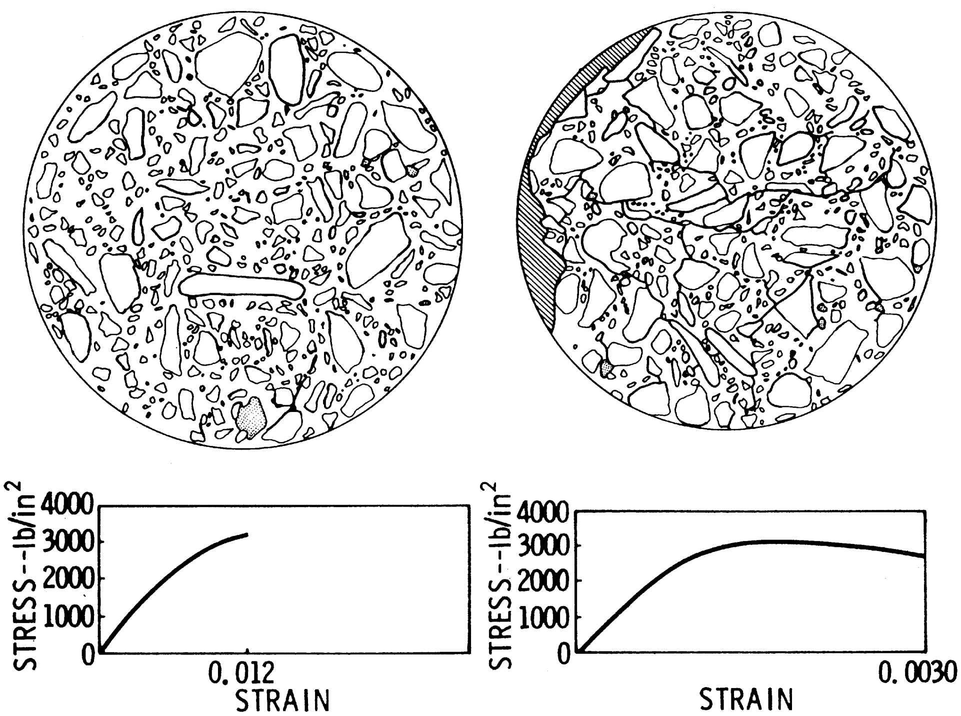

Fig. 2.1—Cracking maps and stress-strain curves for concrete loaded in uniaxial compression (Shah and Slate 1968).

represents the stress field ahead of a sharp crack in a structural member which is a function of the crack geometry and stress. K is further designated with subscripts, I, II, and III, depending upon the nature of the deformation at the crack tip. For a crack at which the deformation is perpendicular to the crack plane, K is designated as K_I, and failure occurs when K_I reaches a critical value K_{Ic}, known as the critical stress-intensity factor. K_{Ic} is a measure of the fracture toughness of the material, which is simply a measure of the resistance to crack propagation. Often the region around the crack tip undergoes nonlinear deformation, such as yielding in metals, as the crack grows. This region is referred to as the plastic zone in metals, or more generally as the fracture process zone. To properly measure K_{Ic} for a material, the test specimen should be large enough so that the fracture process zone is small compared with the specimen dimensions. For LEFM to be applicable, the value of K_{Ic} must be a material property, independent of the specimen geometry (as are other material properties, such as yield strength or compressive strength).

Initial attempts to measure K_{Ic} in concrete were unsuccessful because K_{Ic} depended on the size and geometry of the test specimens (Wittmann 1986). As a result of the heterogeneity inherent in cement paste, mortar, and concrete, these materials exhibit a significant fracture-process zone and the critical load is preceded by a substantial amount of slow crack growth. This precritical crack growth has been studied experimentally by several researchers (John and Shah 1986; Swartz and Go 1984; Bascoul, Kharchi, and Maso 1987; Maji and Shah 1987; Castro-Montero, Shah, and Miller 1990). This research has provided an improved understanding of the fracture process zone and has led to the development of more rational fracture criteria for concrete.

This chapter is divided into two sections. The first section on compressive microcracking presents the current knowledge of the response of concrete and its constituent materials under compressive loading and the role played by the various types of microcracks in this process. The second section discusses the applicability of both linear and nonlinear fracture mechanics models to concrete. A more comprehensive treatment of the fracture of concrete can be found in ACI 446.1R.

2.2—Compressive microcracking

During early microcracking research, a picture developed that closely linked the formation and propagation of microcracks to the load-deformation behavior of concrete. Before loading, volume changes in cement paste cause interfacial cracks to form at the mortar-coarse aggregate boundary (Hsu 1963; Slate and Matheus 1967). Under short-term compressive loads, no additional cracks form until the load reaches about 30% of the compressive strength of the concrete (Hsu, Slate, Sturman, and Winter 1963). Above this value, additional bond cracks are initiated throughout the matrix. Bond cracking increases until the load reaches about 70% of the compressive strength, at which time the microcracks begin to propagate through the mortar. Mortar cracking continues at an accelerated rate, forming continuous cracks parallel to the direction of compressive load, until the concrete is no longer able to sustain the load. The onset of mortar cracking is related to the sustained, or long-term, compressive strength. Derucher (1978) obtained a somewhat different picture of the microscopic behavior of concrete using the scanning electron microscope (SEM). He subjected dried concrete specimens to eccentric compressive loading within the SEM. He observed that microcracks that exist

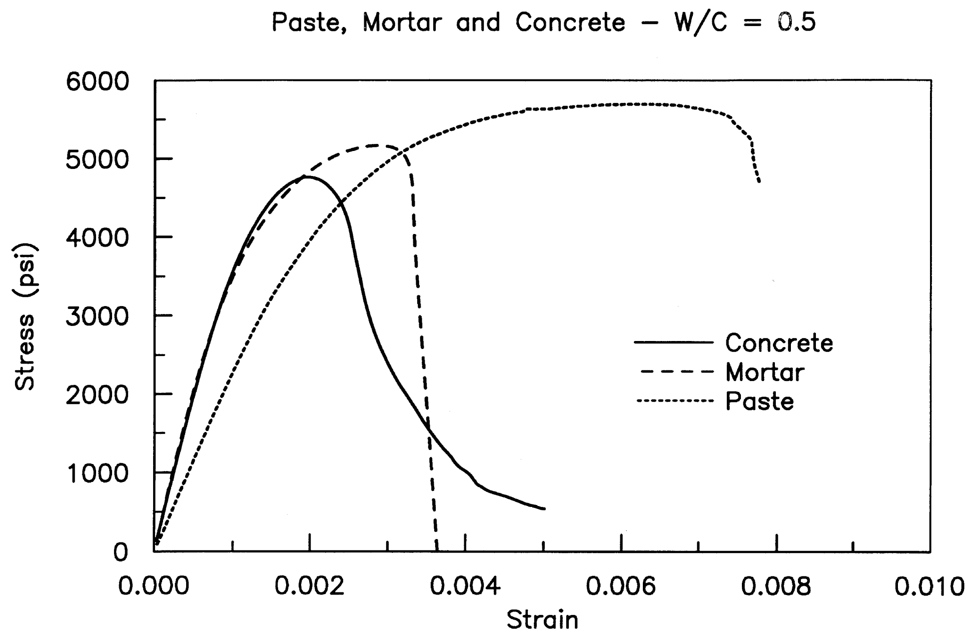

Fig. 2.2—Stress-strain curves for cement paste, mortar, and concrete; w/c *= 0.5 (Martin, Darwin, and Terry 1991).*

before loading are in the form of bond cracks, with extensions into the surrounding mortar perpendicular to the bond cracks. Under increasing compression, these bond cracks widen but do not propagate at loads as low as 15% of the strength. At about 20% of ultimate, the bond cracks begin to propagate, and at about 30%, they begin to bridge between one another. The bridging is almost complete at 45% of the compressive strength. At 75% of ultimate, mortar cracks start to join one another and continue to do so until failure.

In general, microcracking that occurs before loading has little effect on the strength of compressive strength of the concrete.

In studies of high-strength concrete, Carrasquillo, Slate, and Nilson (1981) concluded that it was more appropriate to classify cracks as simple (bond or mortar) and combined (bond and mortar) and that the formation of combined cracks consisting of more than one mortar crack signaled unstable crack growth. They observed that the higher the concrete strength, the higher the strain (relative to the strain at peak stress) at which this unstable crack growth is observed. They observed less total cracking in high-strength concrete than normal-strength concrete at all stages of loading.

Work by Meyers, Slate, and Winter (1969), Shah and Chandra (1970), and Ngab, Slate, and Nilson (1981) demonstrated that microcracks increase under sustained and cyclic loading. Their work indicated that the total amount of microcracking is a function of the total compressive strain in the concrete and is independent of the method in which the strain is applied. Suaris and Fernando (1987) also showed that the failure of concrete under constant amplitude cyclic loading is closely connected with microcrack growth. Sturman, Shah, and Winter (1965) found that the total degree of microcracking is decreased and the total strain capacity in compression is increased when concrete is subjected to a strain gradient.

Since the early work established the existence of bond and mortar microcracks, it has been popular to attribute most, if not all, of the nonlinearity of concrete to the formation of these microscopic cracks (Hsu, Slate, Sturman, and Winter 1963; Shah and Winter 1966; Testa and Stubbs 1977; Carrasquillo, Slate, and Nixon 1981). A cause and effect relationship, however, has never been established (Darwin 1978). Studies by Spooner (1972), Spooner and Dougill (1975), Spooner, Pomeroy, and Dougill (1976), and Maher and Darwin (1982) indicate that the degree of microcracking can be taken as an indication of the level of damage rather than as the controlling factor in the concrete's behavior.

Experimental work by Spooner (1972), Spooner and Dougill (1975), Spooner, Pomeroy, and Dougill (1976), and Martin, Darwin, and Terry (1991) indicates that the nonlinear compressive behavior of concrete is strongly influenced by the nonlinear behavior of cement paste. As illustrated in Fig. 2.2, cement paste under compression is not an elastic, brittle material as stated in the past, but a nonlinear material with a relatively high strain capacity. The nonlinear behavior of cement paste can be tied to damage sustained by the paste, even at very low stresses.

Using a cyclic loading procedure, Spooner (1972), Spooner and Dougill (1975), and Spooner, Pomeroy, and Dougill (1976) demonstrated that both paste and concrete undergo measurable damage at strains (0.0004) at which an increase in bond and mortar microcracking cannot be detected. The level of damage can be detected at low loads by using an energy method and by a change in the initial modulus of elasticity for each load cycle. The process of damage is continuous up to failure.

The physical nature of damage that occurs in cement paste, like that in concrete, appears to be related to cracking. This point was first made by Spooner, Pomeroy, and Dougill (1976) based on volumetric strain measurements and then by

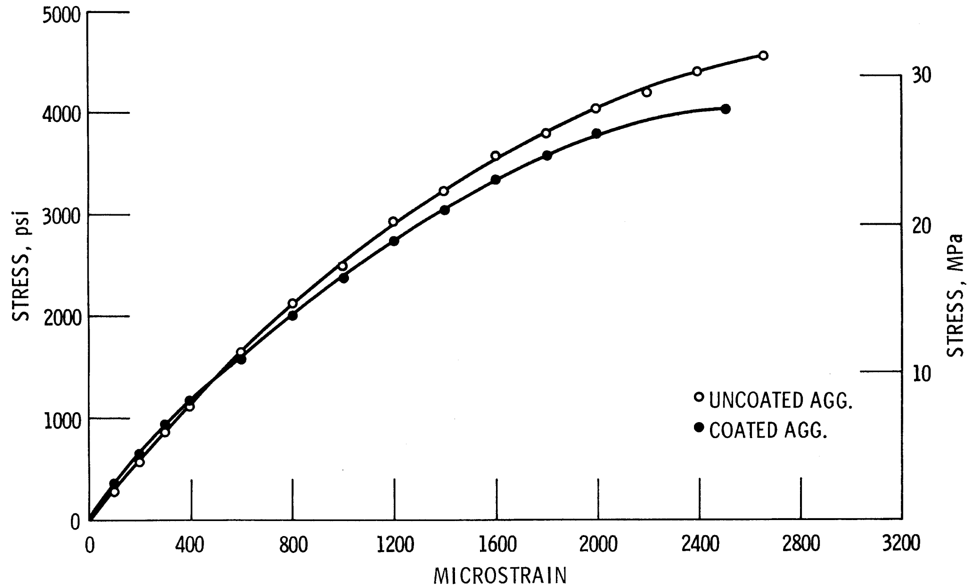

Fig 2.3—Stress-strain curves as influenced by coating aggregates (Darwin and Slate 1970).

Yoshimoto et al. (1972) and Yoshimoto, Ogino, and Kawakami (1976) who reported the formation of "hair-shaped" and "void-shaped" cracks in paste under flexure and compressive loading. The relationship between nonlinear deformation and cracking in cement paste is now firmly established by the work of Attiogbe and Darwin (1987, 1988).

Studies of the stress-strain behavior of concrete under cyclic compressive load (Karsan and Jirsa 1969; Shah and Chandra 1970) indicated the concrete undergoes rapid deterioration once the peak stress exceeds 70% of the short-term compressive strength of the concrete. In their study of cyclic creep, Neville and Hirst (1978) found that heat is generated even when specimens are cycled below this level. They attributed the heat to sliding at the interfacial boundary. The work of Neville and Hirst, along with the work of Spooner, suggests that it can be possible that the heat measured is due to some microscopic sliding within the paste.

Several studies have attempted to establish the importance of interfacial bond strength on the behavior of concrete in compression. Two studies seemed to indicate a very large effect, thus emphasizing the importance of interfacial strength on concrete behavior in compression (Shah and Chandra 1970; Nepper-Christensen and Nielsen 1969). These studies used relatively thick, soft coatings on coarse aggregate to reduce the bond strength. Because these soft coatings isolated the aggregate from the surrounding mortar, the effect was more like inducing a large number of voids in the concrete matrix.

Two other studies (Darwin and Slate 1970; Perry and Gillott 1977) that did not isolate the coarse aggregate from the mortar indicated that interfacial strength plays only a minor role in controlling the compressive stress-strain behavior of concrete. Darwin and Slate (1970) used a thin coating of polystyrene on natural coarse aggregate. They found that a large reduction in interfacial bond strength causes no change in the initial stiffness of concrete under short-term compressive loads and results in about a 10% reduction in the compressive strength, compared with similar concrete made with aggregate with normal interfacial strength (Fig. 2.3). Darwin and Slate also monitored microcracking. In every case, however, the average amount of mortar cracking was slightly greater for specimens made with coated aggregate. This small yet consistent difference may explain the differences in the stress-strain curves. Perry and Gillott (1977) used glass spheres with different degrees of surface roughness as coarse aggregate. Their results also indicate that reducing the interfacial strength of the aggregate decreases the compressive strength by about 10%.

Work by Carino (1977), using polymer-impregnated concrete, corroborated these last two studies. Carino found that polymer impregnation did not increase the interfacial bond strength but did increase the compressive strength of concrete. He attributed the increase in strength to the polymer's effect on mortar strength, therefore downgrading the importance of interfacial bond.

The importance of mortar in controlling the stress-strain behavior of concrete is illustrated by the finite-element work of Buyukozturk (1970) and Maher and Darwin (1976, 1977). Buyukozturk (1970) used a finite-element representation of a physical model of concrete. The model treated mortar (in compression) and aggregate (in compression and tension) as linear elastic materials while allowing cracks to form in the mortar and at mortar aggregate boundaries. Buyukozturk simulated the overall crack patterns under uniaxial loading. His finite-element model, however, could not duplicate the full nonlinear behavior of the physical model using the formation of interfacial bond cracks and mortar cracks as the only nonlinear effects. Maher and Darwin (1976, 1977) have shown that a very close representation of the actual stress-strain behavior can be obtained using a nonlinear representation for the mortar constituent of the physical model.

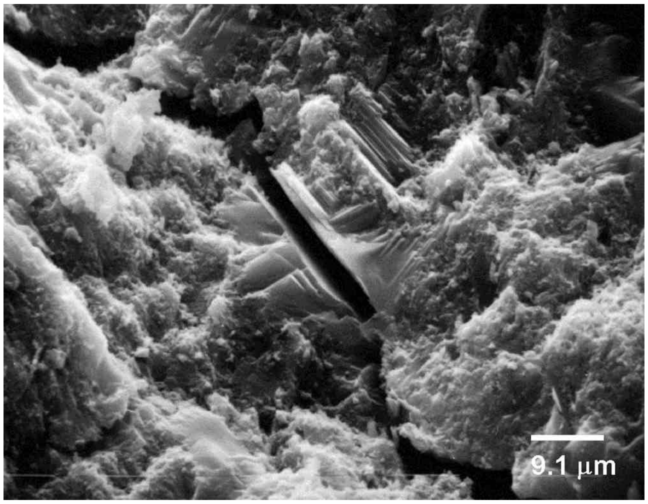

Fig 2.4—Crack through calcium silicate-hydrate and calcium hydroxide in cement paste (Attiogbe and Darwin 1987).

Maher and Darwin also studied the behavior of the mortar constituent of concrete under monotonic and cyclic compression (1982). Degradation in mortar was shown to be a continuous process and a function of both total strain and load history. The study indicated that residual strain as well as the change in the initial modulus of elasticity are good measures of structural change within the material. Accumulations of residual strain were obtained for values of maximum strain as low as 0.00027. The work showed that the maximum strain alone does not control the degradation of mortar in compression and that the total strain range (both loading and unloading) adds to the degradation in stiffness and accumulation of residual strain. Their work concludes as was previously observed (Meyers, Slate, and Winter 1969; Shah and Chandra 1970; Ngab, Slate, and Nilson 1981) that bond and mortar microcracking in concrete is a function of the compressive strain in the concrete and is independent of the method in which the strain is applied. Because the maximum strain does not appear to completely control degradation, factors other than bond and mortar cracks can dominate the degradation of concrete during cyclic loading.

Martin, Darwin, and Terry (1991) studied the behavior of paste, mortar, and concrete under cyclic and short-term sustained compression. They found a great similarity in the behavior of concrete and its mortar constituent although the bond and mortar microcracking found in concrete were not observed in the mortar specimens. Of the three materials studied, cement paste has the greatest strain capacity and strength, followed by mortar and concrete (Fig. 2.2).

To understand the compressive response of the cement paste and mortar constituents of concrete, Attiogbe and Darwin (1987, 1988) used the SEM to study submicroscopic cracking under uniaxial compression (*Fig. 2.4*). Materials with water-cement ratios (*w/c*) of 0.3, 0.5, and 0.7 were subjected to monotonic, cyclic, and short-term sustained loading. Their observations showed that most cracks in cement paste range in width from 0.2 to 0.7 µm (8 to 28 × 10^{-5} in.) and in length from 10 to over 200 µm (4 to over 80 × 10^{-4} in.). Tests on mortar showed that nonloaded specimens had about 40% of the crack density of the corresponding cement paste specimens. As the applied strain was increased, however, the crack density increased more rapidly in the mortar, eventually surpassing the value obtained in the cement paste. While sand particles can reduce crack density due to volume changes in cement paste, these results indicate that they act as stress raisers when load is applied. This increase in crack density under applied loading may explain the reduction in ultimate strain capacity exhibited in *Fig. 2.2* (Martin, Darwin, and Terry 1991) for mortar, compared with cement paste at the same *w/c*.

Using analytical procedures, Attiogbe and Darwin (1988) established that a significant portion of the nonlinear strain in cement paste and mortar can be attributed to the microcracks within the cement paste.

Overall, the damage to cement paste in compression seems to play a dominant role in controlling the primary stress-strain behavior of concrete under compression. In normalweight concrete, aggregate particles act as stress risers, increasing the initial stiffness and decreasing the strength of the paste and controlling the compressive strength of the concrete. An understanding of concrete behavior in compression, thus, requires an understanding of both the behavior of cement paste in compression and the interaction of cement paste with aggregate particles.

2.3—Fracture

2.3.1 *Applicability of linear elastic fracture mechanics—* The fracture toughness of a brittle material, which is characterized by a critical stress-intensity factor K_{Ic} can be measured by using a single-edge notched beam subjected to a monotonically increasing load. The load is applied so that a constant rate of crack-mouth-opening displacement (CMOD) is maintained. If the load-CMOD curve is linear, LEFM can be used to calculate K_{Ic} based on the measured maximum load and the length of the crack just before failure (ASTM E 399). K_{Ic} is used in the design of metal structures to prevent brittle failure where fatigue crack growth is expected to occur. For LEFM to be applicable, however, the value of K_{Ic} should be a material property independent of the specimen geometry.

When K_{Ic} was calculated for concrete, as described previously, significant effects of the size and geometry of the test specimen were observed by many investigators (Kaplan 1961; Naus and Lott 1969; Higgins and Bailey 1976). The data presented in Fig. 2.5 (Higgins and Bailey 1976) shows that K_{Ic} increases with the specimen depth. Such results led many to question the applicability of LEFM to concrete. Results obtained from single-edge notched beams were also analyzed by several investigators to determine if concrete displays any notch sensitivity. Notch sensitivity can be expressed as the ratio of net stress at the crack tip to the modulus of rupture of an unnotched specimen. Data on the notch sensitivity of hardened cement paste, mortar, and concrete are shown in Fig. 2.6 (Higgins and Bailey 1976; Kesler, Naus, and Lott 1972; Shah and McGarry 1971; Gjørv, Sorenson, and Arneson 1977; Hillemeier and Hilsdorf 1977). The specimens showing no notch sensitivity are likely the result of deficiencies in the

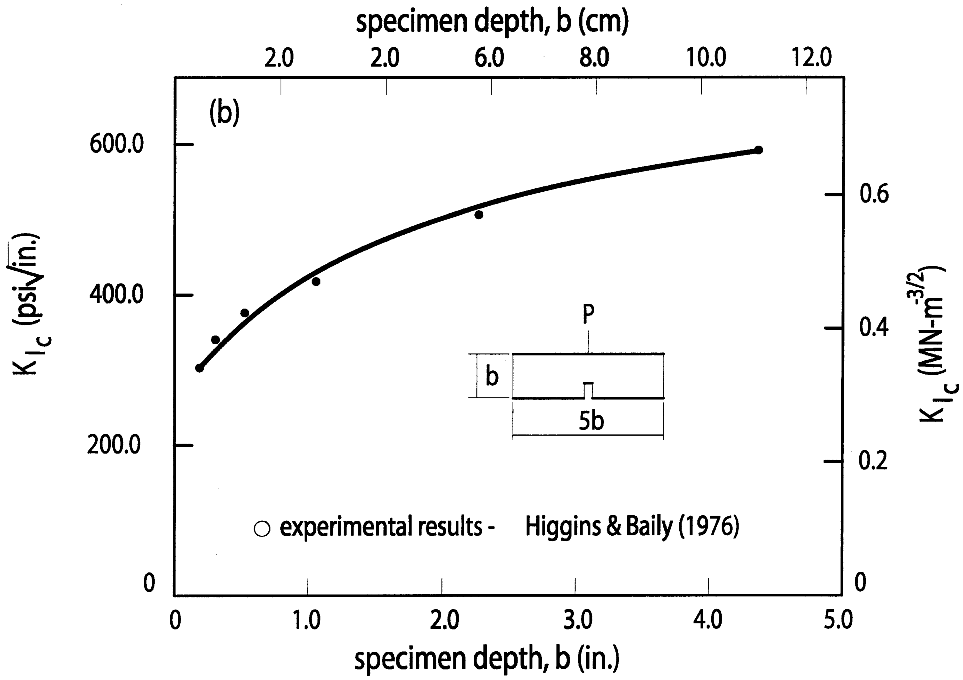

Fig. 2.5—Size effect on stress-intensity factor (based on data from Higgins and Bailey 1976).

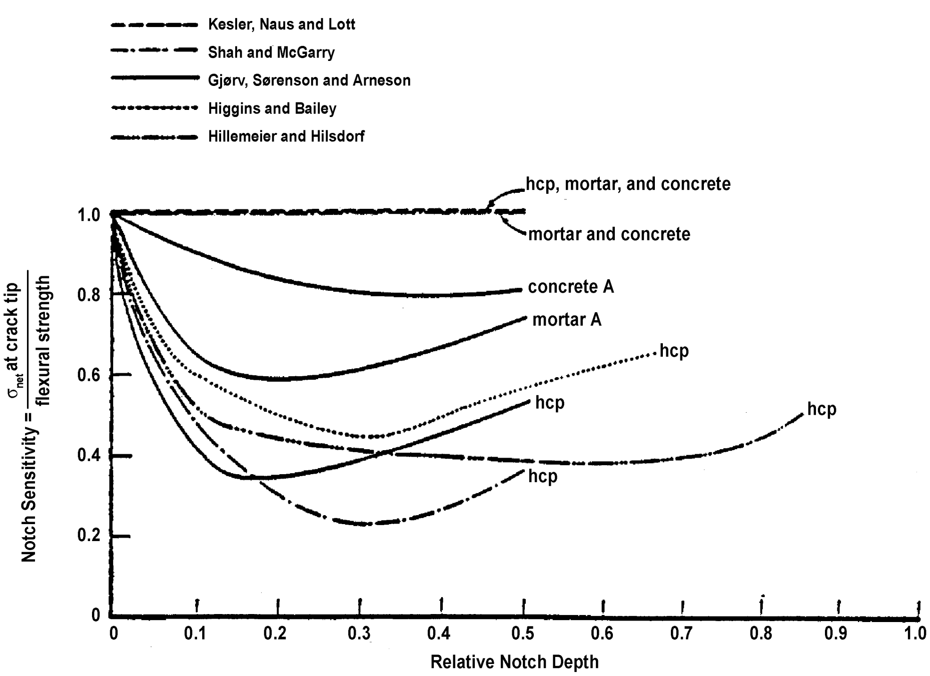

Fig. 2.6—Effect of relative notch depth on notch sensitivity (based on data from Higgins and Bailey 1976; Kesler, Naus, and Lott 1972; Shah and McGarry 1971; Gjørv, Sorenson, and Arneson 1977; Hillemeier and Hilsdorf 1977).

test methods, as explained by Gjørv et al. (1977). The results indicate, however, that both mortar and concrete display less notch sensitivity than hardened cement paste. It is widely accepted today that this lower notch sensitivity for the relatively more heterogeneous materials, particularly concrete, is due to the fact that LEFM is inapplicable for laboratory-size specimens of these materials (Gjørv et al. 1977; Wittmann 1986). It is also widely accepted (Linsbauer et al. 1989a, 1989b), however, that LEFM is a valid tool for analyzing large concrete structures, such as dams, where the heterogeneities and the fracture process zone are small compared with the structure dimensions.

2.3.2 *Nonlinear fracture models for concrete*—The inapplicability of LEFM to laboratory-size concrete specimens is the result of the heterogeneity inherent in the concrete. This heterogeneity results in a relatively large fracture process zone that results in a substantial amount of crack growth (crack extension) preceding the critical (maximum) load and

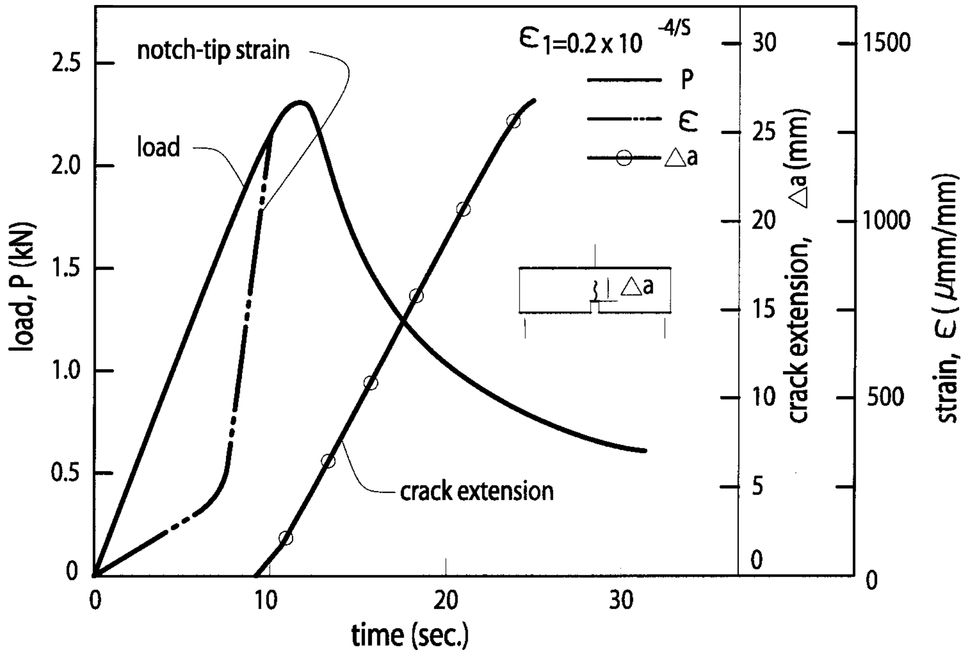

Fig. 2.7—Precritical crack growth (John and Shah 1986).

