

Control of Cracking of Concrete Structures

Reported by ACI Committee 224



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Control of Cracking of Concrete Structures

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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; crack width and spacing; cracking (fracturing); drying shrinkage; heat of hydration; mass concrete; microcracking; polymer-modified concrete; prestressed concrete; reinforced concrete; restraint; shrinkage; shrinkage-compensating concrete; temperature; tensile stresses; thermal expansion; volume change.

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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 (Attigbo 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

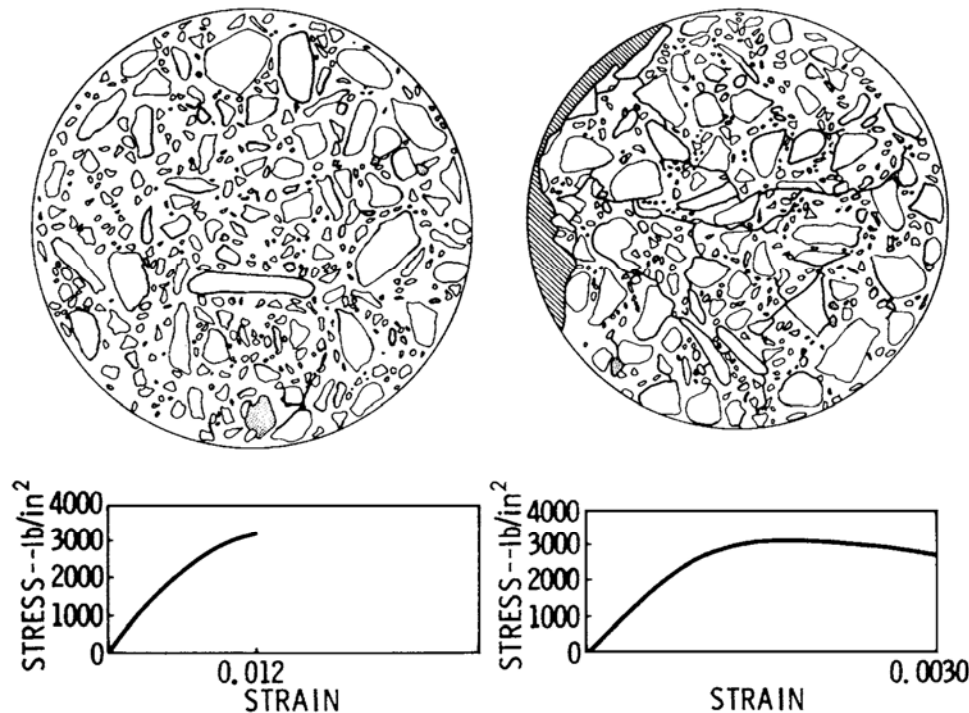


Fig. 2.1—Cracking maps and stress-strain curves for concrete loaded in uniaxial compression (Shah and Slate 1968).

material. The theory uses the stress-intensity factor K that 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 scanning electron microscope (SEM). He subjected dried concrete

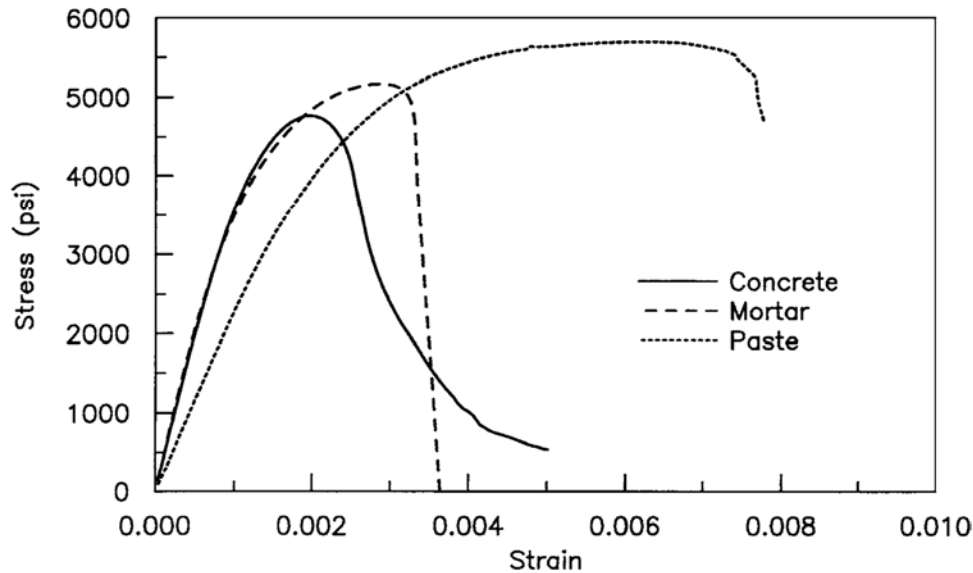
Paste, Mortar and Concrete – $W/C = 0.5$ 