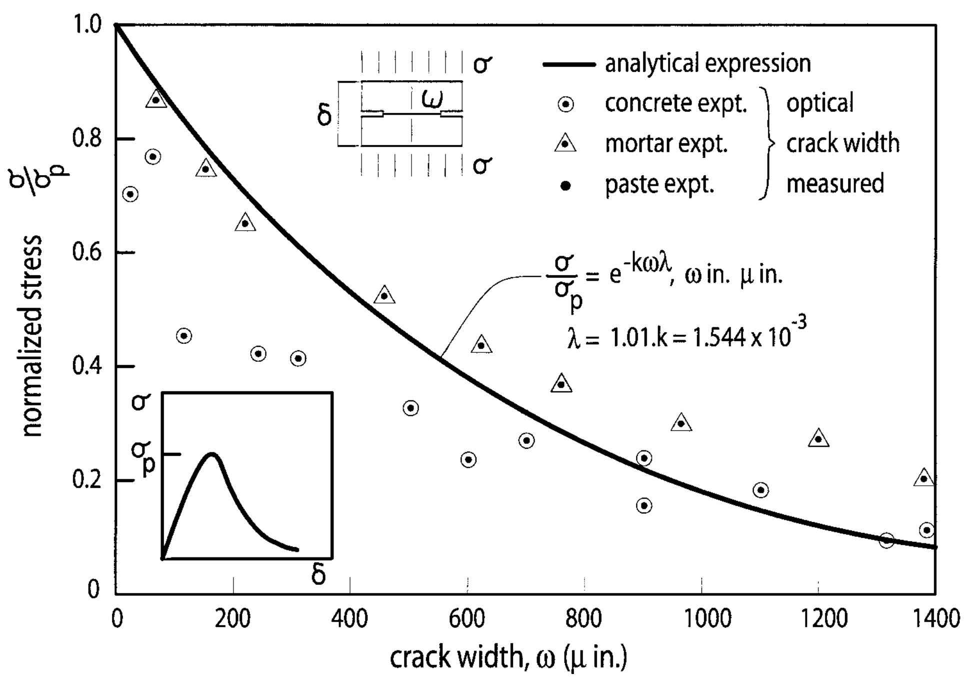

Fig. 2.8—Normalized peak stress versus crack width in unaxial tension (Gopalratnam and Shah 1986).

is responsible for the strong dependence of K_{Ic} on the size and geometry of test specimens. Precritical crack growth (crack extension) for a notched beam test is shown in Fig. 2.7, where the crack growth ahead of the notch was continuously monitored using a specially developed brittle crack gage (John and Shah 1986).

The fracture process zone in concrete is substantially different from the plastic zone in metals. For metals, the plastic zone is defined as a zone where the material has yielded ahead of the crack. LEFM is based on the assumption that the plastic zone is substantially smaller than the dimensions of the test specimen. Laboratory-size specimens satisfy this criterion for metals. For concrete, Bažant (1979) stated that the fracture process zone has a negligible effect if the cross-sectional dimensions of a member is at least 100 times the maximum aggregate size, which would lead to prohibitive size requirements. For instance, concrete with 19 mm (3/4 in.) aggregates would require a beam with a depth of at least of 2 m (6 ft). In view of these specimen size requirements, when LEFM is not applicable for many of the fracture tests that have been conducted on concrete. Therefore, if laboratory-size specimens are used to evaluate the fracture toughness of

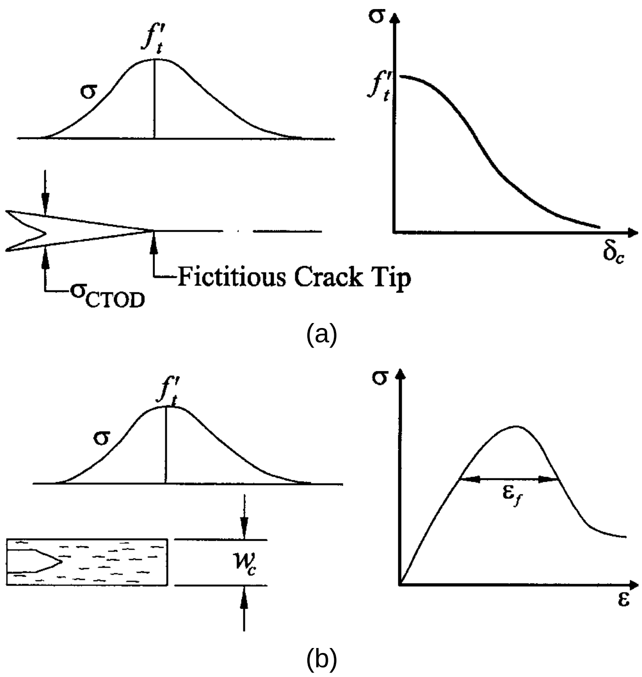

Fig 2.9—(a) Fictitious crack model; and (b) crack band model.

concrete, it is imperative that the effect of the process zone is considered.

Figure 2.8 shows the results of a uniaxial tensile test conducted by Gopalaratnam and Shah (1986). The average (surface) crack opening displacements during this test were measured microscopically. The peak of the curve, shown in Fig. 2.8 at zero displacement, is assumed to be equal to the tensile strength of the concrete, and the area under the curve is considered to be the fracture energy of the concrete G_f.

Hillerborg, Modeer, and Petersson (1976) developed the fictitious crack model, which has been used for finite element analysis of concrete fracture. Figure 2.9(a) illustrates the basic concept of the approach. For a beam in flexure, the left-hand portion of Fig. 2.9(a) shows the variation in stress along the crack path, reaching a peak at the fictitious crack tip, where the stress is equal to f_t' (the tensile strength of the concrete), and the CTOD is zero. Moving to the left of the peak, the stress drops as the crack opens, with the real crack ending at the point where the stress across the crack has dropped to zero. To the right, the stress drops in advance of the crack. The material between the real and fictitious crack tip transmits tensile stress as defined by a (softening) stress-crack opening displacement curve, such as Fig. 2.8 and the right-hand portion of Fig. 2.9(a). If the shape of this softening curve is assumed to be fixed, then the fracture of the concrete is completely characterized by f_t' and G_f.

Bažant and Oh (1983) developed a crack band model to account for the fracture process zone in concrete in a smeared manner through the introduction of a strain-softening constitutive relation. In this model, the crack front has a width of W_c that is equal to the width of a single finite element (Fig. 2.9[b]). The crack band model is designed to produce a response

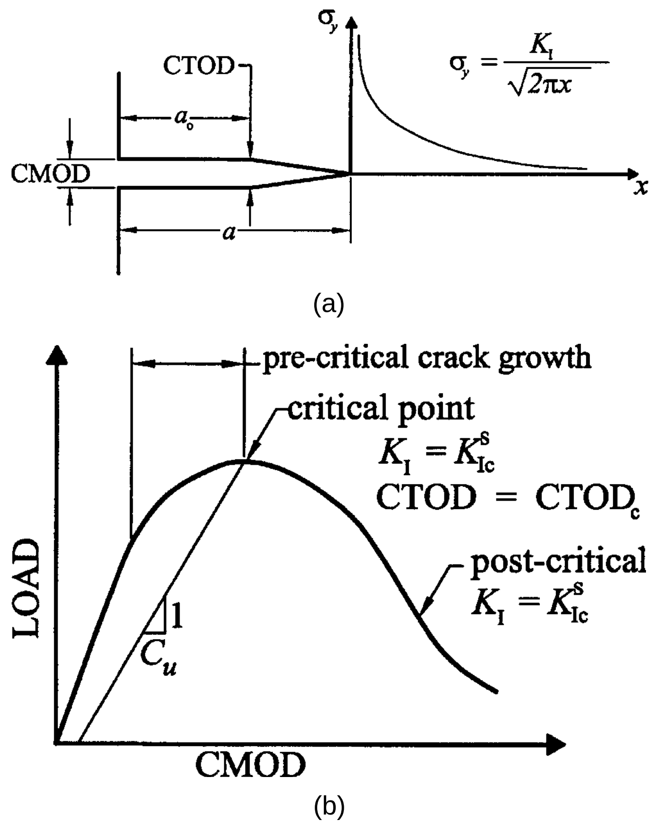

Fig. 2.10—(a) Effective Griffith crack; and (b) typical plot of load versus CMOD (Jenq and Shah 1987).

in a finite element model that essentially matches the results of the fictitious crack model. In the crack band model, the crack is represented by an equivalent change in material properties within an element. In Fig. 2.9(b), the figure on the left-hand side is analogous to the figure on the left-hand side of Fig. 2.9(a), showing a variation in stress along the crack front as a function of location. The right-hand portion of Fig. 2.9(b) shows the stress-strain curve that defines the behavior of an element as the crack grows. The rising portion of the stress-strain curve is used to simulate a slowly opening crack. The product of the strain ε_f shown in Fig. 2.9(b) and the width of the finite element W_c is equal the crack opening displacement δ_c shown in *Fig. 2.9*(a). When used in conjunction with the two material properties used for the fictitious crack model, G_f and f_t', the two procedures produce nearly identical results (Leibengood, Darwin, and Dodds 1986).

2.3.3 *Nonlinear fracture models based on adaptation of LEFM*—Several investigators have proposed the use of an effective crack length a_e to account for the fracture process zone (Catalano and Ingraffea 1982; Nallathambi and Karihaloo 1986; Refai and Swartz 1987). The effective crack length is obtained from the reduction in stiffness at the peak load in a three-point bend test. The effective crack depends on the maximum grain size of the aggregate and on the geometry of the specimen. The term a_e is obtained by comparing the compliance of the test specimen with compliances obtained from a series of prenotched beams. When K_{Ic} is calculated using the effective crack length, a size-independent value is

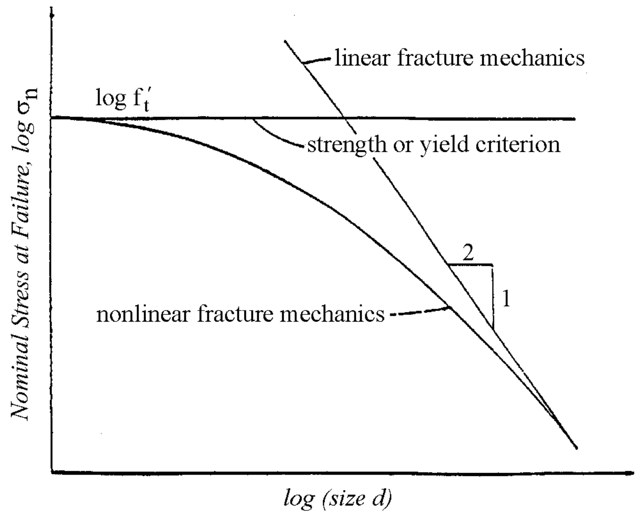

Fig. 2.11—Size-effect law (Bažant, Kim, and Pfeiffer 1986).

obtained. Refai and Swartz (1987) developed empirical equations that relate effective crack length with specimen geometry and material properties.

Jenq and Shah (1987) proposed a method to determine the effective crack length, which is then used to calculate a critical stress-intensity factor K_{Ic}^{s} and a critical crack tip opening displacement (CTOD). Figure 2.10 illustrates the effective crack-length concept. The effective crack length concept itself is the sum of a measurable crack, visible on the side of a specimen, plus the additional crack length represented by the fracture process zone. The effective crack length is evaluated using the unloading compliance measurement C_u of the load-CMOD curve at the point of maximum load, as shown in Fig. 2.10(b). Jeng and Shah found that the effective crack length calculated from compliance measurements is the same as that obtained using LEFM and assuming that CTOD has a critical value, which was found to be independent of the size and geometry of the beams tested and may be considered to be a valid fracture parameter.

2.3.4 *Size effect of fracture*—The effect of structural size on the fracture of concrete is perhaps the most compelling reason for using fracture mechanics (ACI 446.1R).

For blunt fracture (as occurs in a crack with a diffuse fracture process zone in materials such as concrete), the total potential-energy release caused by fracture in a given structure depends on the length of the fracture and the area traversed by the fracture process zone so that the size of the fracture process zone is constant and independent of the size of the structure. Dimensional analysis then shows that the structural size effect for geometrically similar specimens or structures is governed by the simple relation (Bažant, Kim, and Pfeiffer 1986)

$$\sigma_N = \frac{Bf_t'}{\sqrt{(1 + d/d_o)}} \tag{2-1}$$

where

σ_N = P/bd = nominal stress at failure;
P = maximum load (that is, failure load);
b = thickness;
d = characteristic dimension of the specimen or structure;
f_t' = direct tensile strength; and
B, d_o = empirical constants, d_o being a certain multiple of the maximum size of inhomogeneities in the material d_a.

The value of B and the ratio of d_o/d_a depends only on the shape of the structure, not on its size. Figure 2.11 shows the relationship between nominal stress at failure and size.

If the structure is very small, the second term in parentheses, d/d_o of Eq. (2-1), is negligible compared with 1, and $\sigma_N = B f_t'$ is the failure condition that represents the strength criterion and corresponds to the horizontal line in Fig. 2.11. If the structure is very large, 1 is negligible compared with d/d_o and σ_N = constant/$\sqrt{d}$. This is the typical size effect in LEFM; it corresponds to the inclined straight line in Fig. 2.11. According to Eq. (2-1), the size effect in blunt fracture represents a gradual transition from the strength criterion to the energy criterion of LEFM.

The size-effect law has been used by Bažant and Sun (1987); Bažant and Sener (1988); and Bažant, Sener, and Pratt (1988) to predict the size effects for shear, torsion, and bond pullout testing of concrete.

2.3.5 *Effect of material properties on fracture*—Certain material properties, especially *w/cm*, play an important role in controlling the compressive strength and durability of concrete. The effect of these material properties on the fracture of concrete are not certain; however, some studies have specifically addressed this question. Early work by Naus and Lott (1969) indicated that the fracture toughness of cement paste and mortar increases with decreasing *w/cm*, but *w/cm* has little effect on the fracture toughness of concrete. Naus and Lott found that K_{Ic} increases with age and decreases with increasing air content for paste, mortar, and concrete. The fracture toughness of mortar increases with increasing sand content, and the fracture toughness of concrete increases with an increase in the maximum size of the coarse aggregate. Gettu, Bažant, and Karr (1990), in a study of the fracture properties of high-strength concrete, made a number of observations that match those obtained in the earlier work. They observed that the fracture toughness and fracture energy obtained with high-strength concrete is not much higher than that for lower-strength concrete, and any increase that occurs is at a rate less than in proportion to the square root of compressive strength. The work by Gettu, Bažant, and Karr (1990) was carried out with mixtures that maintained a constant maximum-size aggregate. When the results of their work are combined with the typical procedure of using smaller maximum-size aggregate for high-strength concrete, it becomes clear that improvements in compressive strength, obtained with the use of increased cement contents, mineral admixtures, high-range water-reducers, and with the accompanying reduction in total aggregate volume, will not increase fracture toughness. The result is that structural members made with high-strength concrete will exhibit a lower-than-expected capacity when the member strength depends on the concrete tensile strength, and the design is based on $\sqrt{f_c'}$. Specific examples are flexural cracking, shear strength, and bond strength between concrete and reinforcing steel. The impact of using high-strength concrete on these load-carrying mechanisms needs additional study.

CHAPTER 3—CONTROL OF CRACKING DUE TO DRYING SHRINKAGE

3.1—Introduction

Drying shrinkage of concrete is the reduction in volume caused by the loss of water. Drying shrinkage can be defined as the time-dependent linear strain at constant temperature measured on an unloaded specimen that is allowed to dry. From a structural point of view, there is no need to separate drying shrinkage from other kinds of phenomena, such as carbonation shrinkage and autogenous shrinkage. A typical value for the final shrinkage strain of concrete in structures is 600×10^{-6}. Because the concrete tensile-strain capacity can be 150×10^{-6} or less, cracking will result if the shrinkage is restrained in a concrete member. There is a high degree of uncertainty in predicting shrinkage of concrete structures, however, because this property varies considerably with many parameters, including concrete composition, source of aggregate, ambient relative humidity, specimen geometry, and more specifically, the ratio of the exposed surface to the volume of the structural element. Further, the slow development of shrinkage over time makes it difficult to obtain an accurate prediction for a given concrete from short-term laboratory measurements. As a result, a coefficient variation of 20% or more can be expected in predicting long-term shrinkage.

Before true moisture equilibrium has been reached within a member cross section, internal shrinkage restraint occurs because of moisture gradients. Consequently, self-equilibrating internal stresses are present with tension on the surface and compression in the interior. This stress condition can cause cracking if not relieved by creep.

Shrinkage and creep are often responsible for excessive deflections and curvature, losses in prestress, and redistribution of internal stresses and reactions in statically indeterminate members. If not controlled, drying shrinkage can lead to serviceability problems, such as excessive deflections, and durability problems, such as freeze-thaw deterioration and corrosion at cracks.

Good design and construction practices can minimize the amount of cracking and eliminate or control the visible large cracks by minimizing the restraint using adequate reinforcement and contraction joints. Further information can be found in ACI 209R. Cracking due to drying shrinkage can never be eliminated in most structures. This chapter covers cracking of hardened concrete due to drying shrinkage, factors influencing shrinkage, control of cracking, and the use of expansive cements to minimize cracking. Construction practices and specifications to minimize drying shrinkage are covered in Chapter 8.

3.2—Cause of cracking due to drying shrinkage

The contraction (due to drying shrinkage) of a concrete component within a structure is always subject to some degree of restraint from either the foundation, another part of the structure, or the reinforcing steel embedded in the concrete. The combination of shrinkage and restraint develops tensile stresses within the concrete. Due to the inherent low tensile strength of concrete, cracking will often occur (Fig. 3.1).

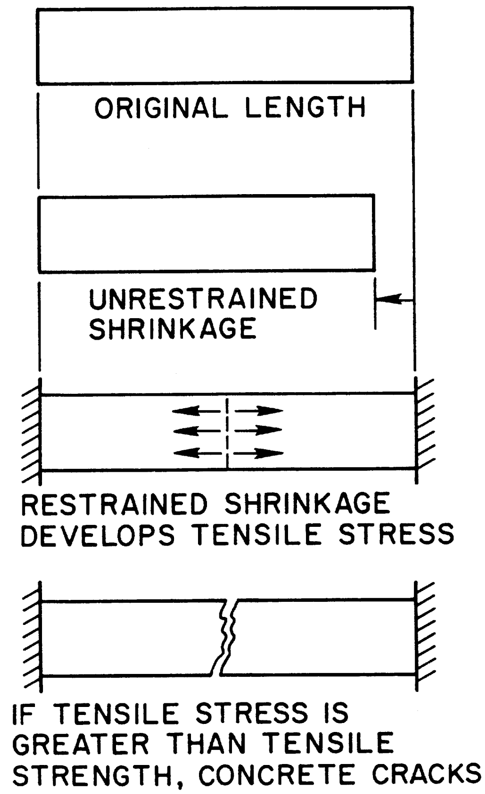

Fig. 3.1—Cracking of concrete due to drying shrinkage.

Additional restraint arises from nonuniform shrinkage. Because drying occurs nonuniformly from the surface towards the concrete core, shrinkage will create internal tensile stresses near the surface and compression in the core. Differential shrinkage can result in warping and surface cracks. The surface cracks can, with time, penetrate deeper into the concrete member as the interior portion is subject to additional shrinkage.

As illustrated in Fig. 3.2, the tensile stress induced by restraining drying shrinkage is reduced with time due to creep or stress relaxation. Cracks develop only when the net tensile stress reaches the tensile strength of concrete. The creep relief decreases with age, however, so that the cracking tendency becomes greater with increased time.

3.3—Drying shrinkage

When concrete dries, it contracts or shrinks. When it is wetted, it expands. The expansion does not occur to the same extent as shrinkage. These volume changes, along with changes in moisture content, are an inherent characteristic of hydraulic-cement concrete. The change in moisture content of cement paste causes concrete to shrink or swell. Aggregate reduces the unit volume of cement paste and provides an internal restraint that significantly reduces the magnitude of these volume changes in concrete.

In addition to drying shrinkage, the cement paste is also subject to carbonation shrinkage. Shrinkage results from the

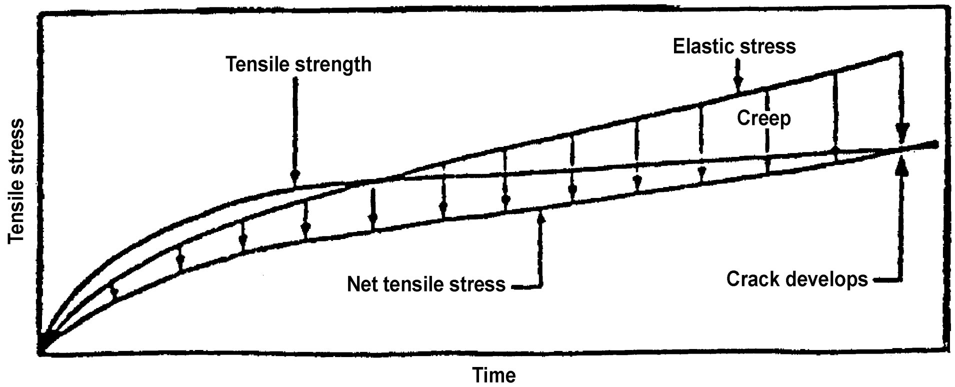

Fig. 3.2—Effect of creep on tensile stress.

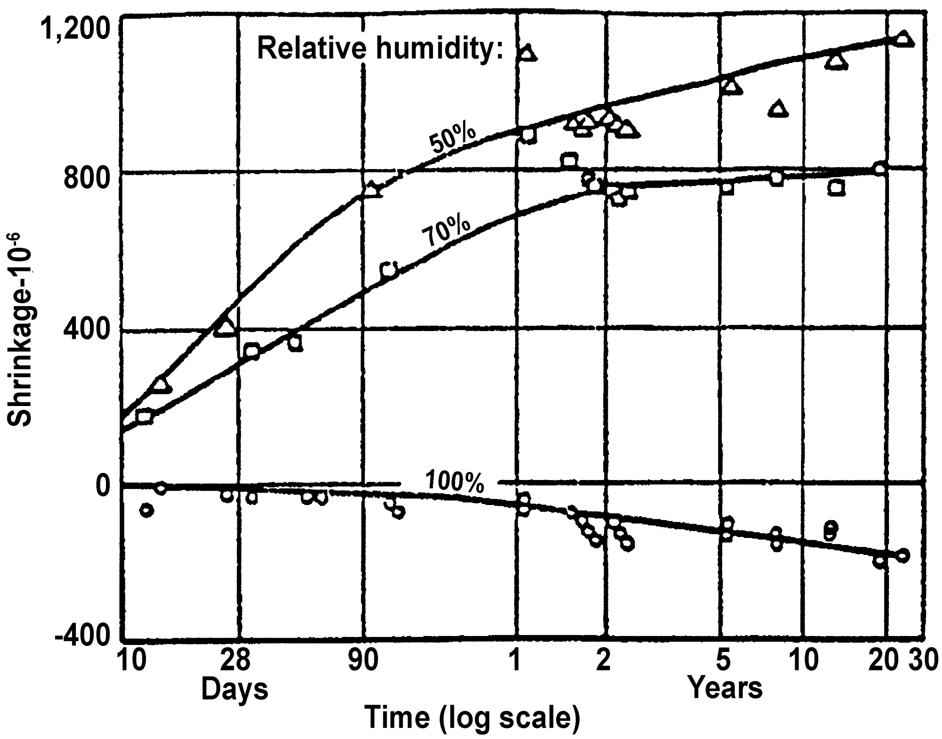

Fig. 3.3—Relations between shrinkage and time for concretes stored at different relative humidities. Time reckoned since end of wet curing at 28 days (Troxell, Raphael, and Davis 1958).

effects of carbon dioxide on the chemical changes of calcium-silicate hydrate and crystalline-hydration products and the drying of the pores by removing absorbed water. Calcium hydroxide will form calcium carbonate by reacting with atmospheric carbon dioxide. Because carbon dioxide does not penetrate more than about 12 mm (0.5 in.) into the surface of high-quality concrete with low porosity, carbonation shrinkage is of minor importance in the overall shrinkage of most concrete structures. Carbonation does, however, play an important role in the shrinkage of small laboratory test specimens and structures constructed with low-quality, porous concrete, particularly when subjected to long-term exposure to drying. The amount of carbonation shrinkage observed on a small laboratory specimen can be greater than the shrinkage of the concrete in the structure. This effect results from the greater surface area to volume ratio in smaller specimens. Shrinkage due to carbonation is discussed in detail by Verbeck (1958).

3.4—Factors controlling drying shrinkage of concrete

The major factors controlling ultimate drying shrinkage of concrete include relative humidity, aggregate type and content (or paste content), water content, and *w/cm*. The rate of moisture loss and shrinkage of a given concrete is influenced by the size of the concrete member, the relative humidity, distance from the exposed surface, and drying time.

3.4.1 *Relative humidity and drying time*—Relative humidity has a major influence on ultimate shrinkage and the rate of

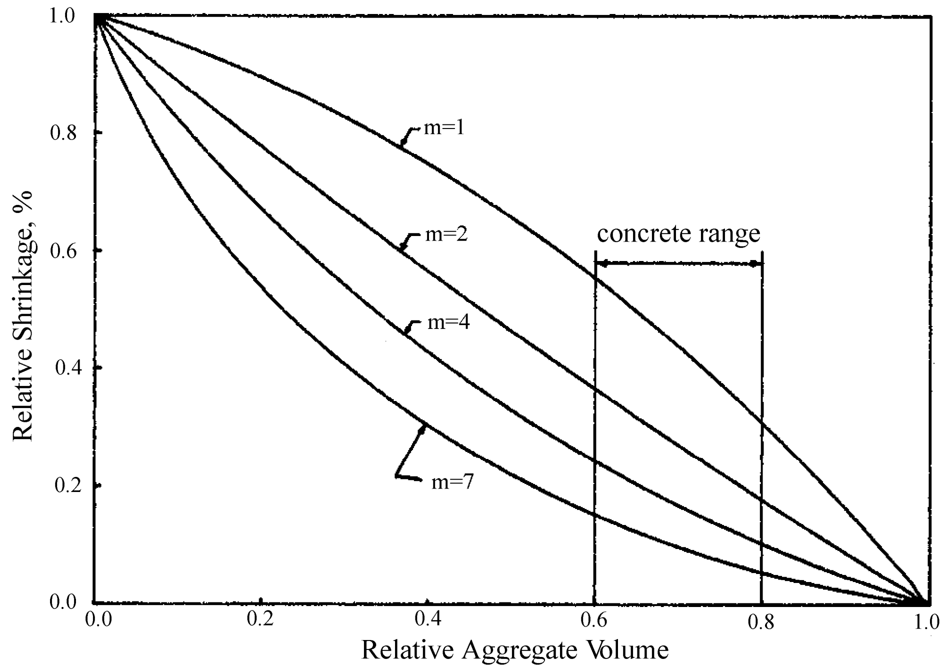

Fig. 3.4—Effect of relative aggregate content and modulus ratio on drying shrinkage of concrete (Hansen and Almudaiheem 1987).

Table 3.1—Effect of aggregate type on concrete shrinkage (after Carlson [1938])

Aggregate	Specific gravity	Absorption	1-year shrinkage, %
Sandstone	2.47	5.0	0.116
Slate	2.75	1.3	0.068
Granite	2.67	0.8	0.047
Limestone	2.74	0.2	0.041
Quartz	2.66	0.3	0.032

shrinkage. Results by Troxell, Raphael, and Davis (1958) showed that the lower the relative humidity, the greater the ultimate shrinkage and rate of shrinkage (Fig. 3.3). Figure 3.3 also illustrates that expansion occurs if concrete is exposed to a continuous supply of water; this process is known as swelling. Swelling is small compared with shrinkage in ordinary concrete and occurs only when the relative humidity is maintained above 94% (Lorman 1940). Swelling can, however, be significant in lightweight concrete (Neville and Brooks 1985). Figure 3.3 also shows that drying is a slow process. It can take many years before ultimate shrinkage is reached because the loss of water from hardened concrete is diffusion controlled.