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

specimens to eccentric compressive loading within the SEM. He observed that microcracks that exist 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

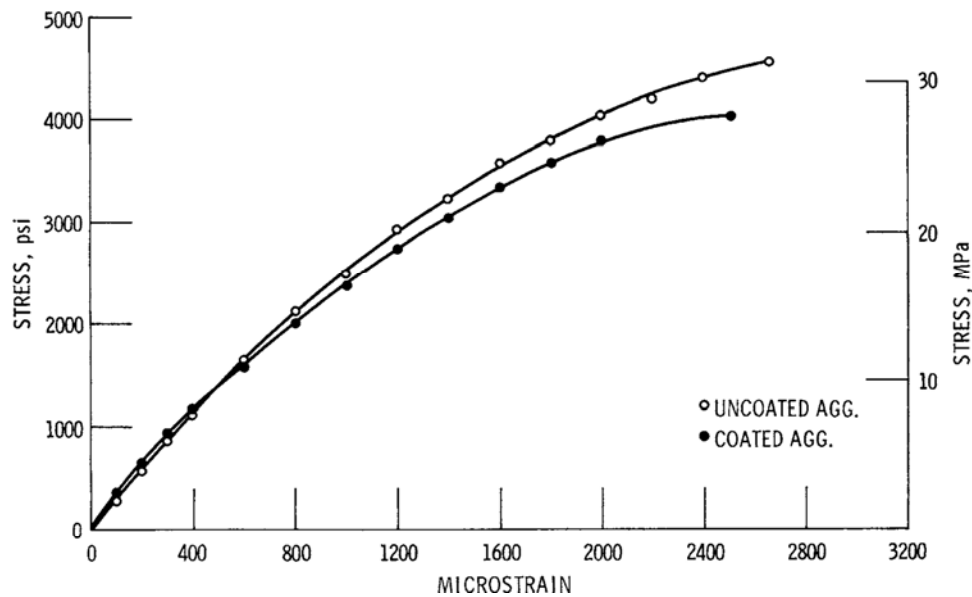


Fig 2.3—Stress-strain curves as influenced by coating aggregates (Darwin and Slate 1970).

point was first made by Spooner, Pomeroy, and Dougill (1976) based on volumetric strain measurements and then by 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-

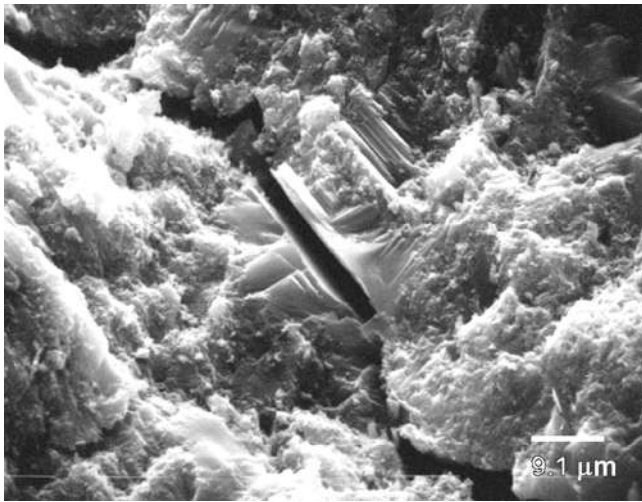


Fig 2.4—Crack through calcium silicate-hydrate and calcium hydroxide in cement paste (Attiogbe and Darwin 1987).

strain behavior can be obtained using a nonlinear representation for the mortar constituent of the physical model.