3.4.2 *Influence of quantity and type of aggregate on shrinkage*—Concrete shrinkage is due primarily to shrinkage of the hardened cement paste. The presence of aggregate in concrete reduces the total shrinkage by providing elastic restraint to paste shrinkage. Concrete shrinkage, however, is not solely related to the relative aggregate content; there is another effect due to the ratio of elastic modulus of aggregate to that of the hydrated paste. When using high-quality aggregates, which are characterized mainly by low absorption capacity, this ratio is typically between four and seven

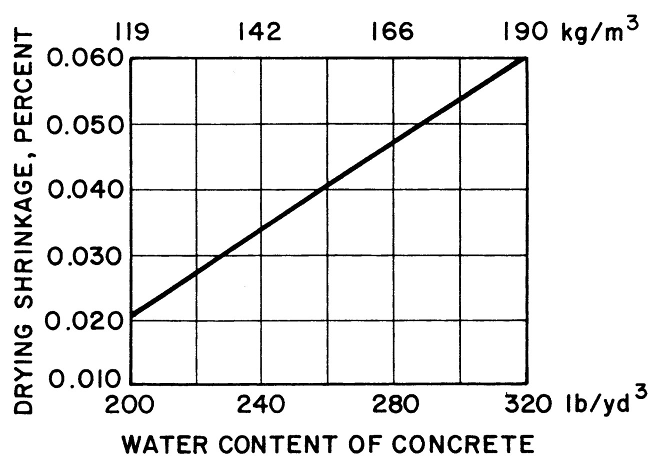

Fig 3.5—Typical effect of water content of concrete on drying shrinkage (USBR 1981).

(Hansen and Almudaiheem 1987). This is also illustrated in Fig. 3.4, where an elastic modulus ratio between 1 and 2 indicates an aggregate stiffness that is much smaller than that of normalweight aggregate.

Pickett (1956) and Hansen and Almudaiheem (1987) developed constitutive models for predicting the influence of relative aggregate content and modulus ratio on ultimate concrete shrinkage. The latter model clearly explains why lightweight concrete for the same relative aggregate content exhibits considerably more shrinkage than ordinary concrete. This is also illustrated in Fig. 3.4 when the modulus ratio is between one and two because the aggregate stiffness is much smaller than that of normalweight aggregate.

The influence of aggregate-absorption capacity on concrete shrinkage was investigated by Carlson (1938) and is illustrated

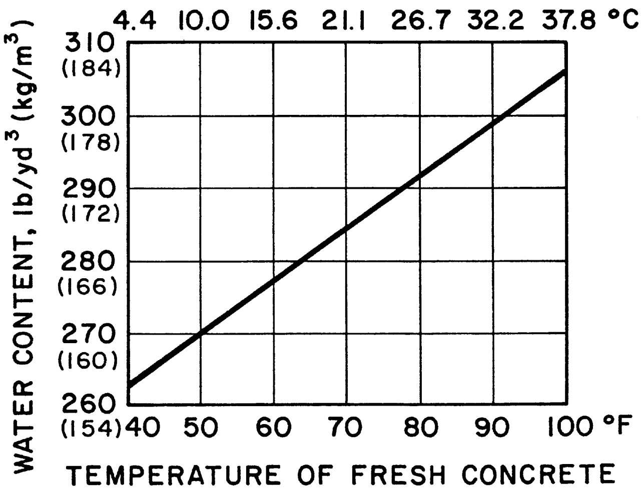

Fig. 3.6—Effect of temperature of fresh concrete on its water requirement (USBR 1981).

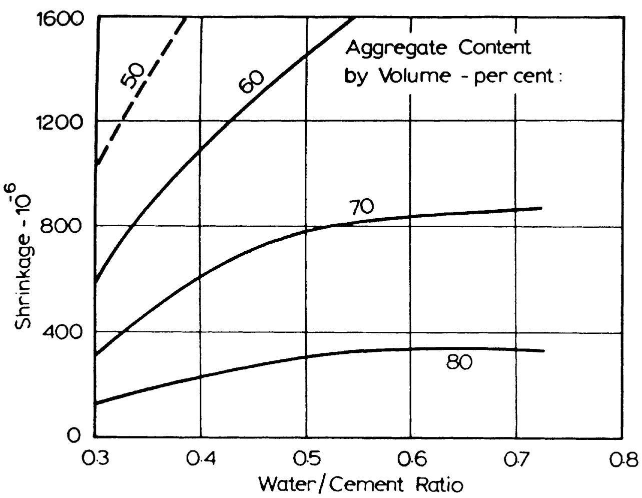

Fig. 3.7—Influence of w/c *and aggregate content on shrinkage (Odman 1968).*

in Table 3.1; the concrete had identical cements and *w/cm*s. The absorption of an aggregate, which is a measure of porosity, influences its modulus or compressibility. A low elastic modulus is usually associated with high absorption.

Quartz, limestone, dolomite, granite, feldspar, and some basalts can be classified as higher-modulus aggregates, which result in lower shrinkage properties of concrete. High-shrinkage concrete often contains sandstone, slate, hornblende, and some types of basalts. Because the rigidity of certain aggregates, such as granite, limestone, or dolomite, can vary over a wide range, their effectiveness in restraining drying shrinkage varies.

Although compressibility is the most important property of aggregate governing concrete shrinkage, the aggregate itself can shrink during drying. This is true for sandstone and other aggregates of high-absorption capacity. In general, aggregate with a high modulus of elasticity and low absorption will produce a concrete with low ultimate shrinkage.

3.4.3 *Paste content and* w/cm—Consistency, as measured by the slump test, is an important parameter in proportioning concrete. The amount of mixing water needed to achieve a given slump is dependent on the maximum aggregate size used because the maximum size influences the total aggregate surface area that needs to be covered with cement paste. Decreasing maximum aggregate size increases the total surface area to be covered with paste. Therefore, more water and cement are needed to achieve a given slump. For the same *w/cm*, concrete shrinkage increases with increasing water content because the paste volume increases; this agrees with the predictions in Fig. 3.4 and results obtained by the U.S. Bureau of Reclamation (1975) shown in Fig. 3.5. For a constant *w/cm*, there is an approximately linear relationship between water content (paste content as well) and concrete shrinkage within the range of water contents listed. Temperature also has an influence on the water requirements of the fresh concrete for same slump (Fig. 3.6). A reduction in water content, which reduces the paste content, will reduce the ultimate drying shrinkage of concrete. Therefore, the water content (and paste content) of a concrete mixture should be kept to a minimum to minimize potential drying shrinkage and the cracking tendency of the concrete.

Figure 3.7 illustrates that concrete shrinkage increases with *w/cm* for a given aggregate content. This effect is more pronounced with lower aggregate contents (Odman 1968).

3.4.4 *Influence of member size*—The size and shape of a concrete member and the porosity of the cement paste influences the drying rate of concrete and, therefore, influences the shrinkage rate. The shape affects the ratio of the surface area to volume of the member, and a higher ratio results in a higher drying rate. For a given concrete, the observed shrinkage at a given time decreases with an increase in the size of the specimen. This effect is illustrated in Fig. 3.8 (Bryant and Vadhanavikkit 1987) in which long-term shrinkage results were obtained on concrete prisms up to 400 mm (8 in.) thick. Ultimate shrinkage may not be reached for structural members during the intended service life.

Another consequence of moisture diffusion is that a moisture gradient develops from the surface to the interior. For a specimen that has moisture evaporation from all surfaces, shrinkage strain is greatest at the surface where moisture content is lowest, and shrinkage strain decreases toward the center where moisture content is highest. Nonuniform self-equilibrating internal stresses develop. Tensile stresses occur at and near the surfaces and compressive stresses develop at and near core, as shown in Fig. 3.9.

Warping occurs if drying takes place in an unsymmetrical manner, either due to drying from one side or due to a nonsymmetrical structure. In slabs-on-grade, the warping mechanism is a primary cause of cracking. Moisture evaporates from the top surface only, which causes higher shrinkage at the top. The concrete near the top surface is partially restrained from shrinking because it is attached to concrete lower in the slab that is more moist and does not shrink as much as the top surface. This restraint produces tensile stresses at and near the top surface, which results in the slab warping or curling, and the free edges of the slab can lift off

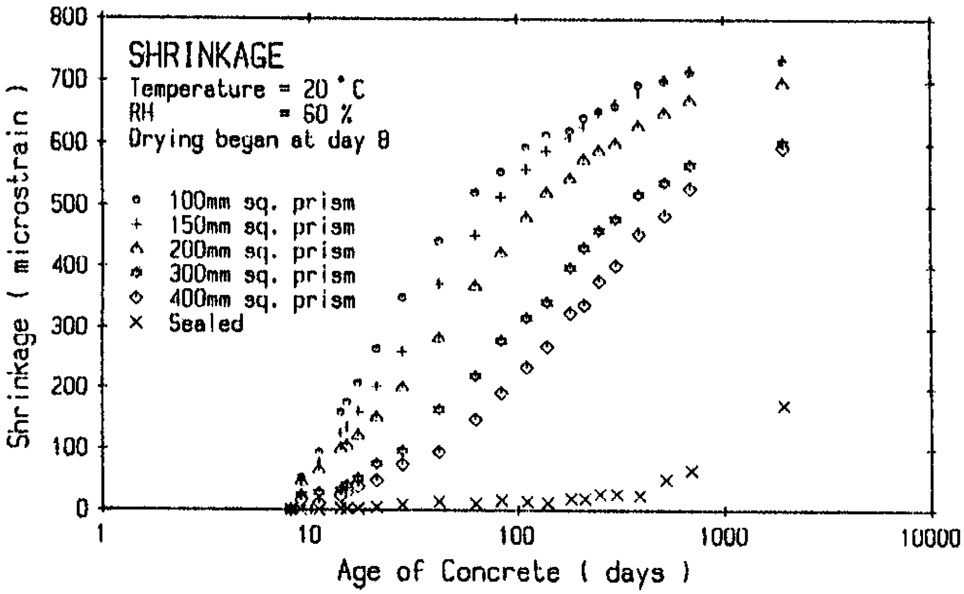

Fig. 3.8—Influence of specimen size on shrinkage (Bryant and Vadhanavikkit 1987).

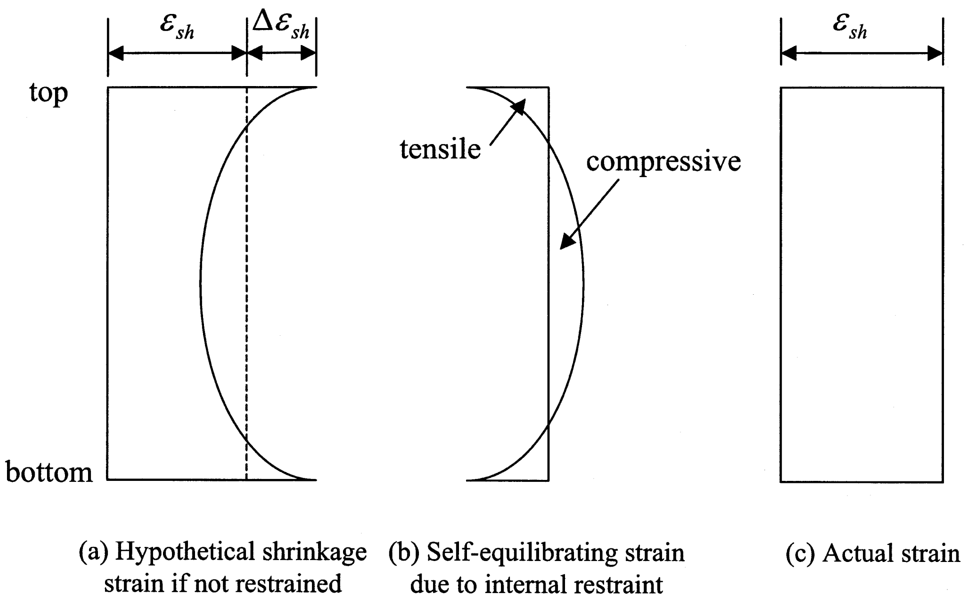

Fig 3.9—Internal restraint of shrinkage.

the ground. If the edges of the slab are restrained from movement, such as footings, and the slab is not allowed to warp, then the top surface has higher tensile stresses. Cracking can result if the tensile stresses from restrained shrinkage exceed the tensile strength of the concrete. Cracking may also result near the edge of the slab when a vertical load is applied on the warped cantilever.

3.4.5 *Effect of curing on shrinkage*—Carlson (1938) reported that the duration of moist curing of concrete does not have much effect on ultimate drying shrinkage. Test results from the California Department of Transportation (1963) show that substantially the same shrinkage occurred in concrete that was moist-cured for 7, 14, and 28 days before drying started. As far as the cracking tendency of the concrete is concerned, prolonged moist curing may not be beneficial. A general recommendation is to continue moist curing for at least 7 days. (For further information, refer to ACI 309.)

Sealed curing is curing without loss or addition of water. It eliminates other kinds of shrinkage so that all the resulting shrinkage will be autogenous. Autogenous shrinkage is a result of the fact that the products of hydration occupy a smaller volume than the original volume of cement and water. Self-dessication is a problem in low *w/c* concretes under sealed conditions in which the pores dry out and hydration slows down. Autogenous shrinkage strain is typically about 40 to 100×10^{-6} (Davis 1940). Houk, Paxton, and Houghton (1969) found that autogenous shrinkage increases with increasing temperature, cement content, and cement fineness.

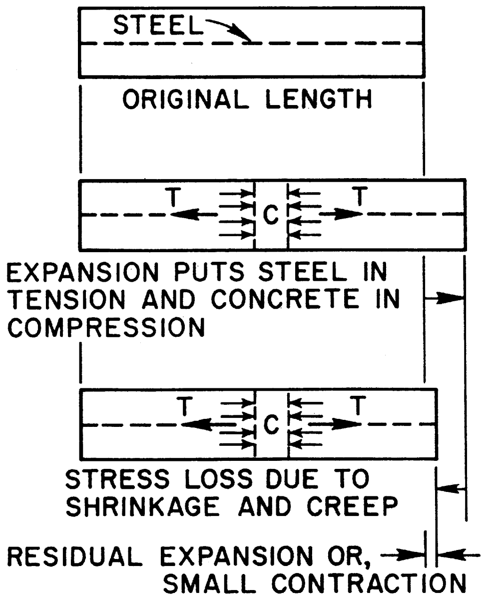

Fig. 3.10—Basic concept of shrinkage-compensating concrete.

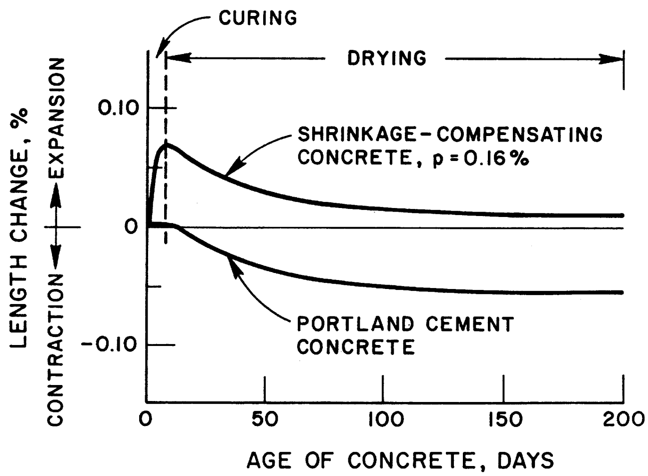

Fig. 3.11—Length-change characteristics for shrinkage-compensating and portland cement concrete (relative humidity = 50%).

3.4.6 *Effect of admixtures*—The effect of admixtures on concrete shrinkage is unclear. As an example, early-age shrinkage appears to increase by about 100% in the presence of calcium chloride, whereas later-age shrinkage is increased by about 40% compared with control specimens (ACI 212.3R).

Air-entrainment does not seem to increase shrinkage by more than 10% for air contents up to about 5% (Carlson 1938).

Results by Ghosh and Malhotra (1979), Brooks, Wainwright, and Neville (1979), and Feldman and Swenson (1975) indicated that the use of high-range water-reducing admixtures increases shrinkage. According to Ytterberg (1987), high-range water-reducing admixtures do not necessarily reduce shrinkage in proportion to their ability to reduce water content.

3.5—Control of shrinkage cracking

Concrete tends to shrink due to drying whenever its surfaces are exposed to air of low relative humidity or high winds. Because various kinds of restraint prevent the concrete from contracting freely, cracking should be expected, unless the ambient relative humidity is kept near 100%. The control of cracking consists of reducing the cracking tendency to a minimum, using adequate and properly positioned reinforcement, and using contraction joints. The CEB-FIP Model Code (1990) gives quantitative recommendations on the control of cracking due to shrinkage by listing various coefficients to determine the shrinkage levels that can be expected. Control of cracking by correct construction practices is covered in Chapter 8.

Cracking can also be minimized by using expansive cements to produce shrinkage-compensating concrete. This is discussed in Section 3.6.

3.5.1 *Reduction of cracking tendency*—Most measures that can be taken to reduce concrete shrinkage will also reduce the cracking tendency. Drying shrinkage can be reduced by using less water in the mixture and the largest practical maximum-size aggregate. A lower water content can be achieved by using a well-graded aggregate, stiffer consistency, and lower initial temperature of the concrete.

Concrete can withstand higher tensile strains if the stress is slowly applied; therefore, it is desirable to prevent rapid drying of concrete. Prevention of rapid drying can be attained by using curing compounds, even after water curing.

3.5.2 *Reinforcement*—Properly placed reinforcement, used in adequate amounts, will reduce the number and widths of cracks, reducing unsightly cracking. By distributing the shrinkage strains along the reinforcement through bond stresses, the cracks are distributed so that a larger number of narrow cracks occur instead of a few wide cracks. Although the use of reinforcement to control cracking in a relatively thin concrete section is practical, it is not needed in massive structures, such as dams, due to the low drying shrinkage of these mass concrete structures. The minimum amount and spacing of reinforcement to be used in structural floors, roof slabs, and walls for control of temperature and shrinkage cracking is given in ACI 318 or in ACI 350R. The minimum-reinforcement percentage, which is between 0.18 and 0.20%, does not normally control cracks to within generally acceptable design limits. To control cracks to a more acceptable level, the percentage requirement needs to exceed about 0.60%.

3.5.3 *Joints*—The use of joints is the an effective method of preventing the formation of unsightly cracking. If a sizeable length or expanse of concrete, such as walls, slabs, or pavements, is not provided with adequate joints to accommodate shrinkage, the concrete will make its own joints by cracking.

Contraction joints in walls are made, for example, by fastening wood or rubber strips to the form, which leave narrow vertical grooves in the concrete on both faces of the wall. Cracking of the wall due to shrinkage should occur at the grooves, relieving the stress in the wall and preventing the formation of unsightly cracks between the joints. These grooves should be sealed to prevent moisture penetration.

Sawed joints are commonly used in pavements and slabs-on-grade. Joint location depends on the particulars of placement. Each element should be studied individually to determine where the joints should be placed. ACI 224.3R discusses the use of joints in concrete construction. Guidance on joint sealants and contraction joint location in slabs is available in ACI 504R and ACI 302.1R.

3.6—Shrinkage-compensating concrete

Shrinkage-compensating concrete made with expansive cements can be used to minimize or eliminate shrinkage cracking. The properties and use of expansive cement concrete are summarized in ACI 223, ACI 223 (1970), ACI SP-38, and ACI SP-64. Of the several expansive cements produced in the past, Type K shrinkage-compensating cement (ASTM C 845) is currently the only one available in the United States. Several component materials are available to produce shrinkage-compensating concrete.

In reinforced shrinkage-compensating concrete, the expansion of the cement paste during the first few days of hydration will develop a low level of prestress, inducing tensile stresses in the steel and compressive stresses in the concrete. The level of compressive stresses developed in the shrinkage-compensating concrete ranges from 0.2 to 0.7 MPa (25 to 100 psi). Normal shrinkage occurs when water starts to evaporate from the concrete. The contraction of the concrete will result in a reduction or elimination of its precompression. The initial expansion of the concrete reduces the magnitude of any tensile stress that develops due to restrained shrinkage. This basic concept of using expansive cement to produce a shrinkage-compensating concrete is illustrated in Fig. 3.10. To allow for adequate expansion, special details may be needed at joints.

A typical length-change history of a shrinkage-compensating concrete is compared to that of a portland cement concrete in Fig. 3.11. The amount of reinforcing steel normally used in reinforced concrete made with portland cements is usually more than adequate to provide the elastic restraint needed for shrinkage-compensating concrete. To take full advantage of the expansive potential of shrinkage-compensating concrete in minimizing or preventing shrinkage cracking of exposed concrete surfaces, it is important that positive and uninterrupted water curing (wet covering or ponding) be started immediately after final finishing. For slabs on well-saturated subgrades, curing by sprayed-on membranes or moisture-proof covers has been successfully used. Inadequate curing of shrinkage-compensating concrete can result in an insufficient expansion to elongate the steel and subsequent cracking from drying shrinkage. Specific recommendations and information on the use of shrinkage-compensating concrete are contained in ACI 223R.

CHAPTER 4—CONTROL OF CRACKING IN FLEXURAL MEMBERS

4.1—Introduction

The control of cracking can be as important as the control of deflection in flexural members. Cracking in the tension zone of a reinforced beam starts at stress levels as low as 20 MPa (3000 psi) in the reinforcement. Crack control is also important to aesthetics of exposed concrete surfaces.

The role of cracks in the corrosion of reinforcing steel is controversial (ACI 222R). One viewpoint is that cracks reduce the service life of structures by permitting more rapid penetration of carbonation and allow chloride ions, moisture, and oxygen to reach the reinforcing steel. Another point of view is that while cracks accelerate the onset of corrosion, the corrosion is localized. With time, chlorides and water penetrate uncracked concrete and initiate more widespread corrosion. Consequently, after a few years of service, there is little difference between the amount of corrosion in cracked and uncracked concrete. More important parameters for corrosion protection are concrete cover and concrete quality.

This chapter is concerned primarily with cracks caused by flexural and tensile stresses, but temperature, shrinkage, shear, and torsion can also lead to cracking. Cracking in certain specialized structures, such as reinforced concrete tanks, bins, silos, and environmental structures is not covered in this report. Cracking of concrete in these structures is described by Yerlici (1975), and in ACI 313 and ACI 350R.

Extensive research studies on the cracking behavior of beams have been conducted over the last 50 years. Most of the work conducted before 1970 was reviewed by ACI Committee 224 (1971) in ACI Bibliography No. 9. Additional work is referenced in this chapter. Leonhardt (1977 and 1988) presents an extensive review of cracking in reinforced- and prestressed-concrete structures. The CEB-FIP Model Code for Concrete Structures (1990) gives the European approach to crack width evaluation and permissible crack widths.

The basis for codes of practice, both in the U.S. and Europe, to limit service-load cracking is rooted in equations to predict crack widths. Several of the most important crack-prediction equations are reviewed in this report. The trend in reinforced- and prestressed concrete design to ensure acceptable cracking at service loads is to provide proper detailing, such as provision of minimum reinforcement and proper selection of bar diameters, bar spacing, and reduction of restraint rather than trying to make use of a sophisticated crack calculation (Schlaich, Schafer, and Jennewien 1987; Halvorsen 1987).

Fiber-reinforced polymer (FRP) bars have been used as a reinforcing material (Nawy and Neuwerth 1977, Dolan 1990). Experience is limited, however, and crack control in structures reinforced with these materials is not addressed in this report.

4.2—Crack-control equations for reinforced concrete beams

A number of equations have been proposed for predicting crack widths in flexural members; most of them were reviewed in the original version of this committee report (ACI Committee 224 1972) and in key publications listed in the references. Crack control is provided by calculating the probable crack width and proportioning structural elements so that the computed width is less than some predefined value. Most equations predict the probable maximum crack width, which usually means that about 90% of the crack widths in the member are below the calculated value. Research, however, has shown that isolated cracks in beams in excess of twice the computed maximum can occur (Holmberg and Lindgren 1970) although generally, the coefficient of variation of crack width is about 40% (Leonhardt 1977). There is evidence that this range in crack width variability can increase with the size of the member (ACI Committee 224 1972).

Crack-control equations are presented in the sections that follow.

4.2.1 *ACI approach through ACI 318-95*—Requirements for flexural crack control in beams and thick one-way slabs (span-depth ratio in the range of 15 to 20) are based on the statistical analysis (Gergely and Lutz 1968) of maximum crack-width data from a number of sources. Based on the analysis, the following general conclusions were reached:

- The reinforcing steel stress is the most important variable;
- The thickness of the concrete cover is an important variable but not the only geometric consideration;
- The area of concrete surrounding each reinforcing bar is also an important geometric variable;
- The bar diameter is not a major variable; and
- The ratio of crack width at the surface to that at the reinforcement level is proportional to the ratio of the nominal strain at the surface and the reinforcement strain.

The equations that were considered to best predict the probable maximum bottom and side crack widths are

$$w_b = 0.091\sqrt[3]{t_b A}\beta(f_s - 5) \times 10^{-3} \quad (4\text{-}1a)$$

$$w_s = \frac{0.091\sqrt[3]{t_b A}}{1 + t_s/h_1}(f_s - 5) \times 10^{-3} \quad (4\text{-}1b)$$

where

w_b = most probable maximum crack width at bottom of beam, in.;

w_s = most probable maximum crack width at level of reinforcement, in.;

f_s = reinforcing steel stress, ksi;

A = area of concrete symmetric with reinforcing steel divided by number of bars, in.^{2};

t_b = bottom cover to center of bar, in.;

t_s = side cover to center of bar, in.;

β = ratio of distance between neutral axis and tension face to distance between neutral axis and reinforcing steel about 1.20 in beams; and

h_1 = distance from neutral axis to the reinforcing steel, in.

Simplification of Eq. (4-1a) yielded the following equation

$$w = 0.076\beta f_s\sqrt[3]{d_c A} \times 10^{-3} \quad (4\text{-}2a)$$

where

w = most probable maximum crack width, in.; and

d_c = thickness of cover from the extreme tension fiber to the closest bar, in.

When the strain ε_s in the steel reinforcement is used instead of stress f_s, Eq. (4-2) becomes

$$w = 2.2\beta\varepsilon_s\sqrt[3]{d_c A} \quad (4\text{-}2b)$$

Eq. (4-3) is valid in any system of units.

The cracking behavior in thick one-way slabs (span-depth ratio 15 to 20) is similar to that in shallow beams. For one-way slabs with a clear concrete cover in excess of 25.4 mm (1 in.), Eq. (4-2) can be properly applied if β = 1.25 to 1.35 is used.

ACI 318-95 Section 10.6 uses Eq. (4-2) with β = 1.2 in the following form

$$z = f_s\sqrt[3]{d_c A} \quad (4\text{-}3)$$

and permits the calculation of z with f_s equal to 60% of the specified yield strength f_y in lieu of exact calculation.

In ACI 318-95 and earlier code versions, the maximum allowable z = 175 kips per in. for interior exposure corresponds to a probable crack width of 0.41 mm (0.016 in.). This level of crack width may be excessive for aesthetic concerns.

ACI 318 has allowed a value of z = 145 kips per in. for exterior exposure based on a crack width value of 0.33 mm (0.013 in.). While application of Eq. (4-2a) ((Eq. 10-4) of ACI 318-95) to beams gives adequate crack-control values, its application to one-way slabs with standard 20 mm (3/4 in.) cover and reinforced with steel of 60 ksi (400 MPa) or lower yield strength results in large reinforcement spacings. The provisions of Section 7.6.5 of ACI 318-95, however, directly limit the spacing of such reinforcement in one-way slabs.

ACI 340R contains design aids for the application of Eq. (4-3).

4.2.2 *ACI 318-99 approach*—ACI Committee 318 now believes that it can be misleading to purport to effectively calculate crack widths, given the inherent variability in cracking. The three important parameters in flexural cracking are steel stress, cover, and bar spacing. Steel stress is the most important parameter.

A reevaluation of cracking data (Frosch 1999) provided a new equation based on the physical phenomenon for the determination of the flexural crack widths of reinforced concrete members. This study showed that previous crack width equations are valid for a relatively narrow range of covers (up to 63 mm [2.5 in.]).

ACI 318-99, Section 10.6, does not make a distinction between interior and exterior exposure. It requires that for crack control in beams and one-way slabs, the spacing of reinforcement closest to a surface in tension shall not exceed that given by

$$s(\text{in.}) = [(540/f_s) - 2.5c_c] \quad (4\text{-}4a)$$

Table 4.1—Guide to reasonable* crack widths, reinforced concrete under service loads

Exposure condition	Crack width, in.	Crack width, mm
Dry air or protective membrane	0.016	0.41
Humidity, moist air, soil	0.012	0.30
Deicing chemicals	0.007	0.18
Seawater and seawater spray, wetting and drying	0.006	0.15
Water-retaining structures†	0.004	0.10

* It should be expected that a portion of the cracks in the structure will exceed these values. With time, a significant portion can exceed these values. These are general guidelines for design to be used in conjunction with sound engineering judgement.
†Exclusing nonpressure pipes.

but not greater than $12(36/f_s)$ or 12 in., where

f_s = calculated stress in reinforcement at service load (ksi) = unfactored moment divided by the product of steel area and internal moment arm. Alternatively, f_s can be taken as 0.60;
c_c = clear cover from the nearest surface in tension to the flexural tension reinforcement, in.; and
s = center-to-center spacing of flexural tension reinforcement nearest to the surface of the extreme tension face, in.

The SI expression for the reinforcement spacing in Eq. (4-4a) (f_s in MPa) is

$$s(\text{mm}) = [(95{,}000/540f_s) - 2.5c_c] \quad (4\text{-}4b)$$

but not to exceed $300(252/f_s)$ mm.