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

$\times 10^{-5}$ in.) and in length from 10 to over 200 μm (4 to over 80 $\times 10^{-4}$ 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 normal-weight 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 McGarity 1972; Sorenson, and Arneson 1977; Hillmeier and Hilsdorf 1977).

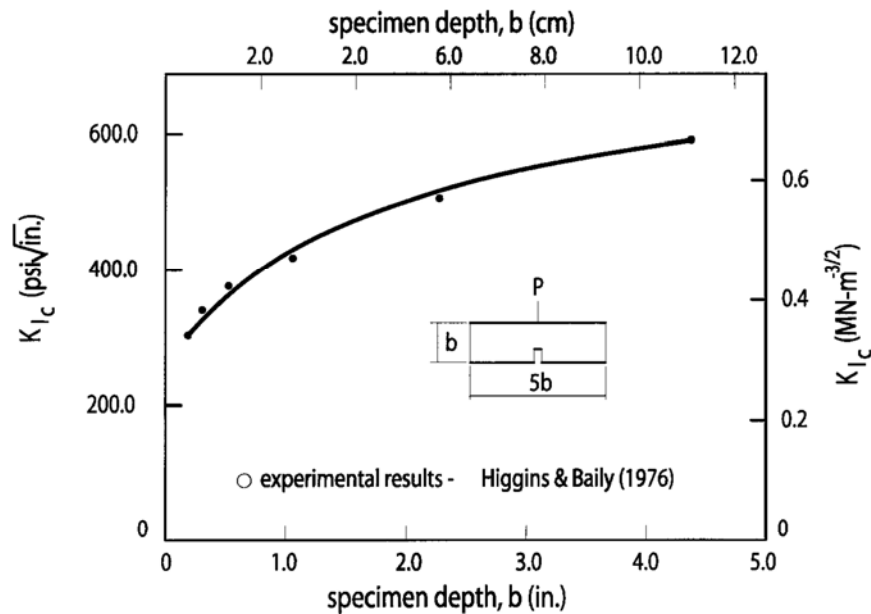


Fig. 2.5—Size effect on stress-intensity factor (based on data from Higgins and Bailey 1976).