4.2.3 *CEB-FIP and Eurocode EC2 recommendations*—Other organizations around the world have developed procedures for predicting crack widths in structural concrete ranging from conventionally reinforced through partially and fully prestressed. ACI 318 procedures only deal with conventionally reinforced concrete. Crack-control recommendations proposed in the European Model Code for Concrete Structures (CEB-FIP 1990; Euro EC2 1997) apply to prestressed as well as reinforced concrete with modifications and can be summarized in the following sections.

4.2.3.1 *CEB-FIP 1990 provisions*—The characteristic crack width w_k in beams is expressed as follows in terms of the length $l_{s,max}$ over which slip occurs between the steel reinforcement and the concrete (approximating crack spacing in stabilized cracking)

$$w_k = l_{s,max}(\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs}) \quad (4\text{-}5)$$

where

ε_{sm} = average reinforcement strain within segment length, $l_{s,max}$;
ε_{cm} = average concrete strain within segment length, $l_{s,max}$; and
ε_{cs} = strain of concrete due to shrinkage.

The characteristic crack width w_k cannot exceed the limiting crack with w_{lim}, namely

$$w_k \le w_{lim} \quad (4\text{-}6)$$

where w_{lim} = nominal limit value of the crack width specified for cases with expected functional consequences of cracking (such as conditions stipulated in Table 4.1). In the absence of specific requirements, such as water tightness or specific exposure classes as tabulated in the CEB Code, a limiting value of w_{lim} equal to 0.30 mm (0.012 in.) is satisfactory with respect to appearance and ductility.

The length $l_{s,max}$ in Eq. (4-5) can be defined as

$$l_{s,max} = 2 \cdot \frac{(\sigma_{s2} - \sigma_{s1})}{(4\tau_{bk})} \cdot \phi_s \quad (4\text{-}7a)$$

where

σ_{s2} = reinforcement stress at the crack location, MPa;
σ_{s1} = reinforcement stress at point of zero slip, MPa;
ϕ_s = reinforcing bar diameter or equivalent diameter of bundled bars, mm;
τ_{bk} = lower fractile value of the average bond stress, MPa = $1.8 f_{ctm(t)}$; and
$f_{ctm(t)}$ = the mean value of the concrete tensile strength at the time that the crack forms.

For stabilized cracking, the expression can be simplified as follows

$$l_{s,max} = \frac{\phi_s}{3.6\rho_{s,ef}} \quad (4\text{-}7b)$$

For single-crack formation, Eq. (4-6) is expressed as

$$l_{s,max} = \sigma_{s2}\frac{\phi_s}{2\tau_{bk}(1 + n\rho_{s,ef})} \quad (4\text{-}8)$$

The term can be assumed equal to 1.0 for simple calculation, n being the modular ratio E_s/E_c, where

$\rho_{s,ef}$ = effective reinforcement ratio, $A_s/A_{c,ef}$;
A_s = area of tension reinforcement, mm^2; and
$A_{c,ef}$ = effective concrete area in tension, mm^2.

The effective area of concrete in tension can be calculated as

$$A_{c,ef} = b[2.5(h - d)] \quad (4\text{-}9)$$

where

b = beam width at the tension side;
h = total section depth; and
d = effective depth to the centroid of the tensile reinforcement.

For stabilized cracking, the average width of the crack can be estimated on the basis of the average crack spacing such that

$$S_{rm} = \frac{2}{3}l_{s,max} \quad (4\text{-}10)$$

Table 4.2—Maximum bar diameter for high bond bars

Steel stress, MPa	Maximum bar size, mm
160	32
200	25
240	20
280	16
320	12
360	10
400	8
450	6

Table 4.3—Maximum bar spacing for high bond bars

	Maximum bar spacing, mm	
Steel stress, MPa	Pure flexure	Pure tension
160	300	200
200	250	150
240	200	125
280	150	75
320	100	—
360	50	—

where S_{rm} is the mean crack spacing value (mm) in the beam.

4.2.3.2 *Eurocode EC2 provisions*—The Eurocode EC2 requires that cracking should be limited to a level that does not impair the proper functioning of the structure or cause its appearance to be unacceptable (Euro EC2 1997; Beckett and Alexandrou 1997; Nawy 2001). It limits the maximum design crack width to 0.30 mm (0.012 in.) for sustained load under normal environmental conditions. This ceiling is expected to be satisfactory with respect to appearance and durability. Stricter requirements are stipulated for more severe environmental conditions.

The code stipulates that the design crack width be evaluated from the following expression

$$w_k = \beta s_{rm}\varepsilon_{sm} \tag{4-11}$$

where

w_k = design crack width;
s_{rm} = average stabilized crack spacing;
ε_{sm} = mean strain under relevant combination of loads and allowing for the effect such as tension stiffening or shrinkage; and
β = coefficient relating the average crack width to the design value
= 1.7 for load-induced cracking and for restraint cracking in sections with minimum dimension in excess of 800 mm (32 in.).

The strain ε_{sm} in the section is obtained from the following expression:

$$\varepsilon_{sm} = \sigma_s / E_s[1 - \beta_1\beta_2(\sigma_{sr}/\sigma_s)^2] \tag{4-12}$$

where

σ_s = stress in the tension reinforcement computed on the basis of a cracked section, MPa;
σ_{sr} = stress in the tension reinforcement computed on the basis of a cracked section under loading conditions that cause the first crack, MPa;
β_1 = coefficient accounting for bar bond characteristics
= 1.0 for deformed bars and 0.5 for plain bars;
β_2 = coefficient accounting for load duration
= 1.0 for single short-term loading and 0.5 for sustained or cyclic loading; and
E_s = Modulus of elasticity of the reinforcement, MPa.

The average stabilized mean crack spacing s_{rm} is evaluated from the following expression

$$s_{rm} = 50 + 0.25k_1k_2d_b/\rho_t, \text{ mm} \tag{4-13}$$

where

d_b = bar diameter, mm;
ρ_t = effective reinforcement ratio = A_s/A_{ct}; the effective concrete area in tension A_{ct} is generally the concrete area surrounding the tension reinforcement of depth equal to 2.5 times the distance from the tensile face of the concrete section to the centroid of the reinforcement. For slabs where the depth of the tension zone may be small, the height of the effective area should not be taken greater than $[(c - d_b)/3]$, where c = clear cover to the reinforcement, mm;
k_1 = 0.8 for deformed bars and 1.6 for plain bars; and
k_2 = 0.5 for bending and 1.0 for pure tension.

In cases of eccentric tension or for local areas, an average value of $k_2 = (\varepsilon_1 + \varepsilon_2) / 2\varepsilon_1$ can be used, where ε_1 is the greater and ε_2 the lesser tensile strain at the section boundaries, determined on the basis of cracked section.

In the absence of rigorous computations as described thus far, choice of minimum area of reinforcement A_s for crack control is stipulated such that

$$A_s = k_c k f_{ct,eff} A_{ct}/\sigma_s \tag{4-14}$$

where

A_s = reinforcement area within the tensile zone, mm;
A_{ct} = effective area of concrete in tension, mm;
σ_s = maximum stress permitted in the reinforcement after the formation of the crack. The yield strength may be taken in lieu of σ_s, although lower values may be needed to satisfy crack width limits;
$f_{ct,eff}$ = tensile strength of the concrete effective at the formation of the first crack. A value of 3 MPa (435 psi) can be used;
k_c = coefficient representing the nature of stress distribution,
= 1.0 for direct tension and 0.4 for bending; and
k = coefficient accounting for nonuniform stresses due to restraint resulting from intrinsic or extrinsic deformation. It varies between 0.5 and 1.0 (N/mm^2 = 1 MPa).

The EC2 Code also stipulates that for cracks dominantly caused principally by flexure, their widths will not usually exceed the standard 0.30 mm (0.012 in.) if the size and spacing of the reinforcing bars are within the range of values in Tables 4.2 and 4.3 for bar size and spacing (Euro EC2 1997; Beckett and Alexandrou 1997; Nawy 2001). For severe exposure conditions, such as those listed in Table 4.1, crack width computations become mandatory.

4.3—Crack control in two-way slabs and plates

Crack-control equations for beams underestimate the crack widths developed in two-way slabs and plates (Nawy and Blair 1971) and do not indicate to the designer how to space the reinforcement. The cracking widths in two-way slabs and plates are controlled primarily by the steel stress level and the spacing of the reinforcement in the two perpendicular directions. In addition, the clear concrete cover in two-way slabs and plates is nearly constant (20 mm [3/4 in.] for most interior structural slabs), whereas it is a major variable in the crack-control equations for beams.

Analysis of data on cracking in two-way slabs and plates (Nawy and Blair 1971) has provided the following equation for predicting the maximum crack width

$$w = k\beta f_s\sqrt{I} \tag{4-15}$$

where the terms inside the radical are collectively termed the grid index:

$$I = \frac{d_{b1}s_2}{\rho_{t1}} = \left[\frac{s_1 s_2 d_c}{d_{b1}}\frac{8}{\pi}\right]$$

k = fracture coefficient with a value $k = 2.8 \times 10^{-5}$ for uniformly loaded restrained two-way action square slabs and plates. For concentrated loads or reactions or when the ratio of short to long span is less than 0.75 but larger than 0.5, a value of $k = 2.1 \times 10^{-5}$ is applicable. For span aspect ratios less than 0.5, $k = 1.6 \times 10^{-5}$;

β = 1.25 (chosen to simplify calculations, although it varies between 1.20 and 1.35);

f_s = actual average service-load stress level or 40% of the specified yield strength f_y, ksi;

d_{b1} = diameter of the reinforcement in Direction 1 closest to the concrete outer fibers, in.;

s_1 = spacing of the reinforcement in Direction 1, in.;

s_2 = spacing of the reinforcement in perpendicular Direction 2, in.;

ρ_{t1} = active steel ratio, that is, the area of steel A_s per ft width/$[12d_{b1} + 2c_1]$, where c_1 is clear concrete cover measured from the tensile face of concrete to the nearest edge of the reinforcing bar in Direction 1; and

w = crack width at face of concrete caused by flexure, in.

Direction 1 refers to the direction of reinforcement closest to the outer concrete fibers; this is the direction for which crack-control check should be made. Subscripts 1 and 2 pertain to the directions of reinforcement.

For simply supported slabs, the value of k should be multiplied by 1.5. Interpolated k values apply for partial restraint at the boundaries. For zones of flat plates where transverse steel is not used or when its spacing s_2 exceeds 305 mm (12 in.), use s_2 = 305 mm (12 in.) in the equation.

If strain is used instead of stress, Eq. (4-15) becomes

$$w = k_1\beta\varepsilon\sqrt{I} \tag{4-16}$$

where values of $k_1 = 29 \times 10^3$ times the k values previously listed. Nawy (1972) and ACI 340.1R contain design aids for applying these recommendations.

Tam and Scanlon (1986) present a model for determining deflection of two-way slabs subjected to transverse loads. Their model accounts for the net effect on deflection of both restraint cracking and flexural cracking.

4.4—Tolerable crack widths versus exposure conditions in reinforced concrete

Table 4.1 presents a general guide for what could be considered reasonable crack widths at the tensile face of reinforced concrete structures for typical conditions. These reasonable crack width values are intended to serve only as a guide for proportioning reinforcement during design. They are to be used as a general guideline along with sound engineering judgment.

The table is based primarily on Nawy (1968), who compiled information from several sources. It is important to note that these crack width values are not always a reliable indication of the corrosion and deterioration to be expected. In particular, a larger cover, even if it leads to a larger surface crack width, may be preferable for corrosion control in certain environments; therefore, the designer should exercise engineering judgment on the extent of crack control to be used. When used in conjunction with the recommendations presented in Sections 4.2.1 and 4.2.3 to limit crack width, it should be expected that a portion of the cracks in the structure would exceed these values by a significant amount. It is also noted that time effects, such as creep, will cause an increase in crack widths that should be taken into account by the designer.

Another opinion regarding crack control suggests that in the long term there is no link between the level of flexural cracking and corrosion (Beeby 1983). This suggests that independent of exposure conditions, the acceptable level of cracking is primarily an aesthetic issue. Therefore, in cases such as liquid-containing structures where the presence of moisture is constant or leakage is of concern should more restrictive (smaller) crack widths be required. Based on information in Halvorsen (1987), a case could be made that crack widths ranging from 0.15 to 0.3 mm (0.006 to 0.012 in.) could be considered unacceptable for aesthetic reasons as they are visible to the naked eye, hence generating a sense of insecurity or structural failure.

4.5—Flexural cracking in prestressed concrete

Partially prestressed members, in which cracks can appear under working loads, are used extensively. Cracks form in these members when the tensile stress exceeds the modulus of rupture of the concrete (6 to 9 $\sqrt{f_c'}$ psi under short-term conditions). The control of these cracks is necessary primarily for aesthetic reasons, as they are visible to the naked eye, hence generating a sense of structural insecurity. The residual crack width, after removal of the major portion of the live load, is small (about 0.03 to 0.09 mm [0.001 in. to 0.003 in.]) and therefore, crack control is usually not necessary if the live load is transient.

There have been studies concerning the calculation of crack widths in prestressed concrete members (Meier and Gergely 1981; Suzuki and Yoshiteru 1984; Suri and Dilger 1986; Nawy 1989a). The complexity of the crack width calculations is increased over reinforced concrete members by the number of variables that should be considered.

4.5.1 *Crack-prediction equations*—One approach to crack prediction for bonded prestressed beams has two steps. First, the decompression moment is calculated, at which the stress in the concrete at the prestressing steel level is zero. Then the member is treated as a reinforced concrete member and the increase in stress in the steel is calculated for the additional loading. The expressions given for crack prediction in nonprestressed beams can be used to estimate the cracks for the load increase above the decompression moment. A multiplication factor of about 1.5 is needed when strands, rather than deformed bars, are used nearest to the beam surface in the prestressed member to account for the differences in bond properties. This approach is complicated if most of the parameters affecting cracking are considered (Nilson 1987). An approximate method using the nominal-concrete-stress approach was presented by Meier and Gergely (1982). They proposed the following equations for prediction of maximum flexural crack width

$$w_{max} = C_1 \frac{f_{ct}}{E_c} d_c \qquad (4\text{-}17)$$

$$w_{max} = C_2 \frac{f_{ct}}{E_c} d_c \sqrt[3]{A} \qquad (4\text{-}18)$$

where

C_1,C_2 = bond coefficients that depend on the type of steel nearest the tension face;
f_{ct} = nominal tensile stress at the tensile face;
E_c = modulus of elasticity of concrete;
d_c = minimum concrete cover to centroid of steel at the tensile face; and
A = effective concrete area per bar as defined in ACI 318.

Equation (4-17) is dimensionally correct and the coefficient C_1 is dimensionless. In in.-lb units, C_1 = 12 and C_2 = 8.4 for reinforcing bars, and C_1 = 16 and C_2 = 12 for strands. In SI units, if A is specified in mm^2, C_1 = 1.39 and C_2 = 0.97 for reinforcing bars, and C_1 = 1.85 and C_2 = 1.39 for strands.

Equation (4-17) had better application for most data examined; however, Eq. (4-18) shows better accuracy for wide beams with large spacing. These equations predict the average of the maximum crack widths. The scatter is considerable.

The maximum crack width (in in.) at the steel-reinforcement level closest to the tensile face of the concrete, accounting for the stress in the reinforcement in pretensioned and post-tensioned, fully and partially prestressed members can be evaluated from the following simplified expressions (Nawy and Huang 1977; Nawy 1989a):

Pretensioned beams

$$w_{max} = 5.85 \times 10^{-5} \frac{A_t}{\Sigma_O} (\Delta f_s) \qquad (4\text{-}19)$$

Post-tensioned unbonded beams

$$w_{max} = 6.51 \times 10^{-5} \frac{A_t}{\Sigma_O} (\Delta f_s) \qquad (4\text{-}20)$$

The maximum crack width at the tensile face of the concrete can be obtained by multiplying the values obtained from **Eq. (4-19) and (4-20) by a factor** R_i where

R_i = ratio h_2 / h_1;
h_1 = distance from the neutral axis to the centroid of the reinforcement, in.;
h_2 = distance from the neutral axis to the concrete tensile face;
Δf_s = the net stress in the prestressed tendon or the magnitude of the tensile stress in the conventional reinforcement at any load level in which the decompression load (decompression here means $f_c = 0$ at the level of the reinforcing steel) is taken as the reference point, ksi = $(f_{nt} - f_d)$
f_{nt} = stress in the prestressing steel at any load beyond the decompression load, ksi;
f_d = stress in the prestressing steel corresponding to the decompression load, ksi;
Σ_O = sum of reinforcing elements' circumferences, in.; and
A_t = the effective concrete area in uniform tension, in.^{2}, as defined by ACI 318.

Recent work by Nawy on cracking in high strength prestressed beams of compressive strength f_c' in excess of 85 MPa (12,000 psi), showed that the factor in Eq. (4.19), (4.20) becomes 2.75×10^{-5} in U.S. customary units and 4.0×10^{-5} in SI units (Nawy, 2000).

The CEB Model Code has the same equation for predicting the crack width in prestressed members as in nonprestressed members (Section 4.2.2). The increase in steel strain is calculated from the decompression stage. Other equations have been proposed (Abeles 1956; Bennett and Dave 1969; Holmberg and Lindgren 1970; Rao, Gandotra, and Ramazwamy 1976; Bate 1958; Bennett and Chandrasekhar 1971; Hutton and Loov 1966; Krishna, Basavarajuiah, and Ahamed 1973; Stevens 1969; Suri and Dilger 1986; Suzuki and Yoshiteru 1984; Harajli and Naaman 1989).

Aalami and Barth (1989) discuss the mitigation of restraint cracking in buildings constructed with unbonded tendons. Nonprestressed deformed bars can be used to reduce the width of the cracks to acceptable levels.

4.5.2 *Crack widths*—Some authors state that corrosion is a greater problem in prestressed-concrete members because of the smaller area of steel used and because of the possible consequences of corrosion on highly stressed steel. Research (Beeby 1978a, 1978b) indicates that there is no general relationship between cracking and corrosion in most circumstances. Poston, Carrasquillo, and Breen (1987), however, cites contradictory laboratory test results on prestressed and nonprestressed exposure specimens in which chloride-ion concentration at the level of reinforcement due to penetration of chlorides from external sources was proportional to crack width. Poston and Schupack (1990), present results from a field investigation of pretensioned beams in an aggressive chloride environment in which brittle wire failure of a seven-wire strand occurred at a flexural crack, apparently due to corrosion with significant pitting observed on the other wires at the crack location. The surface crack widths were 0.13 mm (0.005 in.) or less. The prestressing strand was generally bright on either side of its crack with no significant sign of corrosion distress.

As discussed by Halvorsen (1987), provisions for surface crack-width control as a means of protecting against corrosion should be strongly tied to provisions for high-quality concrete and plenty of cover. The importance of having high-quality (low *w/cm*) concrete with sufficient cover to provide long-term protection of steel elements, both prestressed and nonprestressed, cannot be overemphasized. The design should provide more stringent crack control than reinforcement spacing stipulated in ACI 318, for prestressed-concrete members, and particularly those subjected to aggressive environments, by providing additional mild steel reinforcement, reducing the allowable extreme fiber tension stresses under service loads to a value below $6\sqrt{f_c'}$ psi, perhaps as low as $2\sqrt{f_c'}$ psi, or both, and to minimize the potential for flexural cracking.

4.6—Anchorage-zone cracking in prestressed concrete

Longitudinal cracks frequently occur in the anchorage zones of prestressed concrete members due to transverse tensile stresses set up by the concentrated forces (Gergely 1969; Zielinski and Rowe 1960; Stone and Breen 1984a). Such cracks can lead to (or in certain cases are equivalent to) the failure of the member. Transverse reinforcement (stirrups), active reinforcement in the form of lateral prestressing, or both, should be designed to restrict these cracks.

Two types of cracks can develop: spalling cracks that begin at top and bottom beam ends outside the end anchorage zones and propagate parallel to the prestressing force, and bursting cracks that develop along the line of the force or forces but away from the end face.

For many years, stirrups were designed to take the entire calculated tensile force based on the analysis of the uncracked section. Classical and finite-element analyses (Stone and Breen 1984a; Nawy 1989b) show similar stress distributions for which the stirrups are to be provided. Because experimental evidence shows that higher stresses can result than those indicated by these analyses (Zielinski and Rowe 1960), and because the consequences of under-reinforcement can be serious, it is advisable to provide more steel than required by this type of analysis. More recently, designs have been based on cracked section analyses. A design procedure for post-tensioned members using a cracked section analysis (Gergely and Sozen 1967) has found acceptance with many designers. For pretensioned members, an empirical equation has proven to be quite useful (Marshall and Mattock 1962).

Stone and Breen (1984b) present a design procedure for post-tensioned beam anchorage zones. A general equation is given for predicting the cracking load in beams without supplemental anchorage zone reinforcement along with provisions for designing supplementary reinforcement and calculating the effect it will have on cracking and ultimate load.

Design recommendations for controlling cracking in anchorage zones of flexural members with closely spaced anchors, such as in slabs and bridge decks, are provided by Burgess, Breen, and Poston (1989) and Sanders, Breen, and Duncan (1987).

Spalling cracks form between anchorages and propagate parallel to the prestressing forces and can cause gradual failure, especially when the force acts near and parallel to a free edge. Because analyses show that the spalling stresses in an uncracked member occur primarily near the end face, it is important to place the first stirrup near the end surface and to distribute the stirrups over a distance equal to at least the depth of the member to fully account for both spalling and bursting stresses. In lieu of normal orthogonal reinforcement to control cracking, Stone and Breen (1984a, 1984b) showed the very beneficial effect of using spiral reinforcement or active reinforcement in the form of transverse prestressing to control cracking in anchorage zones where the prestressing forces are large.

4.7—Crack control in deep beams

Major changes in reinforced concrete design in the past two decades, namely the widespread adoption of strength design, have resulted in some structures with high service-load-reinforcement stresses. Several cases have been reported (Frantz and Breen 1980a, 1980b) where wide cracks have developed on the side faces of beams between main flexural reinforcement and the neutral axis. Although the measured crack widths at the main reinforcement level were within acceptable code limits, the side-face crack widths near middepth were as much as three times as wide.

Based on an experimental and analytical investigation of cracking in deep beams (in the sense of separation of tension and compression force resultants, not span-depth ratio), Frantz and Breen developed recommendations for side-face crack control in beams in which the depth *d* exceeds 915 mm (36 in.). Modifications of these recommendations have been included in ACI 318 since 1989. Section 10.6.7 of ACI 318 requires skin reinforcement to be uniformly distributed along both faces of the member for a distance *d*/2 nearest the flexural tension reinforcement.

4.8—Tension cracking

The cracking behavior of reinforced concrete members in axial tension is similar to that of flexural members, except that the maximum crack width is larger than that predicted by the expressions for flexural members (Broms 1965a,b). The lack of strain gradient and resultant restraint imposed by the compression zone of flexural members is probably the reason for the larger tensile crack width.

Data are limited, but it appears that the maximum tensile crack width can be about expressed in a form similar to that used for flexural crack width

$$w = 0.10 f_s \sqrt[3]{d_c A} \times 10^{-3} \qquad (4\text{-}21)$$

where crack width is measured in in.

A more complicated procedure for predicting crack width in tension members has been developed that incorporates both slip and bond stress (Yang and Chen 1988). Although the crack width prediction equation appears to show good agreement with available test data, the procedure is too complicated for design purposes. A similar approach was also developed for predicting crack widths in concrete tension members reinforced with welded-wire fabric (Lee et al. 1987). A more complete discussion of concrete cracking in direct tension is provided in ACI 224.2R.

CHAPTER 5—LONG-TERM EFFECTS ON CRACKING

5.1—Introduction

Cracking in concrete is affected by the long-term conditions to which the concrete element is subjected. In most cases, long-term exposure and long-term loading extend the magnitude of cracks, principally their width, in both reinforced and plain concrete. The discussion in this chapter summarizes the major long-term factors that affect the crack-control performance of reinforced and prestressed concrete.

5.2—Effects of long-term loading

As discussed in Chapter 2, both sustained and cyclic loading increase the amount of microcracking. Microcracking appears to be a function of the total strain and is largely independent of the method by which the strain is induced. Microcracks formed at service load levels do not seem to have a great effect on the strength or serviceability of reinforced and prestressed concrete.

The effect of sustained or repetitive loading on macroscopic cracking, however, can be an important consideration in the serviceability of reinforced concrete members, especially in terms of corrosion of reinforcing steel and appearance. The increase in crack width due to long-term or repetitive loading can vary between 100 and 200% over several years (Bate 1963; Brendel and Ruhle 1964; Lutz, Sharma and Gergely 1968; Abeles, Brown, and Morrow 1968; Bennett and Dave 1969; Holmberg and Lindgren 1970; Illston and Stevens 1972; Holmberg 1973). While there is a large scatter in the data, information obtained from sustained loading tests of up to 2 years (Illston and Stevens 1972) and fatigue tests with up to 1 million cycles (Bennett and Dave 1969; Holmberg 1973; Rehm and Eligehausen 1977) indicate that a doubling of crack width with time can be expected. Under most conditions, the spacing of cracks does not change with time at constant levels of stress (Abeles, Brown, and Morrow 1968; Illston and Stevens 1972; Holmberg 1973). An exception to this occurs at low loads or in beams with high percentages of reinforcement, in which case the total number and width of cracks increase substantially after the loading has begun (Brendel and Ruhle 1964; Abeles, Brown, and Morrow 1968; Holmberg 1973). The largest percentage increase in crack width is then expected in flexural members subjected to low levels of load because the cracks take more time to develop.

For both prestressed and reinforced concrete flexural members, long-term loading and repetitive loading give about the same crack widths and spacing (Rehm and Eligehausen 1977). The rate of crack width development, however, is considerably faster under repetitive loading (Bennett and Dave 1969; Holmberg 1973; Rehm and Eligehausen 1977; Stevens 1969).

As discussed in Chapter 4, crack width is a function of cover. For short-term static and fatigue loading, surface crack width is approximately proportional to the steel strain (Illston and Stevens 1972; Holmberg 1973; Stevens 1969). Crack widths increase under sustained loading at a decreasing rate. The rate of growth in crack width, however, is faster than the average observed surface strain at the level of the steel. For long-term loading, crack width is proportional to the steel strain (including the effects of creep), plus the strain induced in the concrete due to shrinkage (Illston and Stevens 1972).

Under initial loads, cracks intercepting reinforcement are restricted by the bond between the steel and the concrete (Illston and Stevens 1972; Broms 1965b), and the width of surface cracks does not provide a good indication of the exposure of the reinforcing steel to corrosive conditions. Over a period of time, however, the adhesion bond between the steel and the concrete undergoes breakdown. After about 2 years, the crack width at the reinforcement is approximately equal to the crack width at the surface (Illston and Stevens 1972). At this stage, cracks in flexural members are triangular in shape, increasing in width from the neutral axis to the soffit and are approximately uniform across the width of the beam.

5.3—Environmental effects

The long-term effects of an adverse environment in both producing and in enlarging concrete cracks (Mather 1957, 1968) can be damaging to both concrete and reinforcement. If concrete is not resistant to freezing and thawing when critically saturated, cracks will develop due to bursting effects of the freezing water. The lack of such resistance can be due to the following reasons: the use of resistant-resistant coarse aggregate, inadequate air-void system, or failure to protect the concrete from freezing before curing. Critical saturation in nonfrost-resistant concrete can occur by the presence of preexisting cracks that allow entry of water. The initiation of D-cracking near joints or other cracks in pavements is a good example. In more extreme cases, it is not uncommon for cracks caused either by thermal stress or shrinkage of the richer topping mixture in the roadway deck of dams and navigation locks to cause spalling due to the freezing of water in the cracks themselves independent of the frost resistance of the concrete. On the other hand, pre-existing cracks can also function to allow concrete to dry below critical saturation before freezing when this might not occur in the absence of such cracks. The role of cracks as they affect frost resistance will vary with the environmental conditions, such as, typical time of drying after wetting before freezing, crack width, and ability of cracks to drain.

Concrete durability is better when the aggregate used is durable under freezing and thawing conditions and the strength of the concrete is appropriate (ACI 201.2R). Field exposure tests of reinforced concrete beams (Roshore 1967) subjected to freezing and thawing in an ocean-side environment indicate that the use of air-entrained concrete made the beams more resistant to weathering than the use of non-air-entrained concrete. Beams with modern deformed bars were more durable than those using bars with old-style deformations. Maximum crack widths did not increase with time when the steel stress was less than 30 ksi (210 MPa), but did increase substantially (50 to 100%) over a 9-year period when the steel stress was 30 ksi (210 MPa) or more.

5.4—Aggregate and other effects

Concrete can crack as the result of expansive reactions between aggregate and alkalis present in the cement hydration, admixtures, or external sources, such as curing water, groundwater, and alkaline solutions stored or used in the finished structure.