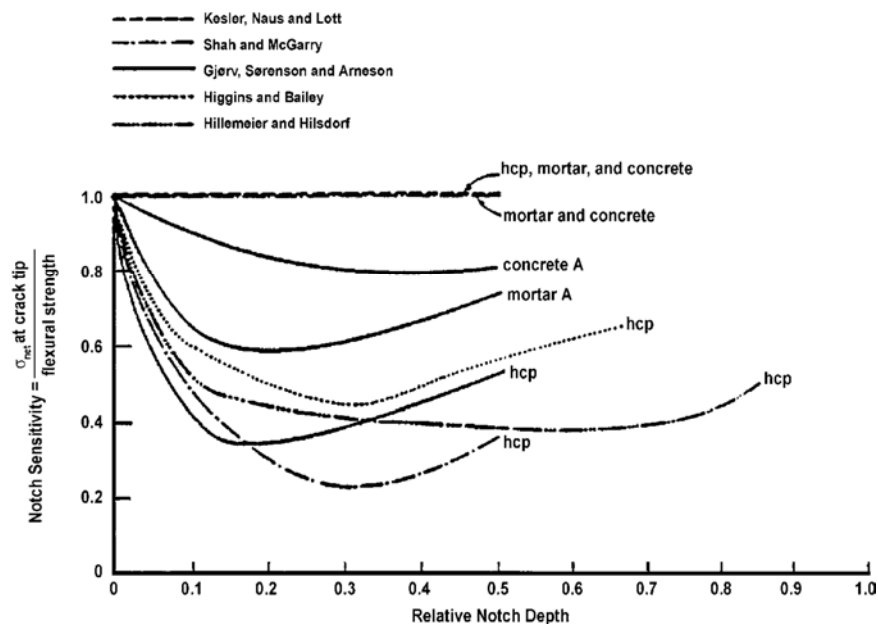


Fig. 2.6—Effect of relative notch depth on notch sensitivity (based on data from Higgins and Bailey 1976; Kesler, Naus, and Lott 1972; Shah and McGarry 1971; Gjorv, Sorenson, and Arneson 1977; Hillemeier and Hilsdorf 1977).

The specimens showing no notch sensitivity are likely the result of deficiencies in the test methods, as explained by Gjorv 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 (Gjorv et al. 1977; Wittmann 1986). It is also widely accepted (Linsbauer et al. 1989a,b), 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 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 tested in three-point bending is shown in Fig. 2.7, where the crack growth ahead of the notch was contin-

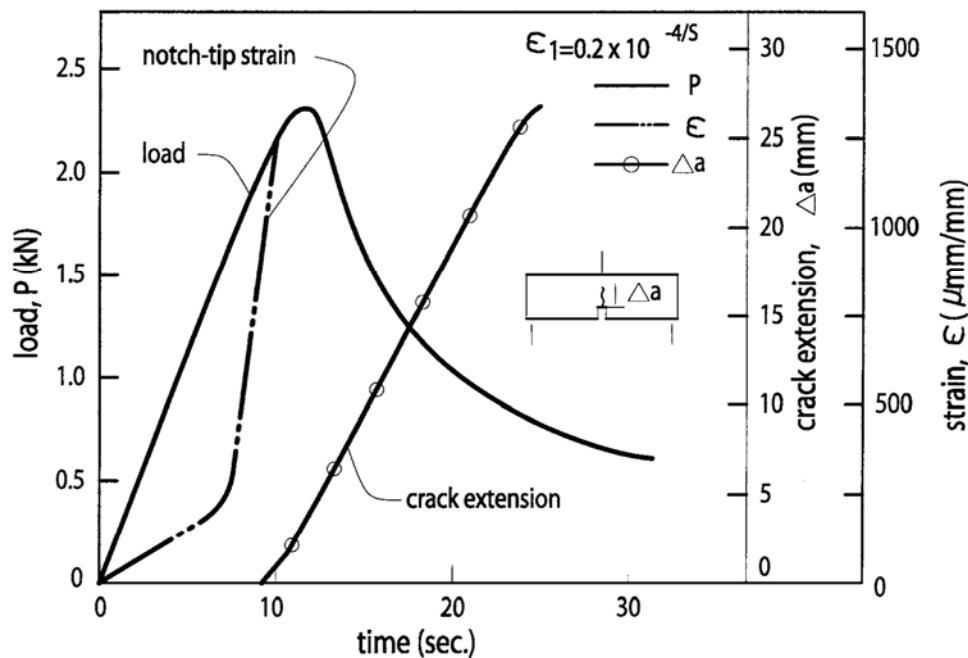


Fig. 2.7—Precritical crack growth (John and Shah 1986).