Possible solutions to these problems include limitations on reactive constituents in the aggregate, limitations on the alkali content of cement, and the addition of a satisfactory pozzolanic material or a combination of these. The potential for some expansive reactions, such as alkali-carbonate, is not reduced by pozzolanic admixtures. ACI 201.2R and Woods (1968) give details on identification and evaluation of aggregate reactivity. ACI 221.1R gives guidelines on the alkali-aggregate reaction and selectivity process for mixture proportioning and durability.

Based on ACI 201.2R, ACI 212.3R, ACI 222R, and Mather (1957, 1968), the hazard of using calcium chloride, which may initiate corrosion, warrants a recommendation against its use when crack control is a major factor affecting long-term performance and durability of a structural system. Also, the use of calcium chloride in reinforced structures exposed to moist environments should be avoided regardless of the presence or absence of water-soluble salts in adjacent waters and soil.

Detrimental conditions can also result from the application of deicing salts to the surface of hardened concrete. Concrete to be subjected to deicers or similar chemicals should be air entrained and properly proportioned and cured to produce low permeability.

5.5—Use of polymers in improving cracking characteristics

Extensive work is available on the use of polymers in modifying the characteristics of concrete (Brookhaven National Laboratory 1968; ACI SP-40; ACI SP-58; ACI 548R). Polymer-portland cement concretes have a large deformation capacity, high tensile and compressive strengths, and negligible permeability. The tensile splitting strength can be as high as 10.7 MPa (1550 psi) (Nawy, Ukadike, and Sauer 1977). Polymer impregnation, though rarely used today, is another method of introducing beneficial polymer systems into concrete. These materials are discussed in greater detail in Chapter 6.

CHAPTER 6—CONTROL OF CRACKING IN OVERLAYS

6.1—Introduction

An overlay can be constructed by placing mortar or concrete over a concrete surface. The use of overlays has rapidly increased since the early 1970s. They are now commonly used for rehabilitation of deteriorated bridge decks; strengthening or renovating pavements, warehouse floors, walkways and other concrete flatwork; and in new two-course construction.

Overlays can be divided into three groups. The first group is when portland cement is used. These overlays can be low-slump dense concrete (LSDC), polymer-modified concrete (also called latex-modified concrete [LMC]), and fiber-reinforced concrete (FRC). These overlays may also contain silica fume, fly ash, or granulated blast-furnace slag. The second group includes polymer and epoxy mortars or concretes. The third group includes polymer-impregnated concrete (PIC), which has not become generally effective, economical, or practical. In a PIC system, hardened concrete is impregnated with a low molecular weight monomer that fills small cracks and voids to a shallow depth (about 5 mm [1/4 in.]) beneath the surface. The monomer is then polymerized and a relatively impervious surface layer results.

If the base slab is relatively crack free, or if the overlay is sufficiently thick and strong to resist the extension of cracks in the original slab, a well-bonded layer with matched joints is generally the best approach. If the overlay has sufficient thickness, a totally unbonded overlay is generally best where severe cracking is present or where it can later develop in the base slab. Systems that are essentially unbonded have been constructed satisfactorily where the overlay is placed over an asphalt layer. The asphalt itself acts as a debonding layer if it has a reasonably smooth surface without potholes. This type of construction lends particularly well to deteriorated airfield slabs that have been resurfaced with asphaltic concrete but require additional rigid pavement to take the increased loads of heavy aircraft. Another technique that has been used when the material to be overlaid is reasonably smooth consists of placing the overlay over a polyethylene sheet. On irregular, spalled, or potholed surfaces, a thin leveling and debonding layer of asphalt is desirable under the polyethylene sheet.

The main causes of cracking in overlays are:

- Plastic shrinkage caused by excessive evaporation due to environmental conditions while the concrete is in its fresh or plastic state;
- Differential drying shrinkage between material in the layer and the substrate concrete;
- Differential thermal stresses between the overlay and the substrate concrete. This can be caused by a different temperature in the layer as compared to the substrate and can also be caused or aggravated by different coefficients of thermal expansion and elastic properties;
- Reflective cracking from cracks in the substrate;
- Edge and corner curling stresses that can lead to delaminations and other cracking; and
- Poor construction practices.

Long-term observations (Schrader and Munch 1976; Bishara 1979; Shah and Skarendahl 1986) of many overlays have shown that cracking due to differential shrinkage is the most common problem. These cracks are also more likely to increase or widen with time. Another problem, delamination of the overlay, has been found to occur only at cracks in the overlay or at boundaries, normally at very early ages. These delaminations will spread with time.

To reduce the incidence of cracking in rigid concrete overlays, the following procedures are recommended:

- The surface of the underlying concrete should be thoroughly prepared to ensure adequate bonding of the overlay. This can be accomplished by mechanical methods, such as shotblasting, scabbling, hand chipping, or sandblasting, and hydraulically by high-pressure waterblasting (hydrodemolition). Scarifying methods that impact the surface can cause cracking in the substrate that can result in delamination. Procedures for each project should be selected considering the condition of the concrete, the availability of equipment, and the environmental conditions. The end result should be a clean, sound concrete surface;
- All equipment used for mixing, placing, and finishing should be designed for the type of overlay being used and should be accurately calibrated and in good working order. Both the contractor and inspecting personnel should be trained in the proper construction techniques of the particular overlay system;
- Material quantities, including total water content, *w/cm*, and amount of polymer, should be closely monitored and recorded;
- Traffic control should be evaluated for highway applications. The maintenance of traffic during reconstruction causes deflections, vibrations, or both in bridge decks. Consideration should be given to placing overlays when traffic is low, when vehicle speed is restricted, or both;
- Contraction joints in the deck should not be overlaid unless a joint or saw cut is immediately provided. Delayed saw cutting will usually result in a crack in the overlay over the joint, and quite possibly, some debonding adjacent to the joint. The preferred method is to form the joint with a compressible material and place the overlay against it. After curing, the compressible material can be removed and replaced with the final joint material;
- In new two-course construction of bridge decks, the overlay should be placed after removing the deck forms and shoring from the base concrete so that stresses caused by the weight of the overlay are carried by the underlying concrete. If placed before the forms are removed, the overlay will have to carry a portion of its own weight and can crack in negative moment regions;
- Overlays should be placed only when the ambient weather conditions are favorable, as defined in ACI 308 or when appropriate actions are taken for hot-weather (ACI 305R) or cold-weather concreting (ACI 306R). Evaporation rates of about 1 $kg/m^3/h$ (0.2 $[lb/ft^3]/h$), as measured from a free water surface, can cause plastic shrinkage cracking that can increase the extent of cracking and increase the probability of delamination. Curing procedures, such as wet mats and fog spraying, can be required. For large construction projects, such as pavement overlays, the evaporation rate should be monitored to determine when more stringent curing procedures should be used; and
- Mechanical shear reinforcement is effective in reducing cracking in overlays placed during periods of high evaporation rates.

6.2—Fiber-reinforced concrete (FRC) overlays

When properly proportioned, mixed, and placed, a crack-resistant topping layer of FRC can be the solution to certain field problems. Fibrous concrete overlays of highways, airfields, warehouse floors, and walkways have been used since the mid-1970s. Fibers are usually steel or polypropylene with lengths between 10 and 70 mm (1/2 and 2-3/4 in.). The effects of fibrous concrete on cracking in an overlay depend largely on the field conditions in each situation. (Schrader and Munch 1976; Shah and Skarendahl 1986; Shah and Batson 1987; ACI 544.2R; ACI 544.3R; ACI 544.4R)

The basic concept of FRC—that fibers arrest the growth of microcracks in concrete—is applicable to steel, synthetic (such as polypropylene), and mineral (such as glass) fibers. Steel fibers have a significant effect on the toughness of the concrete. Synthetic resin fibers have a lower modulus of elasticity and a poorer bond compared with steel fibers; they do not corrode but can reduce bleeding and plastic shrinkage cracking. Glass fibers are used primarily in cladding panels and other precast products that are formed by spraying chopped glass fibers and mortar slurry into forms at a precast plant. Glass fibers do not mix well in conventional concrete mixers. There are significant long-term durability problems associated with glass fibers (Hoff 1987; Shah, Ludirja, and Daniel 1987).

6.2.1 *Steel fiber concrete bond to underlying concrete*—In initial studies of FRC, it was believed that a partially bonded layer was the ideal system. The term partially bonded means that no deliberate attempt is made to improve the bonding between the topping layer and the underlying material through bonding agents, fasteners, and polyethylene sheets. The surface to be overlaid is cleaned of all loose material, usually by hosing, and left in a damp condition. Evaluations of partially bonded projects have indicated that this is the least-desirable technique to use. Over a period of years, many partially bonded FRC overlays have shown noticeable amounts of reflective cracking and edge curling. Curled edges are typical in thin overlays (less than about 75 mm [3 in.]), and they can result in cracks.

6.2.2 *Fiber size and volume*—The theory of FRC is based on a crack-arresting mechanism that depends on many parameters (Shah and Naaman 1976; Shah and Batson 1987). Some of the parameters that influence the reinforcing effect of fibers include the fiber's mechanical properties, aspect ratio (ratio of fiber length to fiber diameter), and the volume fraction of fibers (ratio of volume of fibers to volume of concrete). Increasing the aspect ratio or the volume fraction of fibers can enhance the crack-arresting mechanism, provided that fibers are uniformly distributed. If the number of fibers crossing a crack is relatively small, then the crack-arresting mechanism is limited.

6.2.3 *Fiber type and shape*—Because their resistance to pullout is greater, deformed steel fibers have a significant advantage over smooth ones with regard to both precracking and postcracking behavior.

6.2.4 *Fibers in open cracks*—There has been considerable discussion about the condition and effectiveness of steel fibers that cross a crack. At the time of cracking, fibers lose their adhesion to the concrete but continue to provide a mechanical resistance to pullout. This postcracking strength is one of the most important characteristics of FRC, and it can be significant for deformed fibers. The concern is that after cracking, steel fibers will oxidize and provide no long-term benefit. Investigations (Schrader and Munch 1976; ACI 544.2R; ACI 544.3R), however, have shown that if the crack widths are small (0.03 to 0.08 mm [0.001 to 0.003 in.]), the fibers will not corrode, even after years of exposure (Schrader and Munch 1976; Schupack 1985).

6.2.5 *Mixture proportioning considerations*—Even with a high-range water-reducer, the water requirement for fibrous concrete is higher than that of the same mixture without fibers due to reduced slump that accompanies the presence of fibers. The higher water demand of FRC tends to cause shrinkage cracks. Through the use of normal- or high-range water-reducing admixtures, the mixture water can be held to reasonable levels (Walker and Lankard 1977). Admixtures should be used to adjust mixture proportioning for bonded overlays so that the *w/cm* and cement content approach the same values as used in the underlying material. If possible, the overlay should have aggregates of similar physical properties, unless the original aggregates are unsuitable.

6.2.6 *Overlays over joints*—Different methods of placing unjointed overlays over joints in the underlying concrete have been tried; most have been unsuccessful (ACI 544.4R). As with conventional concrete overlays, if joints exist in a base slab, they should be maintained through the overlay.

6.3—Latex- and epoxy-modified concrete overlays

Bonded overlays of styrene-butadiene latex-modified mortar and concrete with a minimum thickness of 20 to 40 mm (3/4 to 1-1/2 in.), respectively, have been used in the renovation of bridge decks and in new two-course construction to effectively resist the penetration of chloride ions from deicing salts and prevent the subsequent corrosion of the reinforcing steel and spalling of the concrete deck (Bishara and Tantayanondkul 1974; Clear 1974). Overlays containing water-dispensable epoxy modifiers have also been used successfully, but on a much more limited basis. Latex- and epoxy-modified overlays are discussed in ACI 548R and ACI 548.1R.

Inspection of a large number of bridge decks overlaid with latex-modified concrete (Bishara 1979) revealed fine, random, shrinkage cracks in some projects. This type of cracking is not as extensive in new two-course construction. The random shrinkage cracks deserve special comment. At times they can be attributed to poor control or construction practices, such as the use of concrete with a high water content. Placement of an overlay in hot weather without adequate protection against early drying is also a cause of plastic shrinkage cracking.

On occasion, random pattern cracks have appeared even when the mixture proportions and construction methods followed good practice. Transverse cracks, spaced 3 to 4 ft (0.9 to 1.2 m) apart, have also been noticed in some bridge decks. The cracking can be due to unique conditions that cause thermal contraction of the surface while the substrate and bottom portion of the LMC layer does not experience similar thermal contraction. This shock usually occurs during the first night after placement when the overlay has rigidity but has not yet developed an appreciable tensile strength. Tight, random pattern and transverse cracks have caused concern from the standpoint of aesthetics, but they have not been a cause of overlay failure. Typically, such cracking is shallow (2 to 10 mm [1/16 to 3/8 in.]) and stable. A safe, conservative, and recommended approach is to treat these cracks with a penetrating high molecular weight methacrylate or low-viscosity epoxy or urethane, which can be broomed on the surface after the curing and drying period but before traffic is allowed on the overlay. The penetrant will generally fill and seal the surface cracks.

Finishing and texturing should be done rapidly behind the placement operation and before the polymer in the latex begins to dry or coalesce at the surface. Otherwise, tearing, scarring, and possible cracking can result. If, for example, a rake is used to groove a surface after it has begun to dry, tears about 13 mm (1/2 in.) long and 3 mm (1/8 in.) deep can occur. These will be oriented at right angles to the direction of raking. Texturing can be provided after the concrete has hardened using cutting wheels.

6.4—Polymer-impregnated concrete (PIC) systems

Surface impregnation and polymerization of concrete in place has been used in a number of field projects (Schrader et al. 1978). Practical difficulties were experienced in early projects and it has not become a popular procedure for treatment of slabs.

6.5—Epoxy and other polymer concrete overlays

Epoxy and other polymer concretes and mortars are discussed in ACI 548R and ACI 548.1R. These materials use a monomer or an epoxy as the binder, aggregate as the filler, and no water. Occasionally, portland cement or fly ash is added as a mineral filler. Overlays made with these materials are normally thin and do not use coarse aggregate. Typical applications using smooth surfaces are in food processing and sanitary or clean rooms or where a floor requires chemical resistance without a significant increase in thickness. Textured surface applications include bridge decks, parking garages, and stadium walkways. Because the reactions that harden these materials are normally highly exothermic, they cannot be used in thick placements or in hot weather without thermal stress problems.

Polymers have significantly higher coefficients of thermal expansion than concrete, even when aggregate fillers are used. Changes in temperature create normal and shear stresses at the interface of the overlay and base slab, which may result in cracking or delamination. To reduce cracking in PC overlays, thin overlays with low elastic modulus polymers should be used.

Polymer and epoxy concrete overlays can achieve excellent bond to dry surfaces. Subsurface preparation techniques that use water should be avoided. These overlays are vapor tight and should be carefully evaluated before applying, if transmission of water vapor through the overlay is desirable.

It is important to evaluate the moisture content of concrete to be overlaid. This is done by taping a piece of polyethylene (mat test) to the concrete. If moisture collects on the underside within the time frame that polymer or epoxy needs to cure, then the concrete should be allowed to continue to dry.

CHAPTER 7—CONTROL OF CRACKING IN MASS CONCRETE

7.1—Introduction

7.1.1 *Definition of mass concrete*—Mass concrete is defined by ACI 116R as “any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking.” Mass concrete structures consist of concrete dams, powerplants, bridge piers, and other large structural elements. Common to many mass concrete structures, water is directly in contact with the concrete, creating a high humidity or moist environment. By design, mass concrete water structures have little or no reinforcing steel. They should resist the loads through shape, size, and strength. Consequently, control of cracking is essential. Cracking can occur at any time during the life of the structure, and engineers should design the structure’s shape and materials to adjust to changing conditions (ACI 207.1R; ACI 207.2R; ACI 207.4R; ACI 207.5R).

7.1.2 *Types of cracks*—Cracking in mass concrete structures can result from material, structural causes, or both. Material-induced cracks result from drying shrinkage, a severe nonlinear thermal gradient initiated by heat of hydration and alkali-aggregate reaction. Surface cracking can appear as pattern cracking and result from a decrease in volume of the material near the surface or increase in volume below the surface. Random cracking from material-related causes can pass through a massive concrete element, and the crack widths can vary from hairline to wide. Structural cracking can result from individual loads or load combinations, such as gravity, liquid pressure, and severe impact. Structural cracks are of any width but generally align in a structurally plausible direction. The formation of a single, wide crack usually indicates the existence, before the crack’s formation, of principal tensile stress perpendicular to the crack. Structural distress is also noticeable near changes in geometry.

7.2—Methods of crack control

Given the probable temperatures and strains, the designer should determine what measures are most practical to prevent cracking. Some of the conditions that facilitate crack prevention are:

- Concrete with large tensile-strain capacity;
- Low cement content (permitted by low design stresses);
- Cement of low heat generation or use of pozzolans;
- Cast small concrete segments or blocks;
- Low placement temperature;
- Slow rate of construction when no artificial cooling is used;
- Artificial cooling by an internal network of cold water pipes;
- Insulate concrete surfaces;
- Low degree of restraint, as with yielding foundation, or in portions of the structure well removed from restraining foundation; and
- Absence of stress raisers, such as galleries.

There are two measures that can minimize cracking. The first is to modify the materials and mixture proportions to produce concrete with the best cracking resistance or the greatest tensile-strain capacity. This can require careful aggregate selection, using minimum cement content for interior concrete, restricting the maximum aggregate size. The attempt made to produce a concrete with a large tensile-strain capacity can limit the maximum aggregate size to a value somewhat below that which might be the most economical. Where several sources of aggregate are economical, preference should be given to that which provides the best resistance to cracking. Usually, this will be a crushed material of low thermal expansion and low modulus of elasticity.

The second measure to prevent cracking is to control the factors that produce tensile strain. This may mean precooling, postcooling, insulating, or possibly heating the exposed surfaces of the concrete and designing to minimize strains around galleries and other openings.

From these considerations, it is apparent that the degree of crack control necessary can vary from nothing at all for a dam near the equator or with favorable aggregates to very costly measures in a location where temperature variations are great and where the only economical aggregates have high elastic moduli and high thermal expansion.

In the case of a dam, the height affects the need for crack control. If the dam is very high, the design stresses can be high and a higher cement content is needed to attain the strength needed for the required safety margin. This results in more heat generation and a consequent tendency toward higher internal temperatures. Also, the higher dam will have greater horizontal dimensions that cause greater restraint and slow the rate of heat dissipation, resulting in the need for closer temperature control.

Recommended practice calls for both precooling and post-cooling and for the application of thermal insulation to exposed surfaces during cold weather. The insulation is left in place long enough to permit the concrete temperatures at the surface to slowly approach ambient or until additional concrete is placed on or against the surface being protected.

After the anticipated temperature history is established, the determination of probable tensile stress is the next step. This can be accomplished using finite-element computer programs (Wilson 1968; Polivka and Wilson 1976; Liu, Campbell, and Bombieh 1979) that are formulated from a heat-transfer viewpoint. Even with the finite-element approach, analysis is laborious because of the time-dependent variables. Problems include that the data must be generated to establish these time-dependent properties for the concretes to be used. The analysis should include many time steps to account properly for creep (or relaxation) and for the different or changing properties of each lift of concrete.

On the other hand, stresses near a boundary due to brief thermal shocks can be computed quite readily (ACI 207.2R) because the concrete can be assumed to be fully restrained in such cases. If there is full restraint, the strain is zero, and the stress is obtained by multiplying the elastic modulus by the strain that would have occurred if there were no restraint. The strain is the temperature drop multiplied by the coefficient of thermal expansion. This situation is important because the control of boundary strain is sufficient to control cracking in many cases. Internal strains develop slowly enough to be tolerable, even if they are large.

7.3—Design

Design of mass concrete structures means determining the appropriate loads and load combinations together with a reliable method of structural analysis to arrive at an economical geometric shape meeting safety criteria. Loads and combinations will vary for the different structures, such as hydrostatic pressure, gravity, ice, silt, temperature, earthquake, superstructure weight, differential foundation conditions, and impact.

Material tests are conducted on cylindrical specimens that are not completely representative of the structure because aggregate larger than 40 mm (1.5 in.) is removed from the mixture by wet screening before 300 mm (6 in.) diameter cylinders are prepared (USBR 1981). The maximum stress is the design stress that is based upon structural requirements. It is considered good practice to use a safety factor as high as three or four, meaning that the strength should be three or four times the expected maximum stress. Strength tests at 2, 7, 14, 28, 90, 180 days or 1 year are often used. The selected time period is related to the expected time until loading. For example, if full loading is not until 1 year, a lower early-strength concrete can be prescribed consistent with the construction sequence anticipated. Because the cylinders made at the site are made from wet-screened concrete, the measured strength is corrected to an equivalent mass concrete by applying a reduction factor of about 0.80 for typical conditions or else developing a factor by comparing wet-screened cylinder strengths to those obtained from 12 or 24 in. (300 or 600 mm) cylinders. Specific data on appropriate reduction factors with varying parameters can be found in USBR (1981).

For interior concrete, the lowest practical strength should be specified to reduce the cement content. This will reduce the heat of hydration and the consequent thermal gradient, decreasing the likelihood of cracking. More than the necessary amount of cement is detrimental rather than advantageous. A compromise between early strength and heat of hydration can often be obtained by replacing a significant part of the cement with pozzolan. A one-for-one replacement of portland cement with pozzolan can reduce the heat of hydration by one-half of that generated by the same amount of cement.

7.3.1 *Safety against sliding*—Uncracked concrete provides a large factor of safety against sliding.Weak horizontal lift joints, however, can impair safety. Therefore, the specifications should require care in the preparation of lift surfaces and in the placement and compaction of concrete onto them. In the case of a dam, the lift surfaces should slope slightly upward toward the downstream edge to improve drainage during lift surface preparation as well as shear resistance during operation of the dam. It is not necessary to use a mortar layer on lift surfaces before placing the next lift. These conditions are appropriate for conventionally placed concrete. In roller-compacted concrete construction, a layer of mortar is placed between lifts near the upstream face as a protection against seepage.

7.3.2 *Economy*—Many factors that affect economy affect crack resistance. For example, the least-expensive aggregate can have undesirable thermal properties and require expensive temperature control to prevent cracking. The aggregate that imparts concrete with the highest tensile-strain capacity can increase the water requirement and the cement requirement, offsetting the benefits of high strain capacity. Similarly, lesser-quality aggregates can have a lower stiffness than the surrounding matrix and not carry their proportionate share of the load.

7.3.2.1 *Selection of aggregates*—The aggregates chosen should be those that make good concrete with the lowest overall cost. If natural aggregate near the site has unfavorable properties for crack prevention, crushing to increase crack resistance can be an economical expedient because of the consequent saving in temperature control. When crushing is either advantageous or necessary, rock that has the most favorable properties should be chosen. The rock should have a low coefficient of thermal expansion and a low modulus of elasticity, and it should produce particles with good shape and surface texture. Rock elastic moduli higher than the matrix will concentrate load on the rock, producing an unequal stress distribution in compression. Stronger rock will also act as a crack arrestor to cracking of the matrix. These factors are important considerations in increasing the resistance of the concrete to cracking.

7.3.2.2 *Aggregate size*—The maximum-size aggregate should be determined by the ability to place concrete properly in the structure. Up to a 150 mm (6 in.) diameter aggregate can be used, except when concrete should resist high-velocity water flow and when structural concrete is needed. Larger aggregates permit the use of less water and cement for a given volume with a reduction in the amount of temperature control required to achieve a particular level of crack resistance. The advantages of larger aggregates should be weighed against the need for larger equipment and the potential for workability and segregation problems. The trend in recent years is to limit maximum-size aggregate to 75 mm (3 in.) for conventional mass concrete and 50 mm (2 in.) for roller-compacted concrete.

7.3.2.3 *Water content*—A reduction in the water content of concrete permits a corresponding reduction in the cement content. Concrete with less water and cement is superior because it undergoes less temperature change due to hydration effects, less drying shrinkage, and is more durable and crack resistant. Minimum water content can be achieved by specifying adequately powered vibrators that permit the use of low-slump concrete by using a water-reducing admixture when appropriate and by producing and placing the concrete at lower temperatures.

7.3.2.4 *Pozzolan*—In most cases, good pozzolans, such as fly ash, are available and can be used to replace a portion of the cement. This can result in a considerable cost saving, and more importantly for mass construction, can reduce the heat generation and improve the resistance against cracking. Another advantage of using pozzolan is that when used in adequate amounts it can reduce the expansion due to reactive aggregates. The appropriate amount of pozzolan for a reactive aggregate should be based on test data obtained with the specific aggregate, pozzolan, and cement; it can be in the range of 25 to 35%. Pozzolans tend to reduce early strength gain that could delay form stripping (ACI 221R).

7.3.3 *Durability*—Durability of portland cement concrete is defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration (ACI 201.2R). The most common weathering action is multiple cycles of freezing and thawing of water near the surface that fracture and deteriorate the concrete over time. There are chemicals in the environment that lessen the life of the concrete. Naturally occurring sulfate can combine with calcium and alumina in hardened concrete to form products that can cause an increase in solid volume accompanied by cracking and deterioration or a substantial reduction in strength of the paste matrix. Abrasion on dams and bridge abutments from abrasive materials carried by flowing water or from high-velocity flow causing cavitation can be mitigated with the proper mixture, hydraulic design, or both.

Mass concrete structures, normally unreinforced, can have an outer layer of temperature reinforcement that requires corrosion protection similar to that of reinforced structural concrete.

Table 7.1—Elastic and thermal properties of mass concrete

Elastic properties					
Static modulus of elasticity E at indicated age of test, days					Poisson's ratio
1	3	7	28	90	
46.4 GPa (0.66×10^6 psi)	141 GPa (2.00×10^6 psi)	180 GPa (2.56×10^6 psi)	281 GPa (4.00×10^6 psi)	352 GPa (5.00×10^6 psi)	0.15 to 0.25

Thermal properties			
Coefficient of linear expansion	Conductivity	Diffusivity	Specific heat
7 to 15×10^{-6}/C	2.00 to 3.31 kg·cal/m·h·C	0.0037 to 0.0062 m^2/h	0.22 cal/g·C
(4 to 8×10^{-6}/F)	(1.35 to 2.24 BTU/ft·h·F)	(0.040 to 0.067 ft^2/h)	(0.22 BTU/lb·F)

An alkali-silica reaction can result when alkali in cement (or from other sources) reacts with certain aggregates, such as argillites, graywakes, phyllites, quartzites, schists, strained quartz, opal, chalcedony, and rhyolites (ACI 221R). Cracking usually appears first on the surface as pattern cracking (ACI 201.1R) within a few years after construction.

Concrete durability is closely related to exposure conditions. In tropical climates, for example, there may be no deteriorating influences acting on the concrete except cavitation in areas subjected to high-velocity water flow. For the main structure areas not subject to high-velocity water flow, concrete that has the required strength can be expected to last well beyond the design life of the structure. In those cases, cement content should be kept low to minimize heat generation and cracking.

In a severe climate, with repeated freezing and thawing in winter, the *w/cm* of surface concrete should be kept lower than that necessary for strength alone. Higher-strength concretes (28 MPa [4000 psi] at 1 year) are often specified for durability. Air entrainment should be mandatory. For any concrete that might be subjected to both freezing and water pressure, the *w/cm* should be about 0.4. The effect of the higher-quality concrete on thermally induced cracking will be minimized by keeping the thickness of the boundary between layers to a minimum, probably 0.6 m (2 ft) or less.

7.3.4 *Material properties*—The concrete's properties affect crack control. Concretes differ widely in the amount of tensile strain they can withstand before cracking. For strain that is applied rapidly, the two factors that govern the strain capacity are the modulus of elasticity and tensile strength. For strain that is applied slowly, the creep (or relaxation) of the concrete is important.

When loading is applied rapidly, many tests on lean concretes, such as those used for the interior of large dams, have shown that tensile failure occurs without much plastic strain. For these lean concretes, the tensile strain that the concrete can withstand is approximately equal to the tensile strength divided by the modulus of elasticity of the concrete in tension. It is accurate to assume that the tensile-strain capacity is inversely proportional to the modulus of elasticity of the concrete. The modulus of elasticity of the aggregate is important because of its large effect on the deformability of the concrete.

Tensile strength is also important, and for this reason, crushed aggregates are apt to be superior to natural aggregates for crack prevention.

7.3.4.1 *Modulus of elasticity*—This subject is discussed in detail in ACI 207.1R. Table 7.1 shows values of the modulus of elasticity of a particular concrete after various ages of curing.