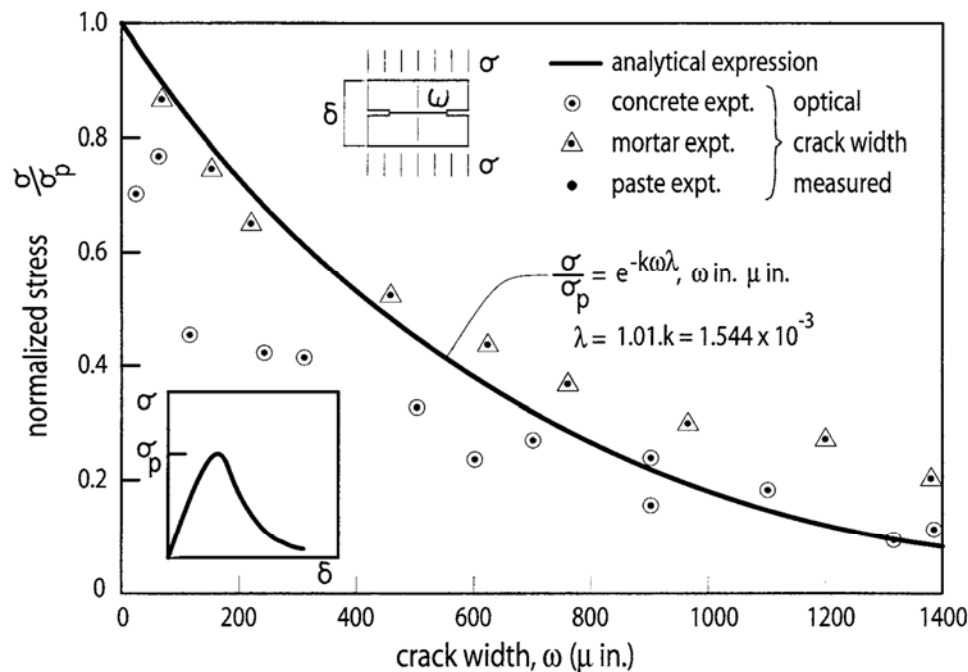


Fig. 2.8—Normalized peak stress versus crack width in uniaxial tension (Gopalratnam and Shah 1986).