7.3.4.2 *Crack resistance*—The tensile strain that concrete can withstand varies greatly with the composition of the concrete and the strain rate. When strain is applied slowly, the strain capacity is far greater than when the strain is applied. Therefore, concrete in the interior of a large mass that must cool slowly can undergo a large strain before cracking. If concrete contains rough-textured aggregate with small maximum size, the strain capacity will be high. There is an optimum, however, with respect to aggregate size. Concrete with smaller aggregates requires more cement for a given strength. This results in more heat, a higher maximum temperature, and greater subsequent strain due to cooling. Therefore, these effects can offset the strain increase developed from the use of smaller aggregates.

7.3.4.3 *Tensile-strain capacity*—Tensile-strain capacity tests are performed on plain concrete beams under third-point flexural loading. Relatively large beams, ranging from 300×300 mm (12×12 in.) to 600×600 mm (24×24 in.) in cross section and 160 to 3250 mm (64 to 130 in.) long are often used (Houghton 1976). Strain capacity is determined from these tests under both rapid and slow loading to simulate both rapid and slow temperature changes in the concrete. Expressed as extreme fiber stresses, the loading rates are generally 0.3 MPa/min (40 psi/min) for rapid loading and 0.17 MPa/week (25 psi/week) for slow loading.

For rapid loading, the strain can be measured using either surface or embedded strain gages or meters (Houk, Borge, and Houghton 1969; Houghton 1976). Embedded meters are best for long-term tests. The strain can also be determined from deflection measurements. The concrete test beam used for determining the strain capacity should be protected by wrapping with an impermeable material during the test to prevent loss of moisture. Testing should be conducted at a constant temperature for maximum accuracy in measurement. Detailed test procedures can be found in Houk, Borge, and Houghton (1969) and McDonald, Bombieh, and Sullivan (1972). In the preliminary studies of temperature and construction control plans for mass concrete projects, approximate methods for estimating tensile-strain capacity under rapid and slow loadings given by Houghton (1976) and Liu and McDonald (1978) can be used.

7.3.4.4 *Thermal properties*—Thermal diffusivity and thermal expansion are important in the control of cracking due to temperature change; their determination is detailed in ACI 207.1R. The approximate range of thermal properties is shown in Table 7.1.

The coefficient of thermal expansion is an important property of concrete. The amount of strain that a temperature change will produce is directly proportional to the coefficient of thermal expansion of the concrete. The average coefficient of thermal expansion of mass concrete is about 5×10^{-6} F (9×10^{-6} C), but the coefficient can range from 4 to 8×10^{-6} F (7 to 15×10^{-6} C). Therefore, in the extreme case where a concrete has a low tensile strength, a high modulus of elasticity, a high coefficient of thermal expansion, and it is fully restrained, the concrete can crack when there is a rapid drop in temperature of only 6 F (3 C). On the other hand, some concretes can withstand a quick drop in temperature of as much as 20 F (10 C), even when fully restrained. More data on the thermal expansion of concrete can be found in ACI 207.1R and ACI 207.2R.

Tensile stress in mass concrete results mainly from the restraint of thermal contraction, and to a far lesser degree, from autogenous shrinkage. Drying shrinkage is important because it can cause shallow cracks to occur at the surfaces. Temperature change is a principal contributor to tensile strain in mass concrete. Predicting probable strain requires the expected temperature, which can be determined if the adiabatic temperature curve for the concrete is known, as well as the thermal diffusivity, boundary temperatures, and dimensions. The temperature rise that would occur if there were no heat loss is defined as adiabatic temperature rise. The finite-element method can be used for predicting the temperature distribution. The main problem is choosing the correct boundary temperatures, which often depend upon the ambient air or water temperatures. It is satisfactory to use air temperatures provided by the National Weather Service. For dams, measured reservoir temperatures are used. Refer to ACI 207.1R for information on other methods for predicting temperatures in massive concrete.

The heat-producing characteristics of cement play an important role in the temperature rise. ASTM C 150 Type IV, low-heat cement is recommended; however, it usually is not available. Type II moderate-heat cement is also recommended, and it usually is available. Pozzolans can be used to replace a portion of the cement to reduce the peak temperature due to the heat of hydration (ACI 207.2R). In some cases, 35 to 50% of the cement can be replaced by an equal volume of a suitable pozzolan and still produce the specified strength at 90 days or 1 year. Some of the more common pozzolans used in massive concrete include calcined clays, diatomaceous earth, volcanic tuffs and pumicites, and fly ash. The actual type of pozzolan to be used and its appropriate replacement percentage are determined by test, cost, and availability.

7.3.4.5 *Adiabatic temperature rise*—ACI 207.1R describes test methods and provides data on adiabatic temperature rise of concretes with a single cement content but with different types of portland cement. Figure 7.1 gives typical adiabatic curves for Type II cement and various quantities of cement and pozzolan. The data show the effect of pozzolan replacement of cement on temperature reduction.

7.4—Construction

7.4.1 *Basic considerations for construction*—Mass concrete structures are required to be safe, economical, durable, and aesthetically pleasing. Each of these requirements influences the crack resistance. The cost will depend on features such as the best choice of aggregates, adequate but not excessive temperature control, and low cement content. Durability

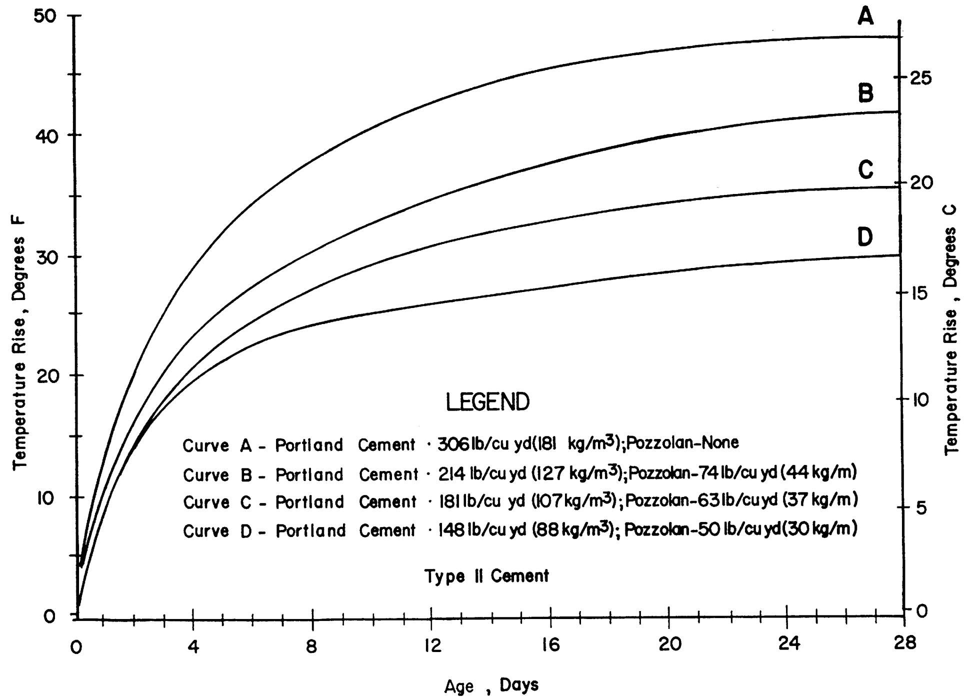

Fig. 7.1—Typical adiabatic temperature curves for mass concrete (Houghton 1969).

will depend on the quality of the concrete, the exposure conditions, and the freedom from deteriorating chemical reactions. Aesthetics will come from good workmanship, freedom from cracks and strains, and absence of leakage and leaching. It is important to have a comprehensive materials test program to establish necessary control before preparing construction controls and specifications.

7.4.2 *Thermal effects*—Temperature-induced cracking in a large mass of concrete can be minimized if proper measures are taken to reduce the amount and rate of temperature change. Measures commonly used include precooling; postcooling, or a combination of the two; and thermal insulation to protect exposed surfaces. The degree of temperature control necessary to control cracking varies with factors such as location, height, and thickness of the structure, character of the aggregate, properties of the concrete, and the external restraints. Although a large amount of the data for this chapter has been obtained by experience from using mass concrete in dams, it applies equally well in mass concrete used in other structures, such as steam powerplants, powerhouses, bridge and building foundations, and navigation locks. Tremie concrete is covered in ACI 304R. Roller-compacted concrete is discussed in ACI 207.5R.

Precooling the concrete during its production and postcooling it with embedded pipe systems after it is placed are effective measures. Details on pipe cooling are given in Section 7.4.2.1.

Another promising method of crack control is placing crack-resistant concrete at boundaries (sides and top of lifts). Even though the more crack-resistant concrete can be too costly to be used throughout the structure, it can be used to limited extent without serious effect on cost. Casting thin layers of concrete next to the forms is difficult, but the use of precast panels is practical; these are left in place as a permanent part of the structure. The panels should be of good quality for durability and lightweight to provide thermal insulation. Because most cracks originate at boundaries, this method improves crack control. More information on the use of precast panels for protecting mass concrete can be found in ACI 347R.

7.4.2.1 *Artificial cooling*—The overall program for cooling concrete, including important field-control criteria, should be determined during the design stage. Precooling concrete before placement is accomplished by a variety of methods, including evaporative cooling of the stockpiled aggregates using sprinklers, cooling all ingredients of the mixture by liquid nitrogen, or using small ice particles as a replacement of part of the mixing water. When these precautions and natural cooling are ineffective in reducing cracking, postcooling is prescribed, especially for large concrete dams. Postcooling concrete is accomplished by circulating cool liquids (usually water) through pipes embedded in the concrete. The water source from a nearby lake or river can be directly circulated or refrigerated as needed.

Studies made during the design stage will establish items such as lift height, pipe spacing, water temperature, flow rate, acceptable rate of temperature drop (for both rapid and slow drops), and approximate duration of cooling.

Steep cooling gradients, which can result in cracking the mass, should be avoided. This is particularly true in smaller masses where circulation of cooling water should be stopped when the maximum temperature has been reached and begins to drop. A vulnerable location in pipe cooling systems is centered at the cooling coils where sharp gradients and cracking can be induced if termination of cooling water circulation is not timely. A primary advantage in artificial cooling is the uniform removal of the heat of hydration and the precise control of the rate. In mass concrete, a maximum rate of cooling of 0.6 C (1 F) per day or less is recommended but should be monitored for suitability during construction.

Thermal couples should be used in sufficient numbers to permit adequate monitoring and control of the internal concrete temperatures. They should be located to record representative temperatures. For example, a grid that records thermal gradients in a concrete dam would consist of three to five (depending on structure size) transverse planes, each containing five to 10 thermal couples. Spacing of the thermal couples should be closer near the dam faces.

Construction drawings should show basic pipe layout, including minimum spacing, the layout at faces, transverse construction joints, interior openings and in sloping, partial, and isolated concrete lifts, pipe diameter, risers, and pipe connections (Houghton 1969). In most of the dam, a uniform spacing can be maintained for the cooling pipe, but isolated areas exist in all dams that tend to have a concentration of pipes. These concentrations tend to occur at the downstream face of the dam where inlets and outlets to cooling pipes are located, adjacent to openings in the dam, and at isolated and sloping lifts of concrete. Proper planning will alleviate many of the undesirable conditions that can result from these concentrations. For example, it should be determined to what extent the cost-saving procedure of concentrating cooling pipe inlets and outlets near contraction joints can be permitted at the face of the dam. Also, it should be decided if cooling pipes to isolated areas in the foundation and at openings, such as galleries, can extend from the downstream face of the dam or if a vertical riser should be used.

For ease of installation, the pipe used for postcooling should be thinwall tubing. Aluminum tubing is lightweight and easy to handle; however, breakdown from corrosion is a potential problem if cooling activities are carried on over a period of several months. In this case, steel tubing is preferred. Surface connections to the cooling pipe should be removable to a depth of 100 to 150 mm (4 to 6 in.) so that holes can be reamed and dry-packed when connections are removed.

Coils should be pressure tested for leaks at the maximum pressure they will receive from the cooling system before placing concrete. Pressure should also be maintained during concrete placement to prevent crushing and permit early detection of damage, should it occur.

Sight-flow indicators should be installed at the end of each embedded pipe coil to permit ready observance of cooling water flow. In addition to regular observance of flows, water temperatures, and pressures, concrete temperatures should be observed and recorded at least once daily while the lift is being cooled. Thermometers are placed at the inlet and outlet of the cooling water system to record the temperature and change throughout the placement.

The refrigeration plant for cooling water can be located centrally, or several complete smaller portable plants can be used to permit moving the refrigeration system as the dam progresses upward. Sufficient standby components, equal in capacity to the largest individual refrigeration units, should be provided. Insufficient plant size can cause premature shutdown of cooling before the heat of hydration has been sufficiently dissipated or slow concrete placement to a rate that interferes with the overall construction schedule.

Cooling should continue until the prescribed temperature is reached. This temperature is dependent on the type of structure and loading. The prescribed temperature can be the mean annual temperature or other temperature as a compromise for the seasonal variations.

Some mass concrete structures can have vertical contraction joints to facilitate construction; these joints will open as the concrete cools. In some dams (especially arch type), these contraction joints are later grouted to restore monolithic behavior.

After cooling is completed and the pipe is no longer needed, it should be grouted full with a cementitious mixture. Grout techniques used for post-tensioned concrete should be used.

7.4.2.2 *Natural cooling*—Thermal insulation on exposed surfaces during cold weather can protect concrete from cracking if enough insulation is used for a proper length of time. If the insulation is sufficient to allow slow cooling, the tensile strain need never exceed the cracking limit. The concrete can relax at the same rate as the tensile stress tends to develop until stable temperatures are reached. If the concrete has a very slow relaxation rate (or creep rate), however, the amount of insulation and the long protection time required can make this measure difficult.

In extreme environments where large amounts of insulation will be required during severely cold months, it may be necessary to remove the insulation in stages as the warmer months approach. Temperatures within the concrete just below the insulation should be allowed to slowly approach the environmental temperature to prevent thermal shock, which could induce cracking at or below the surface with possible subsequent deeper penetrations into the mass. Avoid using too much insulation or leaving it in place too long, which could result in stopping the desired cooling of the interior mass and possibly cause the interior temperature to begin to increase again.

Insulation can be obtained in a variety of forms and materials. It can be either semirigid panel-type material, or foamed spray-on material that becomes semirigid in place. The semirigid panels are usually installed on the inside face of the forms. Temporary anchors embedded in the newly placed lift of concrete retain the insulation on the concrete surface when the forms are lifted. The insulation can be easily removed from the surface. Roll-out application is particularly applicable for use on horizontal lift joints. It is easy to install and remove and can be reused many times. Spray-on insulation can be used on either horizontal or vertical surfaces. This type of insulation is particularly useful for increasing the thickness and effectiveness of insulation already in place and for insulating forms. Insulation that permits transmission of light rays should not be used because a temperature rise occurs between the insulation and the concrete when the insulation is subjected to direct sunlight. Precast panels made of low-conductance lightweight concrete or regular-weight concrete cast with laminated or sandwich layers of low-conductance cellular concrete are also acceptable for insulation. These panels would then serve as both forms and face concrete.

Table 7.2(a)—Sustained modulus

Sustained modulus E_s at time of loading in days, GPa (psi)				
Time after loading, *d*	1	3	7	28
0	47.6 (680,000)	134 (1,920,000)	183 (2,610,000)	303 (4,330,000)
1	46.2 (660,000)	123 (1,760,000)	172 (2,460,000)	263 (3,760,000)
3	44.8 (640,000)	113 (1,620,000)	151 (2,150,000)	234 (3,334,000)
7	44.1 (630,000)	95 (1,350,000)	139 (1,980,000)	210 (2,990,000)

Table 7.2(b)—Stress coefficients

Tensile stress coefficient for condition of full restraint and decreasing temperature for age of concrete at time of loading in days, MPa/C (psi/F)				
Time after loading *d*	1	3	7	28
0	0.047 (3.7)	0.133 (11.0)	0.1181 (14)	0.300 (24)
1	0.046 (3.6)	0.122 (9.7)	0.1170 (14)	0.260 (21)
3	0.045 (3.5)	0.112 (8.9)	0.1150 (12)	0.231 (18)
7	0.044 (3.5)	0.094 (7.4)	0.1138 (11)	0.208 (16)

Cooling mass concrete structures can be permitted without insulation in relatively thin sections of about 7 m (20 ft) in moderate climates where the natural dissipation of heat is gradual and the thermal gradient is not steep.

7.4.3 *Autogenous volume change*—Autogenous volume change is the expansion or contraction of the concrete due to causes other than changes in temperature, moisture, or stress; therefore, it is a self-induced expansion or contraction. Expansion can be helpful in preventing cracks, but contraction increases the tendency to crack. Autogenous volume change is usually measured by strain meters embedded in concrete cylinders that are carefully sealed to ensure that there is no loss of moisture, and the concrete kept at constant temperature. Measurements begin as soon as the specimens are hardened and sealed, and are continued periodically for months.

7.5—Operation

After construction, the structure is exposed to the annual cycle of load combinations for which it is designed. During the initial operation period, which may last for a few months or even years, the structure and the immediate foundation will undergo some elastic movement superimposed on a permanent set. During the succeeding years, small permanent deformations occur in the structure, foundation, or both, that are categorized as creep. Creep can initiate unanticipated cracking by causing a load shift to structural areas not intended to deform or carry more of the loads. These conditions can be somewhat anticipated with analyses that include concrete creep and designing the structure to accommodate these movements.

7.5.1 *Concrete creep*—Creep is the continued deformation of concrete during sustained stress. ASTM C 512 details a standard test for determining concrete creep in compression. Concrete creep in tension is difficult to measure; therefore, creep properties in tension and compression are assumed to be the same.

Concrete creep is measured on carefully sealed specimens stored at constant temperature and loaded by a constant stress. Measurement is usually made by means of embedded strain meters, although any reliable method of measuring strain can be employed. Butyl rubber is satisfactory for sealing the specimens, but neoprene should be avoided because it allows some moisture to escape. Specimens should be loaded at the same ages as specified for the modulus of elasticity tests, but loading at the early age of 1 day is not always practical. Again, the specimens should be large enough to permit concrete like that used in the structure. Frequently, 280 × 560 mm (9 × 18 in.) cylinders with 75 mm (3 in.) maximum-sized aggregate, or 150 × 400 mm (6 × 16 in.) cylinders with 40 mm (1-1/2 in.) aggregate are used. Useful coefficients for converting concrete creep with smaller-size aggregates to concrete creep for larger aggregate sizes are found in ACI SP-9. Figure 7.2 shows typical creep data obtained from laboratory investigations (McCoy, Thorton, and **Allgood 1964). Table 7.2** indicates the values for the sustained modulus of elasticity, which are used to develop tensile stress coefficients for a temperature drop with full restraint. ACI 209R presents a discussion of the sustained elastic modulus. A high rate of creep for concrete is helpful when the tensile strain is applied gradually. Because the tensile strength of concrete is nearly independent of prior loading, creep tends to increase the strain capacity. In the case of Dworshak Dam, for example, the strain to failure was almost three times as great for strain applied over 2 months, compared with quickly applied strain (Houk, Paxton, and Houghton 1970).

The concrete creep under sustained stress is affected by the stiffness of the aggregate. When the modulus is high, the creep is low and vice versa. The importance of aggregate rigidity on concrete creep can be illustrated by two examples. First, assume that the aggregate and the cement paste have the same modulus of elasticity. When compressive stress is applied, the stress and corresponding strain will be the same in the aggregate as in the cement paste. The aggregate does not creep under moderate stress but the paste does, and the paste that is between aggregate particles relaxes and transfers load to the aggregate to maintain equilibrium. This imposes an elastic strain on the aggregate that accounts for a large part of the concrete creep. The amount of elastic strain is directly related to the modulus of elasticity of the aggregate; the more rigid the aggregate, the lower the creep. Next, assume that the aggregate has a much higher modulus than the cement paste. When compressive stress is applied, the average stress in the aggregate will be higher than that in the paste and the paste will creep less than it did when the moduli were equal. The elastic strain in the aggregate due to the creep of the paste will then be less than it was when the moduli were equal. Therefore, an increase in the rigidity of the aggregate acts in two ways to reduce the creep of the concrete.

CHAPTER 8—CONTROL OF CRACKING BY PROPER CONSTRUCTION PRACTICES

8.1—Introduction

Construction practices discussed in this chapter include specifications, materials, designs, mixture considerations, and on-the-job construction performance. Before discussing the control of cracking by proper construction practices, it is

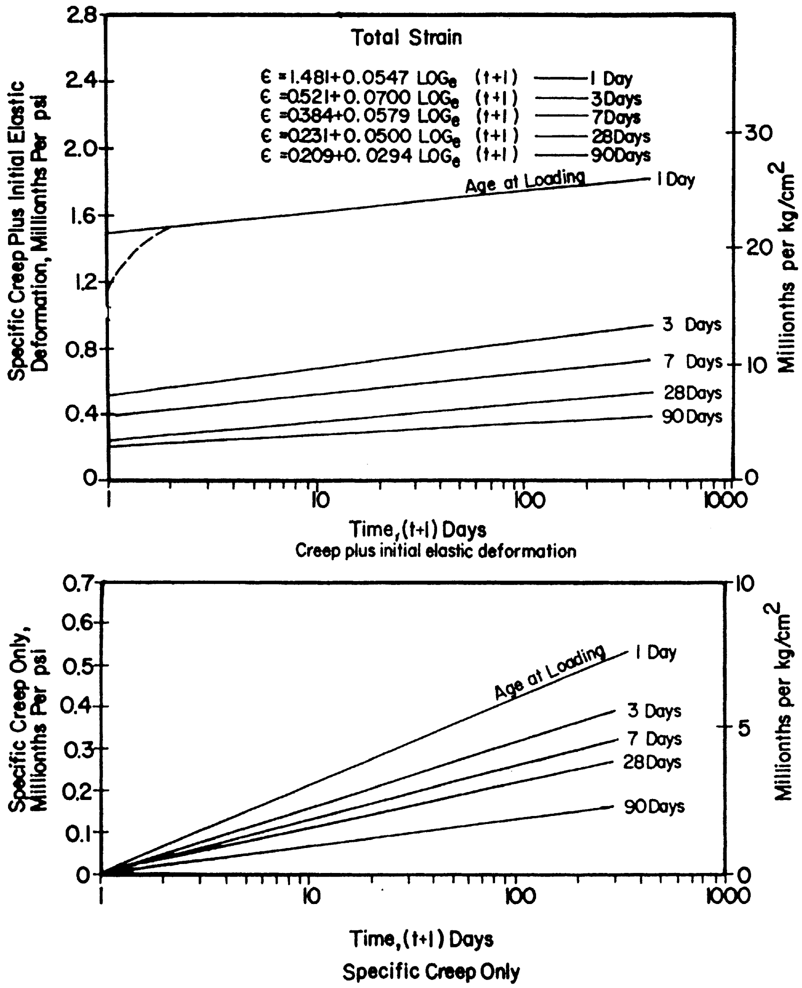

Fig. 7.2—Typical concrete creep curves for mass concrete.

worthwhile to mention the basic cause of cracking related to volume change of concrete—restraint. If all parts of the concrete in a concrete structure are free to move as the concrete expands or contracts due to volume change, there will be no cracking.

Obviously, however, all parts of concrete structures are not free, and inherently, cannot be free to respond in the same degree to volume changes. Consequently, differential strains develop and tensile stresses are induced. When these differential stresses exceed the capability of the concrete to withstand them, cracking occurs. New concrete should be protected for as long as possible from the loss of moisture or drop in temperature. These considerations can result in stresses capable of causing cracks at an early age but that might be sustained at greater maturity. Concrete should have a high tensile-strain capacity, which is influenced by the aggregate. Therefore, a low modulus of elasticity in tension is desirable.

8.2—Restraint

Restraint exists in many circumstances under which the structure and its concrete elements should perform. Typical examples illustrate how restraint can cause cracking if the concrete is not strong enough to withstand the tensile stresses developed.

A wall or parapet anchored along its base to the foundation, or to lower structural elements less responsive to volume changes, can be restrained from shrinking when its upper portions shorten due to drying or cooling (ACI 224.3R). Cracking is usually inevitable. Due consideration should also be given for temperature drops in wall construction. Contraction joints (or at least grooves of a depth not less that 10% of the wall thickness on both sides) in which the cracks can occur and be hidden should be provided at intervals ranging from one (for high walls) to three (for low walls) times the height of the wall.

The interior and exterior concrete, particularly in larger, thicker sections, can be subjected to changes in temperature or moisture content at different rates and different degrees. When this happens, the interior concrete restrains the exterior concrete from shrinking, and tensile stresses develop that can cause the surface to crack. This occurs when the surface cools while the interior is still warm from the heat of hydration or when the surface concrete dries faster than the interior concrete. It is often feasible to protect the surface at early ages so that stress-inducing differential strains cannot develop before the concrete is strong enough to withstand the strain without cracking. Reinforcement provided for both thermal contraction and drying shrinkage can only partially restrain the contraction of surface concrete, but more and narrower cracks can result.

Restraint can occur at changes in sections because of differences in temperature or drying shrinkage in the two sections. If feasible, a contraction joint can be used to relieve the restraint. Restraint of flatwork results from anchoring slab reinforcements in perimeter concrete footings. When a slab is free to shrink from all sides toward its center, there is minimal cracking. Contraction joints and perimeter supports should be designed accordingly.

Walls, slabs, and tunnel linings placed against the irregular surface of a rock excavation are restrained from moving when the surface expands or contracts in response to changes in temperature or moisture content. Closely spaced contraction joints or deep grooves should be provided to hide the cracks that would disfigure such surfaces. In tunnel linings, the contraction in the first few weeks is primarily thermal, and the use of cold concrete 10 C (50 F) can reduce cracking materially. By the time drying is significant, the concrete lining has a greater tensile strain capacity and therefore is able to resist shrinkage cracking. Circumferential cracks in tunnel linings and other cast-in-place concrete conduits and pipe lines, however, can be greatly reduced in number and width if a bulkhead is used to prevent air movement through the tunnel, and shallow ponds of water are placed in the invert as soon as possible after placing the lining. They should remain there until the tunnel is put into service (USBR 1981). The concrete can become much stronger in the humid environment and will be better able to resist shrinkage-induced tensile stresses if allowed to dry in service. If the tunnel carries water, there will be no further drying shrinkage.

The previous examples indicate that many crack-control procedures should be considered during design. While proper construction performance can contribute a great deal (as will be discussed), the contractor cannot be expected to use the best procedures unless these procedures are included in the contract documents.

8.3—Shrinkage

8.3.1 *Effect of water content*—As shown in Fig 3.5, concrete drying shrinkage is proportional to mixture water content. The use of the lowest practical slump adds in the control of cracking. Of major importance is the selection of mixture proportions that require the least amount of water for the desired concrete strength and durability. This means avoiding oversanded mixtures, using the largest maximum aggregate size practical, and using aggregate with the most favorable shape and grading conducive to workability. Also use a well-graded sand with a minimum of fines passing the 100 mesh and free of clay, such that its sand equivalent value is not less than 80% (AASHTO T 176). Shrinkage tends to be proportional to the total paste volume in the mixture.

8.3.2 *Surface drying*—Surface drying will occur except when the surface is submerged or below grade. Drying shrinkage strains of up to 600×10^{-6} or more are likely. The amount of shrinkage cracking depends on factors such as surface dryness; mixture proportions, especially mixing water; the character and degree of restraint; and the extensibility (tensile-strain capacity) of the concrete. The extensibility represents how much the concrete can be strained without exceeding its tensile strength and is the sum of creep plus elastic-strain capacity. The latter is largely related to the composition of the aggregate and can vary widely. Typically, some concretes of highly quartzitic gravels have a low strain capacity and a high modulus of elasticity. Concretes with a low strain capacity are much more sensitive to shrinkage due to drying (and to a decrease in temperature) and will be subjected to a greater amount of cracking.

A prime objective of crack-control procedures is to keep the concrete wet as long as possible so that the concrete will have time to develop more strength to resist cracking forces. As described in Chapter 3 and Carlson (1938), there are some cases where prolonged moist curing is not beneficial. The importance of moist curing will vary with the weather and the time of year. Cold concrete (below 10 C [50 F]) dries very slowly, provided the relative humidity is above 40%. At some depth, concrete loses moisture slowly. Where surface drying can be rapid, more care should be devoted to uninterrupted curing to improve surface strength. Cracking will be further reduced by creep if the surface is prevented from drying quickly at the end of the curing period. To accomplish slow drying, wet curing should remain for several days without wetting after the specified curing period (preferably 7 to 10 days) until the cover and the concrete under it appear to be dry. If these procedures are required to be used, they should be included in project contract documents.

8.3.3 *Plastic shrinkage*—Objectionable plastic shrinkage cracks commonly occur in the surfaces of floors and slabs when job conditions are so dry that moisture is removed from the concrete surface faster than it is replaced by bleedwater. These cracks occur before the start of curing and can occur either before or after final finishing. As the moisture is removed, the surface concrete contracts, resulting in tensile stresses in the weak, stiffening, plastic concrete that cause short random cracks in the surface. These cracks are usually wide at the surface but only a few millimeters (inches) in depth, although full-depth cracks can occur in pavements. The cracks range from a few millimeters (inches) to a few meters (feet) in length and are a few millimeters (inches) to 2/3 m (2 ft) apart.