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

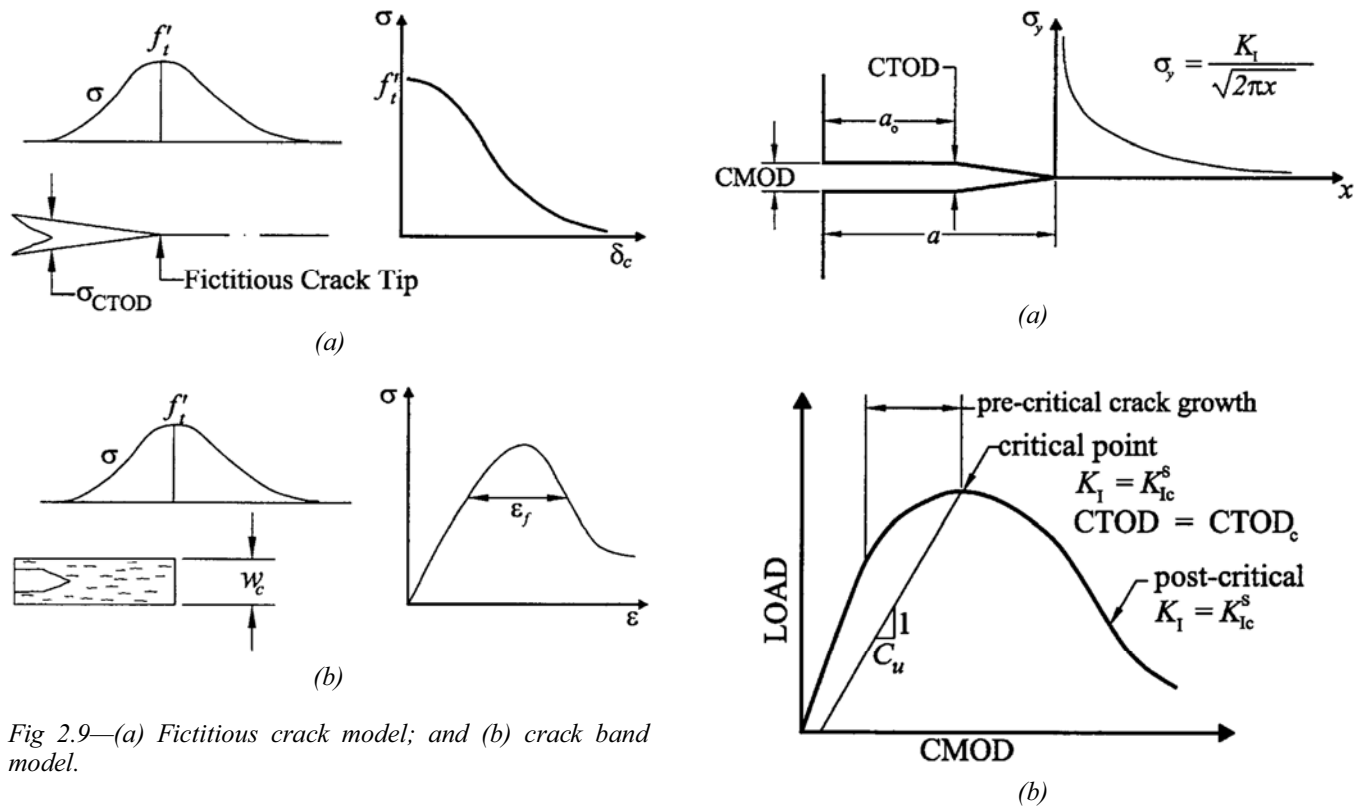


Fig. 2.9—(a) Fictitious crack model; and (b) crack band model.

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 .

Bazant 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 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

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

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

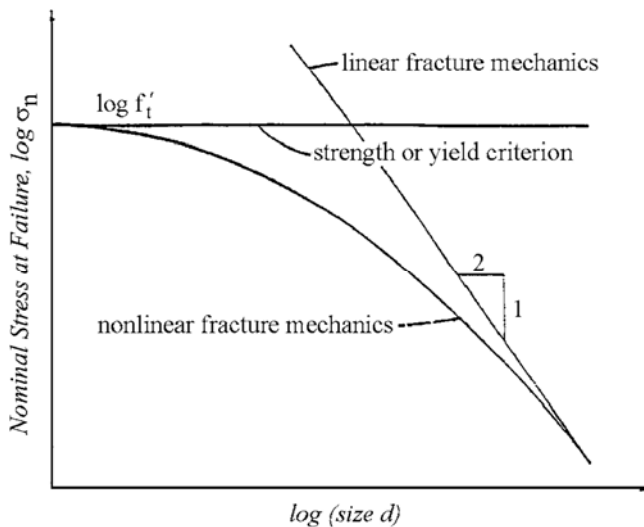


Fig. 2.11—Size-effect law (Bažant, Kim, and Pfeiffer 1986).

Jeng 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)

$$\sigma_N = \frac{Bf'_t}{\sqrt{(1 + d/d_o)}} \quad (2-1)$$

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 $\sigma_N = Bf'_t$ 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 $\sigma_N = \text{constant}/\sqrt{d}$. 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 in 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).

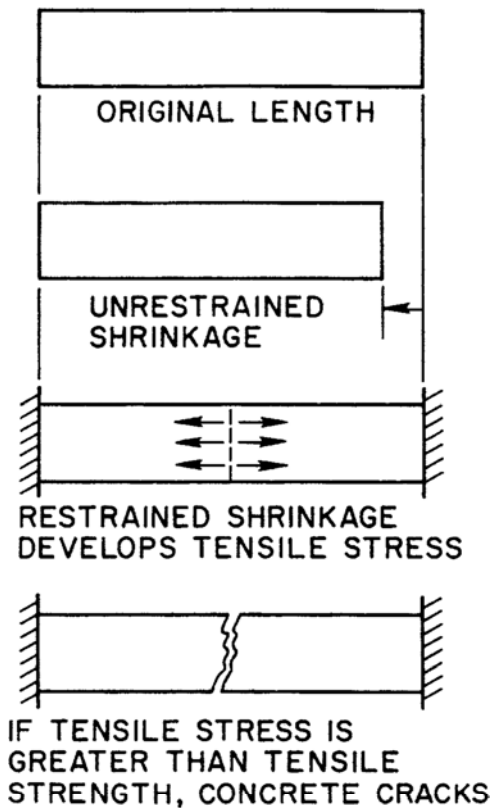


Fig. 3.1—Cracking of concrete due to drying shrinkage.