Sometimes plastic shrinkage cracks appear early enough to be removed in later floating or troweling operations. When this occurs, it is advisable to postpone these operations as long as possible to get their maximum benefit without the recurrence of cracking.

In other cases, early floating can destroy the growing tension by reworking the surface mortar and prevent plastic cracking. At the first appearance of cracking while the concrete is still responsive, a vigorous effort should be made to close the cracks by tamping or beating with a float. If firmly closed, they are unlikely to reappear. They can, however, reappear if they are merely troweled over. In any event, curing should be started as soon as possible.

Plastic shrinkage cracking is most likely when environmental conditions, concrete temperature, and mixture proportions combine to cause a rapid loss of available surface moisture. These conditions can develop in either hot or cold weather when low humidity and high wind speed combine with a warm concrete surface temperature. Project specifications should stipulate that effective moisture-control precautions should be taken to prevent a serious loss of surface moisture under such conditions. Principal among these precautions are the use of fog (not spray) nozzles to maintain a sheen of moisture on the surface between the finishing operations. Plastic sheeting can be rolled on and off before and after floating, preferably exposing only the area being worked on at that time. Less effective but helpful precautions include sprayed monomolecular films that inhibit evaporation. Windbreaks are helpful; therefore, it is desirable to schedule flatwork after the walls are erected (ACI 305R; ACI 302.1R).

Plastic shrinkage cracking is associated with the rate loss of surface moisture relative to the replenishment rate of moisture by bleeding. As a result, the tolerable rate of moisture loss is significantly affected by the rate of bleeding. Further, the rate of moisture loss from the surface of concrete is driven by a combination of factors that include air temperature, concrete temperature, relative humidity of the air above the concrete, degree of saturation of the surface of the concrete (amount of bleedwater), and the wind velocity at the concrete surface.

The rate of evaporation can be greater during cold-weather-concreting conditions than during hot weather, particularly when the concrete has been heated, the air is cold and dry, and the wind speed is high. The reason for this is that the thermal driving force for surface evaporation is the difference between air and concrete temperature. When the concrete is warmer than the air, evaporation is favored; this is dependent on humidity and wind speed as well.

Other helpful practices that can counteract the excessive loss of surface moisture are using a well-dampened subgrade, cooling the aggregates by dampening and shading them, and using cold mixing water or chipped ice as mixing water to lower the fresh concrete temperature.

8.3.4 *Surface cooling*—Surface cooling will typically shrink the surface of unrestrained concrete about 10×10^{-6} C (5.5×10^{-6} F) as temperature decreases. This amounts to 9 mm in a 30 m length with a drop of 30 C (1/3 in. in 100 ft length with a drop of 50 F). The amount of shrinkage deformation is reduced by restraint and creep but tensile stresses result. The earlier the age and the slower the rate at which cooling or drying occur, the lower the tensile stresses will be. The influence of creep decreases the effective modulus of concrete loaded at early ages and allows more extensibility.

In ordinary concrete work, winter protection required for the development of adequate strength will prevent the most critical effects of cooling. Contraction joints for control of shrinkage cracking will also control cracking due to drops in surface temperature. In addition to Chapter 7 of this report, Chapters 4 and 5 of ACI 207.1R discuss temperature controls for mass concrete to minimize the early temperature differences between the interior and the exterior concrete. Primarily, these controls lower the interior temperature rise caused by the heat of hydration by using minimum cement content, pozzolans to replace a portion of the cement, water-reducing admixtures, air-entrainment, large aggregate sizes, low slump, and chipped ice for mixing water to maintain the temperature of the fresh concrete as close to 10 C (50 F) as possible. At no time should forms be removed to expose warm surfaces to low temperatures. As mentioned in Section 8.3.2, the extensibility or strain the concrete will withstand before tensile failure is a function of the aggregate and should be evaluated, especially on larger projects. What applies to one project will not necessarily apply to another.

8.4—Settlement

Settlement or subsidence cracks develop while concrete is in the plastic stage after the initial consolidation. Settlement cracks are the natural result of heavy solids settling in a liquid medium. Settlement cracks occur along rigidly supported elements, such as horizontal reinforcement, form ties, or embedments. Sometimes concrete will adhere to the forms. A crack will appear at these locations if the forms are hot at the top or are partially absorbent. Cracks often appear in horizontal construction joints and in bridge deck slabs over reinforcing or form ties with only a few inches of cover (about 75 to 125 mm [3 to 5 in.]). Settlement cracks in bridge decks can be reduced by increasing the concrete cover along with mixture proportioning that minimizes bleeding and settlement. Properly executed late revibration can be used to close settlement cracks and improve the quality and appearance of the concrete in the upper portion of such placements, even though settlement has taken place and slump has been lost (ACI 309R). Use of a low-slump concrete is also recommended to help prevent settlement cracks in bridge decks and slabs.

8.5—Construction

A great deal can be done during construction to minimize cracking. The individual project specifications should be specific with regard to actions that should be taken to cope with extremes of nature and to make enforceable requirements.

8.5.1 *Concrete aggregates*—The aggregates should be selected to make concrete of high-strain capacity, if reasonably available. It is important that fine and coarse aggregates be clean and free of unnecessary fine material, particularly clays. The fine aggregate should have a sand equivalent value greater than 80% and should be verified frequently (AASHTO T 176). The sand should have sufficient time in storage for the moisture content to stabilize at a level of less than 7% on an oven-dry basis.

8.5.2 *Shrinkage-compensating cement*—Shrinkage-compensating cement can be used to compensate for shrinkage in restrained elements or elements with the minimum shrinkage reinforcement required by ACI 318. The principal property of these cements is that the expansion induced in the concrete while setting and hardening is designed to offset the normal drying shrinkage. With correct usage (particularly with early and ample water curing required for maximum expansion), the distance between joints can sometimes be tripled without increasing the level of shrinkage cracking. Details on the types and the correct usage of shrinkage-compensating cements are given in Section 3.6 and ACI 223.

8.5.3 *Handling and batching*—Aggregates should be handled so as to avoid contamination, segregation, and breakage. Handling and batching is best done by finish screening and rinsing coarse aggregate into their various sizes and placing them in the appropriate bins at the batch plant. Every effort should be made to uniformly batch and mix the concrete so that there will be a minimum of variation in slump and workability, which invariably lead to demands for a greater margin of workability.

8.5.4 *Cold concrete*—In mass concrete structures, reducing water and cement contents to a practical minimum and using cold concrete will reduce temperature differentials that cause cracking. Less mixing water reduces drying shrinkage. In warm weather, cold concrete reduces slump loss, increases pumpability, and improves the response to vibration. Chipped ice can be substituted for all or a part of the batched mixing water. In cold weather, concrete is naturally cold, and every effort should be made to use it as cold as possible without inviting damage from freezing. It is pointless to try protecting surfaces, edges, and corners by placing needlessly warm concrete in cold weather. These vulnerable parts require protection by insulation or protective enclosures (ACI 306R).

8.5.5 *Revibration*—When done as late as the formed concrete will respond to the vibrator, revibration can eliminate cracks and checks where something rigidly fixed in the placement prevents a part of the concrete from settling with the rest of it. Settlement cracks are most apparent in the upper part of wall and column placements where revibration can be readily used. Deep revibration corrects cracks caused by differential settlement around blockout and window forms and where slabs and walls are placed monolithically (ACI 309R).

8.5.6 *Finishing*—Proper flatwork finishing can make a difference in many types of cracking (ACI 302.1R). Low-slump concrete should be used. More than a 75 mm (3 in.) slump is rarely necessary, except in hot weather, where both slump and moisture are lost quite rapidly. Finishing should not be done in the presence of surface water. Precautions (Section 8.3.3) should be taken to prevent plastic shrinkage. Any required marking and grooving should be carefully cut to the specified depth. Curing should be promptly conducted.

8.5.7 *Curing and protection*—Concrete should be brought to a level of adequate strength and protected from low temperatures and drying conditions that would otherwise cause cracking. The curing and protection should not be discontinued abruptly. If the new concrete is given a few days to gradually dry or cool, creep can reduce the possibility of cracking when the curing and protection are fully discontinued. Subsequent application of a curing compound after initial curing will slow drying out.

8.5.8 *Miscellaneous*—Some items normally covered in project specifications (or which should be covered where appropriate) require special attention during construction because of their potential effects on cracking.

- Reinforcement and embedments should be firmly positioned with the designated thickness of cover to prevent corrosion, expansion, and cracking;
- Concrete should not be placed against hot reinforcement or forms;
- Formwork support should be both strong and stiff enough to be free of early failures and distortion causing cracking;
- Subgrade and other supports should not settle unevenly, which may lead to cracking due to overstress in the structure;
- Calcium chloride should not be used if steel reinforcement is present. If acceleration in setting or strength gain is needed, additional cement, hot water, or a nonchloride accelerator should be used;
- Special care is needed in handling precast units to prevent overstress due to handling. Pickup points and rigging should be considered;
- Avoid the use of unvented salamanders in cold weather (ACI 306R) or gasoline-operated equipment where ventilation is not adequate. These will increase the danger of carbonation, causing shrinkage and surface cracking;
- Contraction joints should not be omitted and grooves should be of sufficient depth and well within the maximum permitted spacing. In hot weather or arid environments, contraction joints should be installed in the fresh concrete as inserts or saw-cut when the concrete is hard enough not to be torn or damaged by the blade;
- Reactive elements of coarse aggregate should be neutralized through the use of low-alkali cement, suitable pozzolans, or both. Certain cherts and other expansive aggregates can cause cracks and popouts. Project specifications should cover these aggregate properties and the cleanliness of these aggregates; and
- Correct amounts of entrained air should be used and monitored to prevent cracking due to freezing and thawing and exposure to deicing salts.

8.6—Specifications to minimize drying shrinkage

Actions during construction to obtain the lowest possible change in volume of the concrete should be supported by the specifications. Unless bids are taken on this basis, the contractors cannot be expected to provide other than ordinary materials, mixtures, and procedures. The following items should be carefully spelled out in the project specifications.

8.6.1 *Concrete materials*—Cement should be ASTM C 150 Types I, II, V, ASTM C 595 Type IS, or ASTM C 845 for expansive hydraulic cement. Use special care if Type III cement is used because of its high rate of heat generation and strength (and stiffness) development. Recommendation of ACI 224 should be followed when using shrinkage-compensating cement. ASTM C 595 Type IP can also be used. Aggregates favorable to low mixing-water content are well graded, well shaped (not elongated, flat, or splintery), with minimum amounts of clay, dirt, and excessive fines. Aggregate should consist of rock types that will produce low-shrinkage concrete. Utilization of pozzolans and chemical admixtures should be considered as appropriate. Calcium chloride and other admixtures containing a significant amount of chloride should be prohibited.

8.6.2 *Concrete mixtures*—For the least amount of shrinkage, the mixture proportions should incorporate those factors that contribute to the lowest paste content. Use the largest practical maximum size of aggregate (MSA). The lowest practical sand content, slump (water content), and mixture temperature should also be used. Where possible, limit the smaller-size fractions, that is, 4.75 mm to 9.5 or 20 mm (No. 4 to 3/8 or 3/4 in.), of coarse aggregate to half of the total, especially if the aggregate is crushed.

8.6.3 *Concrete handling and placing*—Equipment (chutes, belts, conveyors, pumps, hoppers, and bucket openings) should be capable of working effectively with lower slump, larger MSA concrete wherever it is appropriate and feasible to use it. The actions that tend to make a mixture more pumpable also tend to make the resulting concrete more prone to shrinkage and cracking. The use of pumping equipment capable of handling mixtures favorable to minimize cracking should be required.

Vibrators should be the largest and most powerful that can be operated in the placement. Upper lifts of formed concrete could be revibrated as late as the running vibrator will penetrate under its own weight.

8.6.4 *Finishing*—Finishing should follow the recommendations of ACI 302.1R to minimize or avoid surface cracking. It is particularly important that flatwork joint grooves have a depth of at least one-quarter of slab thickness.

8.6.5 *Forms*—Forms should have ample strength to sustain strong vibration of low-slump concretes. Exposing warm concrete surfaces to fast drying conditions or to low temperatures before curing should be avoided during form removal if drying and thermal shrinkage cracking is to be prevented (ACI 347R).

8.6.6 *Contraction joints*—Project drawings should include an adequate system of contraction joints to provide for shrinkage. Formed grooves should be constructed in both sides of parapet, retaining, and other walls at the appropriate depth and spacing. ACI 224.3R provides useful information on contraction joints.

8.6.7 *Curing and protection*—These procedures should ensure the presence of adequate moisture to sustain hydration in the surface concrete during the early ages of strength development. Rapid drying of the surfaces at the conclusion of the specified curing period should be avoided. Providing time for adjustment and relaxation of restraint-induced stresses minimizes cracking.

The best curing environment is to keep the concrete continuously wet during the curing period. Water curing should use a wet cover in contact with the concrete surfaces. At the end of the wet curing period, at least 7 days, the cover should be left in place until it and the concrete surface appear to be dry, especially in arid weather.

In less-arid areas and for interiors, the forms will provide adequate curing if exposed surfaces are protected from drying and left in contact with the concrete for at least 7 days. Thereafter, the forms should be left on with loosened bolts long enough to allow the concrete surfaces to dry gradually.

If ponding is used for curing in an arid climate, precautions such as a properly applied curing compound should be used when ponding is discontinued to avoid quick drying. Because drying is slow and prolonged, a properly applied curing compound provides good curing for flatwork placed on a well-wetted subgrade, and it provides adequate curing for formed surfaces. In an arid climate, curing compounds are not adequate for thinner structural sections. When used on formed surfaces, they should be applied when the surface is still damp but no longer wet (ACI 308).

8.7—Conclusion

It is the responsibility of the engineer to develop effective designs and clear, specific provisions in project specifications. To ensure both the owner's and the engineer's satisfaction with the results, the engineer should have the owner arrange for inspection by either the owner's personnel, the engineer, or a reliable professional inspection service that will ensure that the construction is performed on the same basis as it was bid. Without the full and firm intent to confirm the specified character and degree of performance, there is a likelihood that undesirable results will be obtained. Without firm inspection, a quality-control-assurance program, and a clear understanding of the project requirements by the contractor, it is likely that concrete will contain more water than it should. With less water content in the concrete, the finishing operations can be expedited and the curing process can start earlier. If properly applied, the procedures discussed in this chapter can have an effective influence on producing high-quality concrete with minimal cracking.

CHAPTER 9—REFERENCES

9.1—Referenced standards and reports

The standards and reports listed as follows were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Association of State Highway and Transportation Officials (AASHTO)

T 176	Plastic Fines In Graded Aggregate and Soils by Use of the Sand Equivalent Test

American Concrete Institute (ACI)

116R	Cement and Concrete Terminology
201.1	Guide for Making a Condition Survey of Concrete in Service
201.2R	Guide to Durable Concrete
207.1R	Mass Concrete
207.2R	Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete
207.4R	Cooling and Insulating Systems for Mass Concrete
207.5R	Roller Compacted Mass Concrete
209R	Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures

212.3R	Chemical Admixtures for Concrete
221R	Guide for Use of Normal Weight and Heavyweight Aggregates in Concrete
221.1R	State-of-the-Art Report on Alkali-Aggregate Reactivity
222R	Corrosion of Metals in Concrete
223	Standard Practice for the Use of Shrinkage-Compensating Concrete
224.3R	Joints in Concrete Construction
302.1R	Guide for Concrete Floor and Slab Construction
304R	Guide for Measuring, Mixing, Transporting, and Placing Concrete
305R	Hot Weather Concreting
306R	Cold Weather Concreting
308	Standard Practice for Curing Concrete
309R	Guide for Consolidation of Concrete
313	Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials
318/318R	Building Code Requirements for Structural Concrete and Commentary
340R	ACI Design Handbook (SP-17(97))
347R	Guide to Formwork for Concrete
350	Code Requirements For Environmental Engineering Concrete Structures
446.1R	Fracture Mechanics of Concrete: Concepts, Models, and Determination of Material Properties
504R	Guide to Sealing Joints for Concrete Structures
544.2R	Measurement of Properties of Fiber-Reinforced Concrete
544.3R	Guide for Specifying, Proportioning, Mixing, Placing, and Finishing Steel Fiber Reinforced Concrete
544.4R	Design Considerations for Steel Fiber Reinforced Concrete
548R	Polymers in Concrete
548.1R	Guide for the Use of Polymers in Concrete
SP-38	Klein Symposium on Expansive Cement Concretes
SP-40	Polymers in Concrete
SP-58	Polymers in Concrete
SP-64	Cedric Wilson Symposium on Expansive Cement

American Society for Testing and Materials (ASTM)

C 150	Specification for Portland Cement
C 845	Specification for Expansive Hydraulic Cement
E 399	Test Method for Plain-Strain Fracture Toughness of Metallic Materials

European Committee for Concrete-International Federation of Prestressed Concrete (CEB-FIP)

CEB-FIP	European Model Code for Concrete Structures

The above publications may be obtained from the following organizations:

American Association of State Highway and Transportation Officials
444 N. Capitol Street NW
Suite 225
Washington, DC 20001

American Concrete Institute
PO Box 9094
Farmington Hills, MI 48333-9094

ASTM
100 Barr Harbor Drive
West Conshohocken, PA 19428

CEB-FIP
c/o British Cement Association
Century House
Telford Avenue
Crowthorne, Berkshire RG45 6YS
United Kingdom

9.2—Cited references

Aalami, B. O., and Barth, F. G., 1989, "Restraint Cracks and their Mitigation in Unbonded Post-Tensioned Building Structures," *Cracking in Prestressed Concrete Structures*, SP-113, G. T. Halvorsen and N. H. Burns, eds., American Concrete Institute, Farmington Hills, Mich., pp. 157-202.

Abeles, P. W., 1956, "Cracks in Prestressed Concrete Beams," *Proceedings*, 5th IABSE Congress, International Association for Bridge and Structural Engineering, Zurich, pp. 707-720.

Abeles, P. W.; Brown, E. L., II; and Morrow, J. W., 1968, "Development and Distribution of Cracks in Rectangular Prestressed Beams during Static and Fatigue Loading," *Journal*, Prestressed Concrete Institute, V. 13, No. 5, Oct., pp. 36-51.

ACI Committee 223, 1970, "Expansive Cement Concretes: Present State of Knowledge," ACI JOURNAL, *Proceedings* V. 67, No. 8, Aug., pp. 583-610.

ACI Committee 224, 1971, "Causes, Mechanism, and Control of Cracking in Concrete," *ACI Bibliography* No. 9, American Concrete Institute, Farmington Hills, Mich., 92 pp.

ACI Committee 224, 1972, "Control of Cracking in Concrete Structures," ACI JOURNAL, *Proceedings* V. 69, No. 12, Dec., pp. 717-753.

ACI Committee 224, 1980, "Control of Cracking in Concrete Structures," *Concrete International: Design and Construction*, V. 2, No. 10, Oct., pp. 35-76.

ACI Committee 318, 1999, "Discussion of a Report Published in the November 1998 Issue of *Concrete International*, pp. S1-S90," *Concrete International*, V. 21, No. 5., May, pp. 318.1-318.49.

Atimtay, E., and Ferguson, P. M., 1973, "Early Chloride Corrosion of Reinforced Concrete—A Test Report," ACI JOURNAL, *Proceedings* V. 70, No. 9, Sept., pp. 606-611.

Attiogbe, E. K., and Darwin, D., 1987, "Submicrocracking in Cement Paste and Mortar," *ACI Materials Journal*, V. 84, No. 6, Nov.-Dec., pp. 491-500.

Attiogbe, E. K., and Darwin, D., 1988, “Strain Due to Submicrocracking in Cement Paste and Mortar,” *ACI Materials Journal*, V. 85, No. 1, Jan.-Feb., pp. 3-11.

Bascoul, A.; Kharchi, F.; and Maso, J. C., 1987, “Concerning the Measurement of the Fracture Energy of a Micro-Concrete According to Crack Growth in Three Point Bending Tests of Notched Beams,” *Proceedings*, SEM-RILEM International Conference on Fracture of Concrete and Rock, S. P. Shah and S. E. Swartz, eds., Houston, Tex.

Bate, S. C. C., 1958, “Relative Merits of Plain and Deformed Wires in Prestressed Concrete Beams under Static and Repeated Loading,” *Proceedings*, Institution of Civil Engineers (London), V. 10, Aug., pp. 473-502.

Bate, S. C. C., 1963, “A Comparison Between Prestressed Concrete and Reinforced Concrete Beams under Repeated Loading,” *Proceedings*, Institution of Civil Engineers (London), V. 24, Mar., pp. 331-358.

Bažant, Z. P., 1979, “Material Behavior under Various Types of Loading,” *High Strength Concrete*, S. P. Shah, ed., University of Illinois at Chicago, pp. 79-92.

Bažant, Z. P.; Kim, J. K; and Pfeiffer, P., 1986, “Determination of Non-linear Fracture Parameters from Size Effect Tests,” *Journal of Structural Engineering*, ASCE, V. 112, No. 2, Feb., pp. 289-307.

Bažant, Z. P., and Oh, B. H., 1983, “Crack Band Theory for Fracture of Concrete,” *Materials and Structures*, RILEM, V. 16, No. 93, pp. 155-177.

Bažant, Z. P., and Sener, S., 1988, “Size Effect in Pullout Tests,” *ACI Materials Journal*, V. 85, No. 5, May, pp. 347-351.

Bažant, Z. P.; Sener, S.; and Pratt, P. C., 1988, “Size Effect Tests of Torsional Failure of Plain and Reinforced Concrete Beams,” *Materials and Structures*, V. 21, pp. 425-430.

Bažant, Z. P., and Sun, H. H., 1987, “Size Effect in Diagonal Shear Failure: Influence of Aggregate Size and Stirrups,” *ACI Materials Journal*, V. 84, No. 4, Apr., pp. 259-272.

Beckett, D., and Alexandrou, A., 1997, *Introduction to Eurocode 2— Design of Concrete Structures*, E&FN Spon, 191 pp.

Beeby, A. W., 1978a, “Concrete in the Oceans—Cracking and Corrosion,” *Technical Report* No. 1, Cement and Concrete Association, London.

Beeby, A. W., 1978b, “Corrosion of Reinforcing Steel in Concrete and Its Relation to Cracking,” *The Structural Engineer*, London, V. 56A, No. 3, Mar., pp. 77-81.

Beeby, A. W., 1983, “Cracking Cover at Corrosion of Reinforcement,” *Concrete International: Design and Construction*, V. 5, No. 2, Feb., pp. 35-40.

Bennett, E. W., and Chandrasekhar, C. S., 1971, “Calculation of the Width of Cracks in Class 3 Prestressed Beams,” *Proceedings*, Institution of Civil Engineers, London, V. 49, July, pp. 333-346.

Bennett, E. W., and Dave, N. J., 1969, “Test Performances and Design of Concrete Beams with Limited Prestress,” *The Structural Engineer*, London, V. 47 No. 12, Dec., pp. 487-496.

Bishara, A. G., 1979, “Latex Modified Concrete Bridge Deck Overlays—Field Performance Analysis,” *Report* No. FHWA/OH/79/004, Federal Highway Administration, Washington D.C., Oct., 97 pp.

Bishara, A. G., and Tantayanondkul, P., 1974, “Use of Latex in Concrete Bridge Decks,” *Report* No. EES 435, (ODOT-12-74) Ohio Department of Transportation, Ohio State University, Columbus, Ohio.

Brendel, G., and Ruhle, H., 1964, “Tests on Reinforced Concrete Beams Under Long-Term Loads (Dauerstandvrsuche mit Stahlbetonbalken),” *Proceedings*, Seventh IABSE Congress (Rio de Janeiro, 1964), International Association of Bridge and Structural Engineering, Zurich, Switzerland, pp. 916-922.

Broms, B. B., 1965a, “Technique for Investigation of Internal Cracks in Reinforced Concrete Members,” ACI JOURNAL, *Proceedings* V. 62, No. 1, Jan., pp. 35-44.

Broms, B. B., 1965b, “Crack Width and Crack Spacing in Reinforced Concrete Members,” ACI JOURNAL, *Proceedings* V. 62, No. 10, Oct., pp. 1237-1256.

Brookhaven National Laboratory, 1968, “Concrete Polymer Materials,” *BNL Report 50134 (T-509)*.

Brooks, J. J., and Neville, A. M., 1977, “A Comparison of Creep, Elasticity and Strength of Concrete in Tension and in Compression,” *Magazine of Concrete Research*, London, V. 29, No. 100, Sept., pp. 131-141.

Bryant, A. H., and Vadhanavikkit, C., 1987, “Creep Shrinkage—Size, and Age at Loading Effects,” *ACI Materials Journal*, V. 84, No. 2, Mar.-Apr., pp. 119-123.

Burgess, J. A.; Breen, J. E.; and Poston, R. W., 1989, “Anchorage Zone Cracking of Post-Tensioned Bridge Decks with Closely Spaced Anchors,” *Cracking in Prestressed Concrete Structures*, SP-113, G. T. Halvorsen and N. H. Burns, eds., American Concrete Institute, Farmington Hills, Mich., pp. 79-108.

Buyukozturk, O., 1970, “Stress-Strain Response and Fracture of a Model of Concrete in Biaxial Loading,” PhD thesis, Cornell University, Ithaca, N.Y., June.

California Department of Transportation, 1963, Report: Effect of Curing on Shrinkage of Concrete.

Carino, N. J., 1977, “Effects of Polymer Impregnation on Mortar-Aggregate Bond Strength,” *Cement and Concrete Research*, V. 7, No. 4, July, pp. 439-447.

Carlson, R. W., 1938, “Drying Shrinkage of Concrete as Affected by Many Factors,” *Proceedings*, ASTM, V. 38, Part II, pp. 419-437.

Carrasquillo, R. L.; Slate, F. O.; and Nilson, A. H., 1981, “Microcracking and Behavior of High Strength Concrete Subject to Short-Term Loading,” ACI JOURNAL, *Proceedings* V. 78, No. 3, May-June, pp. 179-186.

Castro-Montero, S.; Shah S. P., and Miller, R. A., 1990, “Study of Fracture Process in Mortar with Laser Holographic Measurements,” *Proceedings of the NATO Advanced Research Workshop on Toughening Mechanisms in Quasi-Brittle Materials*, Evanston, Ill.

Catalano, D., and Ingraffea, A. R., 1982, “Concrete Fracture: A Linear Elastic Fracture Approach,” *Department of Structural Engineering Report*, School of Civil and Environmental Engineering, Cornell University, Ithaca, N.Y., 161 pp.

Clear, K. C., 1974, “Time to Corrosion of Reinforcing Steel in Concrete Slabs,” *Transportation Research Record* No. 500, Transportation Research Board, pp. 16-24.

Cook, D. J., and Chindaprasirt, P., 1980, "Influence of Loading History Upon the Compressive Properties of Concrete," *Magazine of Concrete Research*, London, V. 32, No. 111, June, pp. 89-100.

Darwin, D., 1978, Discussion of "Bond Failure and Inelastic Response of Concrete," by Testa and Stubbs, *Journal of the Engineering Mechanics Division*, ASCE, V. 104, No. EM2, Apr., pp. 507-509.

Darwin, D., and Slate, F. O., 1970, "Effect of Paste-Aggregate Bond Strength on Behavior of Concrete," *Journal of Materials*, V. 5, No. 1, Mar., pp. 86-98.

Davis, H. E., 1940, "Autogenous Volume Changes of Concrete," *Proceedings*, ASTM, V. 40, pp. 1103-1110.

Derucher, K. N., 1978, "Application of the Scanning Electron Microscope to Fracture Studies of Concrete," *Building and Environment*, V. 13, No. 2, 1978, pp. 135-141.

Dolan, C. W., 1990, "Developments in Non-Metallic Prestressing Tendons," *PCI Journal*, V. 35, No. 5, Sept.-Oct., pp. 80-88.

Euro EC2 Code, 1997, *Design of Concrete Structures*, Paris.

Feldman, R. F., and Swenson, E. G., 1975, "Volume Change on First Drying of Hydrate Portland Cement with and without Admixture," *Cement and Concrete Research*, V. 5, No. 1, pp. 25-35.

Frantz, G. C., and Breen, J. E., 1980a, "Cracking on Side Faces of Large Reinforced Concrete Beams," ACI JOURNAL, *Proceedings* V. 77, No. 5, Sept.-Oct., pp. 307-313.

Frantz, G. C., and Breen, J. E., 1980b, "Design Proposal for Side Face Crack Control Reinforcement for Large Reinforced Concrete Beams," *Concrete International: Design and Construction*, V. 2, No. 10, Oct., pp. 29-34.

Frosch, R. J., 1999, "Another Look at Cracking and Crack Control in Reinforced Concrete," *ACI Structural Journal*, V. 96, No. 3, May-June, pp. 437-442.