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 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

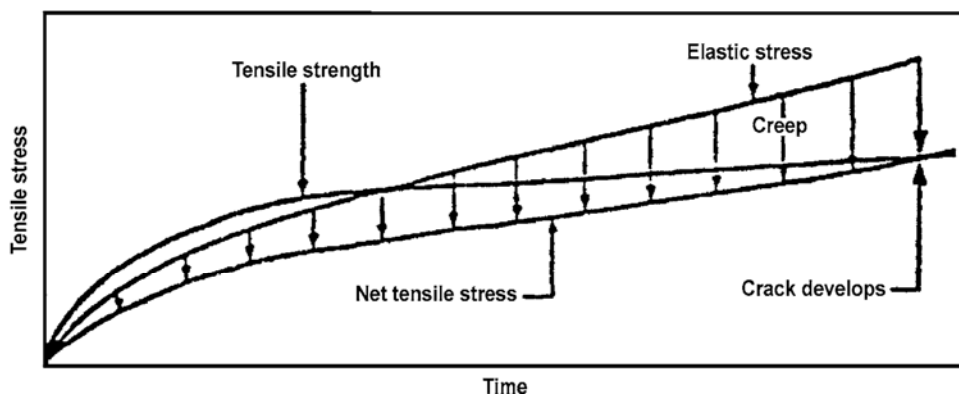


Fig. 3.2—Effect of creep on tensile stress.

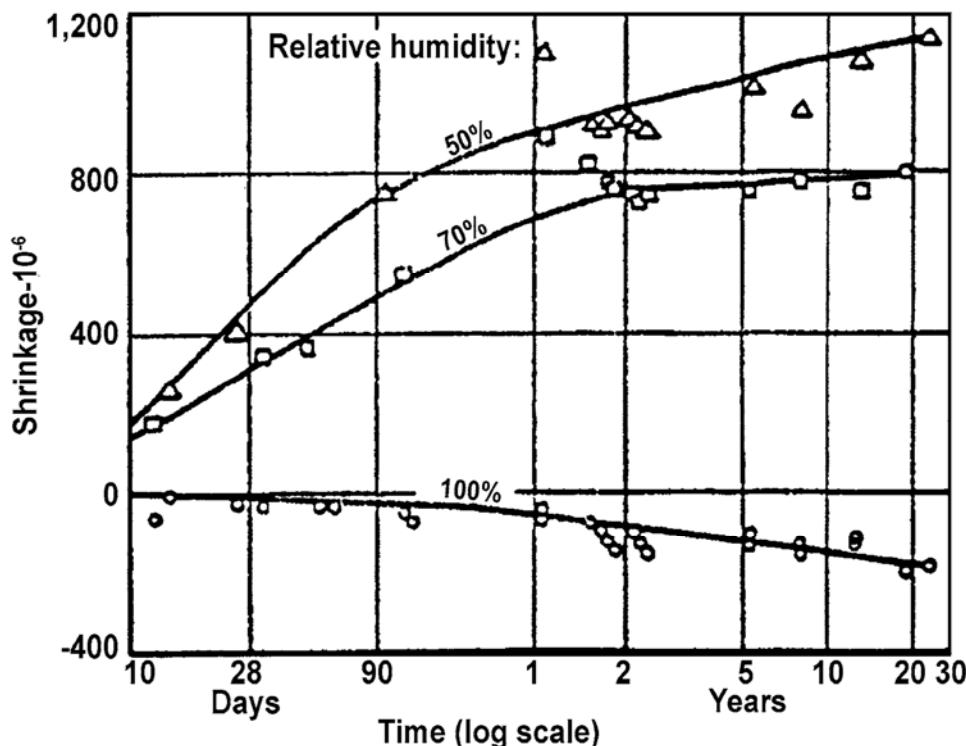


Fig. 3.3—Relations between shrinkage and time for concretes stored at different relative humidities. Time reckoned since end of wet curing at 28 days (Troxell, Raphael, and Davis 1958).

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 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

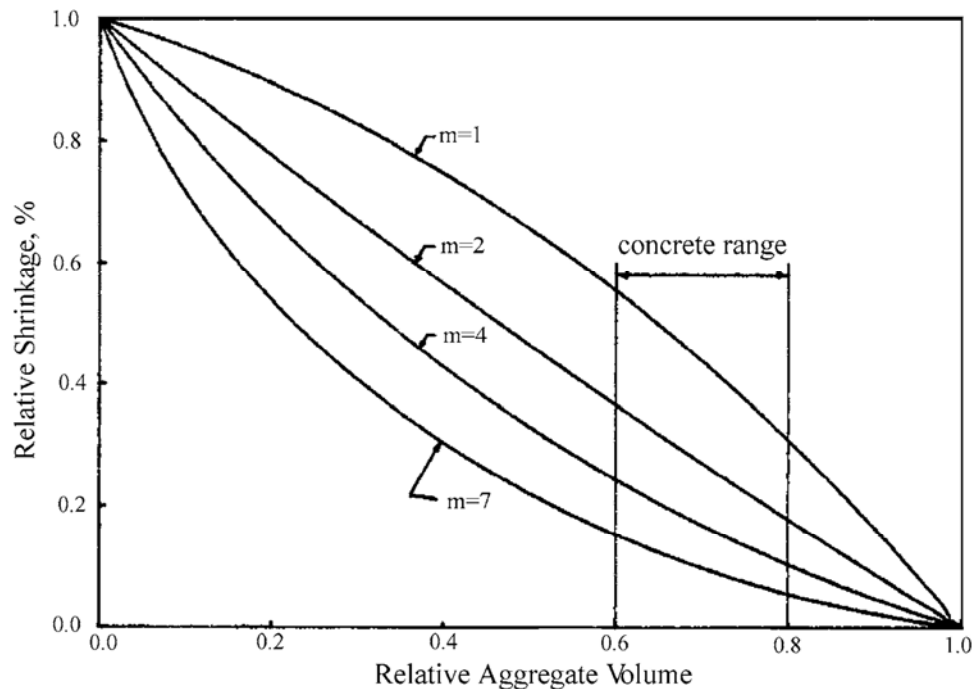


Fig. 3.4—Effect of relative aggregate content and modulus ratio on drying shrinkage of concrete (Hansen and Almudaiheem 1987).

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

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 (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. Constitutive models for predicting the influence of relative aggregate content and

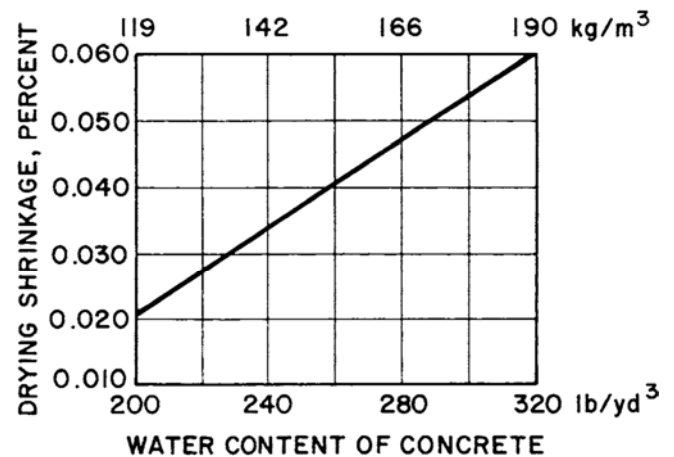


Fig 3.5—Typical effect of water content of concrete on drying shrinkage (USBR 1981).

modulus ratio on ultimate concrete shrinkage have been developed by Pickett (1956) and Hansen and Almudaiheem (1987).

The influence of aggregate-absorption capacity on concrete shrinkage was investigated by Carlson (1938) and is illustrated in Table 3.1; the concrete had identical cements and w/cm . 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 effects on ultimate drying shrinkage varies.