Gergely, P., 1969, "Anchorage Systems in Prestressed Concrete Pressure Vessels' Anchorage Zone Problems," *Report* No. ORNL-TM-2378, Oak Ridge National Laboratory, U.S. Atomic Energy Commission, Oak Ridge, Tenn., pp. 1-49.

Gergely, P., and Lutz, L. A., 1968, "Maximum Crack Width in Reinforced Concrete Flexural Members," *Causes, Mechanism, and Control of Cracking in Concrete*, SP-20, American Concrete Institute, Farmington Hills, Mich., pp. 87-117.

Gergely, P., and Sozen, M. A., 1967, "Design of Anchorage Zone Reinforcement in Prestressed Concrete Beams," *PCI Journal*, V. 13, No. 2, Mar.-Apr., pp. 63-75.

Gettu, R.; Bažant, Z. P.; and Karr, M. E.; 1990, "Brittleness of High Strength Concrete," *Serviceability and Durability of Construction Materials*, B. A. Suprenant, ed., ASCE, New York, V. 2, pp. 976-985.

Ghosh, R. S., and Malhotra, V., 1979, "Use of Superplasticizers as Water Reducers," *Cement Concrete and Aggregates*, ASTM, V. 1, No. 2, pp. 56-63.

Gjørv, O. E.; Sorenson, S. I.; and Arnesen, A., 1977, "Notch Sensitivity and Fracture Toughness of Concrete," *Cement and Concrete Research*, V. 7, pp. 333-344.

Gopalaratnam, V. S., and Shah, S. P., 1986, "Softening Response of Plain Concrete in Direct Tension," ACI JOURNAL, *Proceedings* V. 82, No. 3, May-June, pp. 310-323.

Griffith, A. A., 1920, "The Phenomena of Rupture and Flow of Solids," *Transactions*, Royal Society of London, No. 221A, pp. 163-198.

Halvorsen, G. T., 1987, "Code Requirements for Crack Control," *Concrete and Concrete Construction*, SP-104, American Concrete Institute, Farmington Hills, Mich., pp. 275-322.

Hansen, W., and Almudaiheem, J. A., 1987, "Ultimate Drying Shrinkage of Concrete—Influence of Major Parameters," *ACI Materials Journal*, V. 84, No. 3, May-June, pp. 39-46.

Harajli, M. H., and Naaman, A. E., 1989, "Cracking in Partially Prestressed Beams Under Static and Fatigue Loading," *Cracking in Prestressed Concrete Structures*, SP-113, G. T. Halvorsen and N. H. Burns, eds., American Concrete Institute, Farmington Hills, Mich., pp. 29-56.

Higgins, D. D., and Bailey, J. E., 1976, "Fracture Measurements on Cement Paste," *Journal of Materials Science*, V. 11, pp. 1995-2003.

Hillemeier, B., and Hilsdorf, H. K., 1977, "Fracture Mechanics Studies of Concrete Compounds," *Cement and Concrete Research*, V. 7, No. 5, Sept., pp. 523-536.

Hillerborg, A.; Modeer, M; and Petersson, P. E., 1976, "Analysis of Crack Formation and Crack Growth by Means of Fracture Mechanics and Finite Elements," *Cement and Concrete Research*, V. 6, pp. 773-782.

Hoff, G. C., 1987, "Durability of Fiber Reinforced Concrete in a Severe Marine Environment," *Concrete Durability*, SP-100, American Concrete Institute, Farmington Hills, Mich., pp. 997-1041.

Holmberg, A., 1973, "Crack Width Prediction and Minimum Reinforcement for Crack Control," Dansk Selskab for Byaningsstatik, Copenhagen, V. 44, No. 2, June, pp. 41-50.

Holmberg, A., and Lindgren, S., 1970, "Crack Spacing and Crack Width Due to Normal Force or Bending Moment," *Document D2*, National Swedish Council for Building Research, Stockholm, 57 pp.

Houk, I. E., Jr.; Borge, O. E.; and Houghton, D. L., 1969, "Studies of Autogenous Volume Change in Concrete for Dworshak Dam," ACI JOURNAL, *Proceedings* V. 66, No. 7, pp. 560-568.

Houk, I. E., Jr.; Paxton, J. A.; and Houghton, D. L., 1970, "Prediction of Thermal Stress and Strain Capacity of Concrete by Tests on Small Beams," ACI JOURNAL, *Proceedings* V. 67, No. 3, Mar., pp. 253-261.

Houghton, D. L., 1969, "Concrete Volume Change for Dworshak Dam," *Proceedings*, ASCE, V. 95, No. 2, Oct., pp. 153-166.

Houghton, D. L., 1976, "Determining Tensile Strain Capacity of Mass Concrete," ACI JOURNAL, *Proceedings* V. 73, No. 12, Dec., pp. 691-700.

Hsu, T. T. C., 1963, "Mathematical Analysis of Shrinkage Stresses in a Model of Hardened Concrete," ACI JOURNAL, *Proceedings* V. 60, No. 3, Mar., pp. 371-390.

Hsu, T. T. C.; Slate, F. O.; Sturman, G. M; and Winter, G., 1963, "Microcracking of Plain Concrete and the Shape of the Stress Strain Curve," ACI JOURNAL, *Proceedings* V. 60, No. 2, Feb., pp 209-224.

Hutton, S. G., and Loov, R. E., 1966, "Flexural Behavior of Prestressed, Partially Prestressed, and Reinforced Concrete Beams," ACI JOURNAL, *Proceedings* V. 63, No. 12, Dec., pp. 1401-1410.

Illston, J. M., and Stevens, R. F., 1972, "Long-Term Cracking in Reinforced Concrete Beams," *Proceedings*, Institution of Civil Engineers, London, Part 2, V. 53, Dec., pp. 445-459.

Jenq, Y. S., and Shah, S. P., 1987, "Mixed Mode Fracture Parameters of Concrete," *Proceedings*, SEM-RILEM International Conference on Fracture of Concrete and Rock, S. P. Shah and S. E. Swartz, eds., Houston, Tex., pp. 359-369.

John, R., and Shah, S. P., 1986, "Fracture of Concrete Subject to Impact Loading," *Journal of Cement Concrete and Aggregates*, ASTM, V. 8, No. 1, pp. 24-32.

Kaplan, F. M., 1961, "Crack Propagation and the Fracture of Concrete," ACI JOURNAL, *Proceedings* V. 58, No. 5, Nov., pp. 591-610.

Karsan, I. D., and Jirsa, J. O., 1969, "Behavior of Concrete under Compressive Loadings," *Journal of the Structural Division*, ASCE, V. 95, ST 12, Dec., pp. 2543-2563.

Kesler, C. E.; Naus, D. J.; and Lott, J. L.; 1972, "Fracture Mechanics, Its Applicability to Concrete," *Proceedings*, International Conference on Mechanical Behavior of Materials, Kyoto, Aug. 15-20, 1971, The Society of Materials Science, Japan, V. IV, pp. 113-124.

Krishna, R. N.; Basavarajuiah, B. S.; and Ahamed K. U. C., 1973, "Flexural Behavior of Pretensioned Concrete Beams with Limited Prestress," *Building Science*, V. 8, No. 2, June, pp. 179-185.

Lee, S. L.; Mansur, M. S.; Tan, K. H.; and Kasiraju, K., 1987, "Cracking Behavior of Concrete Tension Members Reinforced with Welded Wire Fabric," *ACI Structural Journal*, V. 84, No. 6, Nov.-Dec., pp. 481-491.

Leibengood, L. D.; Darwin, D.; and Dodds, R. H.; 1986, "Parameters Affecting FE Analysis of Concrete Structures," *Journal of the Structural Division*, ASCE, V. 112, No. 2, Feb., pp. 326-341.

Leonhardt, F., 1977, "Crack Control in Concrete Structures," *IABSE Surveys No. S4/77*, International Association for Bridge and Structural Engineering, Zurich, 26 pp.

Leonhardt, F., 1988, "Crack and Crack Control in Concrete Structures," *PCI Journal*, V. 22, No. 4, pp. 124-145.

Linsbauer, H. N.; Ingraffea, A. R.; Rossmanith, H. P.; and Wawrzynek, P. A., 1989a, "Simulation of Cracking in Large Arch Dam: Part I," *Journal of the Structural Division*, ASCE, V. 115, No. 7, Feb., pp. 1599-1615.

Linsbauer, H. N.; Ingraffea, A. R.; Rossmanith, H. P.; and Wawrzynek, P. A., 1989b, "Simulation of Cracking in Large Arch Dam: Part II," *Journal of the Structural Division*, ASCE, V. 115, No. 7, Feb., pp. 1616-1630.

Liu, T. C.; Campbell, R. L.; and Bombich, A. A.; 1979, "Verification of Temperature and Thermal Stress Analysis Computer Programs for Mass Concrete Structures," *Miscellaneous Paper* No. SL-79-7, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Apr.

Liu, T. C., and McDonald, J. E., 1978, "Prediction of Tensile Strain Capacity of Mass Concrete," ACI JOURNAL, *Proceedings* V. 75, No. 4, May, pp. 192-197.

Lorman, W. R., 1940, "The Theory of Concrete Creep," *Proceedings* ASTM, V. 40, pp. 1082-1102.

Lutz, L. A.; Sharma, N. K.; and Gergely, P., 1968, "Increase in Crack Width in Reinforced Concrete Beams under Sustained Loading," ACI JOURNAL, *Proceedings* V. 64, No. 9, Sept., pp. 538-546.

Maher, A., and Darwin, D., 1976, "A Finite Element Model to Study the Microscopic Behavior of Plain Concrete," *CRINC Report* SL-76-02, University of Kansas Center for Research, Inc., Lawrence, Kans., Nov., 83 pp.

Maher, A., and Darwin, D., 1977, "Microscopic Finite Element Model of Concrete," *Proceedings*, First International Conference on Mathematical Modeling (St. Louis, Aug. 29-Sept. 1, 1977), University of Missouri-Rolla, V. III, pp. 1705-1714.

Maher, A., and Darwin, D., 1982, "Mortar Constituent of Concrete in Compression," ACI JOURNAL, *Proceedings* V. 79, No. 2, Mar.-Apr., pp. 100-109.

Maji, A. K., and Shah, S. P., 1987, "Process Zone and Acoustic Emission Measurements in Concrete," *Experimental Mechanics*, SEM, Paper No. 3609.

Marshall, W. T., and Mattock, A. H., 1962, "Control of Horizontal Cracking in the Ends of Pretensioned Concrete Girders," *Journal*, Prestressed Concrete Institute, V. 7, No. 5, Aug.-Oct., pp. 56-74.

Martin, J. L.; Darwin, D.; and Terry, R. E., 1991, "Cement Paste, Mortar and Concrete in Compression," *SM Report* No. 31, University of Kansas Center for Research, Lawrence, Kans., 161 pp.

Mather, B., 1957, "Factors Affecting Durability of Concrete in Coastal Structures," *Technical Memorandum* No. 96, Beach Erosion Board, Washington D.C., June.

Mather, B., 1968, "Cracking Induced by Environmental Effects," *Causes, Mechanism, and Control of Cracking in Concrete*, SP-20, American Concrete Institute, Farmington Hills, Mich., pp. 67-72.

McCoy, E. E., Jr.; Thorton, H. T.; and Allgood, J. K., 1964, "Concrete Laboratory Studies, Dworshak (Bruce's Eddy) Dam, North Fork Clearwater River near Orofino, Idaho: Creek Tests," *Miscellaneous Paper* No. 6-613, Report 2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Dec.

McDonald, J. E.; Bombich, A. A.; and Sullivan, B. R., 1972, "Ultimate Strain Capacity and Temperature Rise Studies, Trumbull Pond Dam," *Miscellaneous Paper* C-72-20, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Aug.

Meier, S. W., and Gergely, P., 1981, "Flexural Crack Width in Prestressed Concrete Beams," *Journal of the Structural Division*, ASCE, V. 107, No. ST2, pp. 429-433.

Meyers, B. L.; Slate, F. O.; and Winter, G., 1969, "Relationship between Time Dependent Deformation and Microcracking of Plain Concrete," ACI JOURNAL, *Proceedings* V. 66, No. 1, Jan., pp. 60-68.

Nallathambi, P., and Karihaloo, B. L., 1986, "Influence of Slow Crack Growth on the Fracture Toughness of Plain Concrete," *Fracture Toughness and Fracture Energy*, F. H. Wittmann, ed., Elsevier Science Publishers, The Netherlands, pp. 271-280.

Naus, D. J., and Lott, J. L., 1969, "Fracture Toughness of Portland Cement Concrete," ACI JOURNAL, *Proceedings* V. 66, No. 6, June, pp. 481-489.

Nawy, E. G., 1968, "Crack Control in Reinforced Concrete Structures," ACI JOURNAL, *Proceedings* V. 65, No. 10, Oct., pp. 825-836.

Nawy, E. G., 1972, "Crack Control through Reinforcement Distribution in Two-Way Acting Slabs and Plates," ACI JOURNAL, *Proceedings* V. 69, No. 4, Apr., pp. 217-219.

Nawy, E. G., and Blair, K. W., 1971, "Further Studies on Flexural Crack Control in Structural Slab Systems," *Cracking, Deflection, and Ultimate Load of Concrete Slab Systems*, SP-30, American Concrete Institute, Farmington Hills, Mich., pp. 1-41.

Nawy, E. G., and Huang, P. T., 1977, "Crack and Deflection Control of Pretensioned Prestressed Beams," *PCI Journal*, V. 22, No. 3, May-June, pp. 30-47.

Nawy, E. G., and Neuwerth, G. E., 1977, "Behavior of Concrete Slabs, Plates and Beams with Fiber Glass as Main Reinforcement," *Proceedings*, ASCE, V. 103, ST2, Feb., pp. 421-440.

Nawy, E. G.; Ukadike, M. M.; and Sauer, J. A.; 1977, "High Strength Field Polymer Modified Concretes," *Proceedings*, ASCE, V. 103, ST12, Dec., pp. 2307-2322.

Nawy, E. G., 1989a, "Flexural Cracking Behavior of Partially Prestressed Pretensioned and Post-Tensioned Beams—State of the Art," *Cracking in Prestressed Concrete Structures*, SP-113, G. T. Halvorsen and N. H. Burns, eds., American Concrete Institute, Farmington Hills, Mich., pp. 1-28.

Nawy, E. G., 1989b, "Stresses, Strains, and Bursting Cracks in Anchorage Zones of Post-Tensioned Beams," *Cracking in Prestressed Concrete Structures,* SP-113, G. T. Halvorsen and N. H. Burns, eds., American Concrete Institute, Farmington Hills, Mich., pp. 109-132.

Nawy, E. G., 2001, *Fundamentals of High-Performance Concrete*, John Wiley and Sons, New York, 446 pp.

Nepper-Christensen, P., and Nielsen, T. P. H., 1969, "Modal Determination of the Effect of Bond between Coarse Aggregate and Mortar on the Compressive Strength of Concrete," ACI JOURNAL, *Proceedings* V. 66, No. 1, Jan., pp. 69-72.

Neville, A. M., and Hirst, G. A., 1978, "Mechanism of Cyclic Creep of Concrete," *Douglas McHenry Symposium on Concrete and Concrete Structures*, SP-55, B. Bresler, ed., American Concrete Institute, Farmington Hills, Mich., pp. 83-101.

Neville, A. M., and Brooks, J. J., 1985, *Concrete Technology*, John Wiley & Sons, Inc., New York.

Ngab, A. S.; Slate, F. O.; and Nilson, A. H., 1981, "Microcracking and Time-Dependent Strains in High-Strength Concrete," ACI JOURNAL, *Proceedings* V. 78, No. 4, July-Aug., pp. 262-268.

Nilson, A. H., 1987, *Design of Prestressed Concrete*, 2nd Edition, John Wiley and Sons, New York, 592 pp.

Odman, S. T. A., 1968, "Effects of Variations in Volume, Surface Area Exposed to Drying, and Composition of Concrete on Shrinkage," RILEM/CEMBUREAU, *International Colloquium on the Shrinkage of Hydraulic Concretes*, Madrid, 20 pp.

Oh, B. H., and Kang, Y. J., 1987, "New Formulas for Maximum Crack Width and Crack Spacing in Reinforced Concrete Flexural Membranes," *ACI Structural Journal*, V. 84, No. 2, Mar.-Apr., pp. 103-112.

Perry, C., and Gillott, J. E., 1977, "The Influence of Mortar-Aggregate Bond Strength on the Behavior of Concrete in Uniaxial Compression," *Cement and Concrete Research*, V. 7, No. 5, Sept., pp. 553-564.

Pickett, G., 1956, "Effect of Aggregate on Shrinkage of Concrete and Hypothesis Concerning Shrinkage," ACI JOURNAL, *Proceedings* V. 52, No. 5, Jan., pp. 581-580.

Polivka, R. M., and Wilson, E. L., 1976, "Finite Element Analysis of Nonlinear Heat Transfer Problems," *Report* No. UC SESM 76-2, Department of Civil Engineering, University of California, Berkeley, Calif., June.

Poston, R. W.; Carrasquillo, R. L.; and Breen, J. E., 1987, "Durability of Post-Tensioned Bridge Decks," *ACI Materials Journal*, V. 84, No. 4, July-Aug., pp. 315-326.

Poston, R. W., and Schupack, M., 1990, "Durability of a Precast Prestressed Concrete Conveyor Bridge Structure at a Salt Mine after 17 Years of Service," *Paul Klieger Symposium on Performance of Concrete,* SP-122, D. Whiting, ed., American Concrete Institute, Farmington Hills, Mich., pp. 363-384.

Rao, A.; Gandotra, K.; and Ramaswamy, G. S., 1976, "Flexural Tests on Beams Prestressed to Different Degrees of Prestress," *Journal*, Institution of Engineers (Calcutta), V. 56, May.

Refai, T. M. E., and Swartz, S. E., 1987, "Influence of Size Effects on Opening Mode Fracture Parameters for Precracked Concrete Beams in Bending," *Proceedings*, SEM-RILEM International Conference on Fracture of Concrete and Rock, S. P. Shah and S. E. Swartz, eds., Houston, Tex.

Rehm, G., and Eligehausen, R., 1977, "Lapped Splices of Deformed Bars Under Repeated Loadings (Ubergreifungsstoβe von Rippenstahlen unter nicht ruhender Belastung)," *Beton Und Stahlbetonbau* (Berlin), No. 7, pp. 170-174.

Roshore, E. C., 1967, "Field Exposure Tests of Reinforced Concrete Beams," ACI JOURNAL, *Proceedings* V. 64, No. 5, May, pp. 253-257.

Sanders, D. H.; Breen, J. E.; and Duncan III, R. R., 1987, "Strength and Behavior of Closely Spaced Post-Tensioned Monostrand Anchorages," *PTI Report*, Oct., 49 pp.

Schlaich, J.; Schafer, K.; and Jennewein, M.; 1987, "Toward a Consistent Design of Structural Concrete," *PCI Journal*, V. 32, No. 3, May-June, pp. 74-150.

Schrader, E. K. and Munch, A. V., 1976, "Deck Slab Repaired by Fibrous Concrete Overlay," *Proceedings*, ASCE, V. 102, No. CO1, Mar., pp. 179-196.

Schrader, E. K.; Fowler, D. W.; Kaden, R. A.; and Stebbins, R. J., 1978, "Polymer Impregnation Used in Concrete Repairs on Cavitation/Erosion Damage," *Polymers in Concrete,* SP-58, American Concrete Institute, Farmington Hills, Mich., pp. 225-248.

Schupack, M., 1985, "Durability of SFRC Exposed to Severe Environments," *Steel Fiber Concrete*, S. P. Shah and A. Skarendahl, eds., Elsevier Applied Science Publishers, London, pp. 479-496

Shah, S. P., and Batson, G. B., eds., 1987, *Fiber Reinforced Concrete Properties and Applications*, SP-105, American Concrete Institute, Farmington Hills, Mich., 600 pp.

Shah, S. P., and Chandra, S., 1970, "Fracture of Concrete Subjected to Cyclic and Sustained Loading," ACI JOURNAL, *Proceedings* V. 67, No. 10, Oct.

Shah, S. P.; Ludirja, D.; and Daniel, J. I., 1987, "Toughness of Glass Fiber Reinforced Concrete Panels Subjected to Accelerated Aging," *PCI Journal*, V. 32, No. 5, Sept.-Oct., pp. 82-99.

Shah, S. P., and McGarry, F. J., 1971, "Griffith Fracture Criteria and Concrete," *Journal of the Engineering Mechanics Division*, *Proceedings*, ASCE, V. 47, No. EM 6, Dec., pp. 1663-1676.

Shah, S. P., and Naaman, A. E., 1976, "Mechanical Properties of Glass and Steel Fiber Reinforced Mortar," ACI JOURNAL, *Proceedings*, Jan., pp. 50-53.

Shah, S. P., and Skarendahl, A., 1986, *Steel Fiber Concrete,* Elsevier Applied Science Publishers, London, 526 pp.

Shah, S. P., and Slate, F. O., 1968, "Internal Microcracking, Mortar-Aggregate Bond, and the Stress-Strain Curve of Concrete," *Proceedings*, International Conference on the Structure of Concrete (London, Sept. 1965), Cement and Concrete Association, London, pp. 82-92.

Shah, S. P., and Winter, G., 1966, "Inelastic Behavior and Fracture of Concrete," ACI JOURNAL, *Proceedings* V. 63, No. 9, Sept., pp. 925-930.

Slate, F. O., and Matheus, R. E., 1967, "Volume Changes on Setting and Curing of Cement Paste and Concrete from Zero to Seven Days," ACI JOURNAL, *Proceedings* V. 64, No. 1, Jan., pp. 34-39.

Spooner, D. C., 1972, "The Stress-Strain Relationship for Hardened Cement Pastes in Compression," *Magazine of Concrete Research*, London, V. 24, No. 29, June, pp. 85-92.

Spooner, D. C., and Dougill, J. W., 1975, "A Quantitative Assessment of Damage Sustained in Concrete during Compressive Loading," *Magazine of Concrete Research*, London, V. 27, No. 92, Sept., pp. 151-160.

Spooner, D. C.; Pomeroy, C. D.; and Dougill, J. W.; 1976, "Damage and Energy Dissipation in Cement Pastes in Compression," *Magazine of Concrete Research*, London, V. 28, No. 94, Mar., pp. 21-29.

Stevens, R. F., 1969, "Tests on Prestressed Reinforced Concrete Beams," *Concrete*, London, V. 3, No. 11, Nov., pp. 457-462.

Stone, W. C., and Breen, J. E., 1984a, "Behavior of Post-Tensioned Girder Anchorage Zones," *PCI Journal*, V. 29, No. 1, Jan.-Feb., pp. 64-109.

Stone, W. C., and Breen, J. E., 1984b, "Design of Post-Tensioned Girder Anchorage Zones," *PCI Journal*, V. 29, No. 2, Mar.-Apr., pp. 28-61.

Sturman, G. M.; Shah, S. P.; and Winter, G., 1965, "Effects of Flexural Strain Gradients on Micro-Cracking and Stress-Strain Behavior of Concrete," ACI JOURNAL, *Proceedings* V. 62, No. 7, July, pp. 805-822.

Suaris, W., and Fernando, V., 1987, "Ultrasonic Pulse Attenuation as a Measure of Damage Growth during Cyclic Loading of Concrete," *ACI Materials Journal*, V. 84, No. 3, May-June, pp. 185-193.

Suri, K. M., and Dilger, W. H., 1986, "Crack Width of Partially Prestressed Concrete Members," ACI JOURNAL, *Proceedings* V. 83, No. 5, Sept.-Oct., pp. 784-797.

Suzuki, K., and Yoshiteru, O., 1984, "Calculation of Crack Width and Crack Control Design of PPC Members," NATO Advanced Research Workshop on Partial Prestressing, Paris, 35 pp.

Swartz, S. E., and Go, C. G., 1984, "Validity of Compliance Calibration to Cracked Concrete Beams in Bending," *Experimental Mechanics*, SEM, V. 24, No. 2, pp. 129-134.

Tam, K. S., and Scanlon, A., 1986, "Deflection of Two-Way Slabs and Transverse Loads," ACI JOURNAL, *Proceedings* V. 83, No. 5, Sept.-Oct., pp. 737-744.

Testa, R. B., and Stubbs, N., 1977. "Bond Failure and Inelastic Response of Concrete," *Journal of the Engineering Mechanics Division*, ASCE, V. 103, No. EM2, Apr. 1977, pp. 296-310.

Troxell, G. E.; Raphael, J. M.; and Davis, R. E., 1958, "Long-Time Creep and Shrinkage Tests of Plain and Reinforced Concrete," *Proceedings*, ASTM, V. 58, pp. 1101-1120.

USBR, 1981, *Concrete Manual*, 8th Edition, U.S. Bureau of Reclamation, 627 pp.

Verbeck, G. J., 1958, "Carbonation of Hydrated Portland Cement," *Cement and Concrete*, STP-205, American Society for Testing and Materials, West Conshohocken, Penn., pp. 17-36.

Walker, A. J., and Lankard, D. R., 1977, "Bridge Deck Rehabilitation with Steel Fibrous Concrete," Presented at the Third International Exposition on Concrete Construction (New Orleans, LA, Jan.), Battelle Columbus Laboratories.

Wittmann, F. H., 1986, *Fracture Toughness and Fracture Energy of Concrete*, Elsevier Science Publishers, The Netherlands.

Wilson, E. L., 1968, "The Determination of Temperatures within Mass Concrete Structures," *Report* No. 68-17, Structural Engineering Laboratory, University of California, Berkeley, Calif., Dec.

Woods, H., 1968, *Durability of Concrete Construction,* Monograph No. 4, American Concrete Institute/Iowa State University, Farmington Hills, Mich., 187 pp.

Yang, S., and Chen, J., 1988, "Bond Slip and Crack Width Calculations of Tension Members," *ACI Structural Journal*, V. 85, No. 4, July-Aug., pp. 414-422.

Yerlici, V. A., 1975, "Minimum Wall Thickness of Circular Concrete Tanks," *Publication* No. 35-II, International Association for Bridge and Structural Engineering, Zurich, p. 237.

Yoshimoto, A.; Ogino, S.; and Kawakami, M., 1972, "Microcracking Effect on Flexural Strength of Concrete after Repeated Loading," ACI JOURNAL, *Proceedings* V. 69, No. 4, Apr., pp. 232-240.

Yoshimoto, A.; Ogino, S.; and Kawakami, M., 1976, "Microscopic Cracks in Cement Matrix and Deformation Behavior of Concrete," *Proceedings*, 19th Japan Congress on Materials Research, pp. 126-131.

Ytterberg, R. F., 1987, "Shrinkage and Curling of Slabs on Grade," *Concrete International: Design and Construction*, V. 9, No. 4, Apr., pp. 22-31.

Zielinski, J. L., and Rowe, R. E., 1960, "An Investigation of the Stress Distribution in the Anchorage Zones of Post-Tensioned Concrete Members," *Technical Report* No. 9, Cement and Concrete Association, London, Sept., 32 pp.

9.3—Other references

ACI Committee 214, 1977, "Recommended Practice for Evaluation of Strength Test Results of Concrete (214-77)," American Concrete Institute, Farmington Hills, Mich., 14 pp.

ACI Committee 224, 1990, "Control of Cracking in Concrete Structures (224R-90)," American Concrete Institute, Farmington Hills, Mich., 43 pp.

ACI Committee 224, 1992, "Cracking of Concrete Members in Direct Tension (224.2R-92)," American Concrete Institute, Farmington Hills, Mich., 12 pp.

ACI Committee 224, 1993, "Causes, Evaluation, and Repair of Cracks in Concrete Structures (224.1R-93)," American Concrete Institute, Farmington Hills, Mich., 22 pp.

ACI Committee 347, 1969, "Precast Concrete Units Used as Forms for Cast-in-Place Concrete (347.1R-69)," American Concrete Institute, Farmington Hills, Mich.

ACI Committee 544, 1996, "State-of-the-Art Report on Fiber-Reinforced Concrete (544.1R-96)," American Concrete Institute, Farmington Hills, Mich., 66 pp.

ASTM C 512, "Test Method for Creep of Concrete in Compression," American Society of Testing and Materials, West Conshohocken, Penn.

ASTM C 595, "Specifications for Blended Hydraulic Cement," American Society of Testing and Materials, West Conshohocken, Penn.

Goyal, B. B., ed., 1982, *Designing for Creep and Shrinkage in Concrete Structures*, SP-76, American Concrete Institute, Farmington Hills, Mich., 484 pp.

Designing for Effects of Creep, Shrinkage and Temperature in Concrete Structures, 1971, SP-27, American Concrete Institute, Farmington Hills, Mich., 430 pp.

Neville, A. M., ed., 1964, *Symposium on Creep of Concrete*, SP-9, American Concrete Institute, Farmington Hills, Mich., 160 pp.