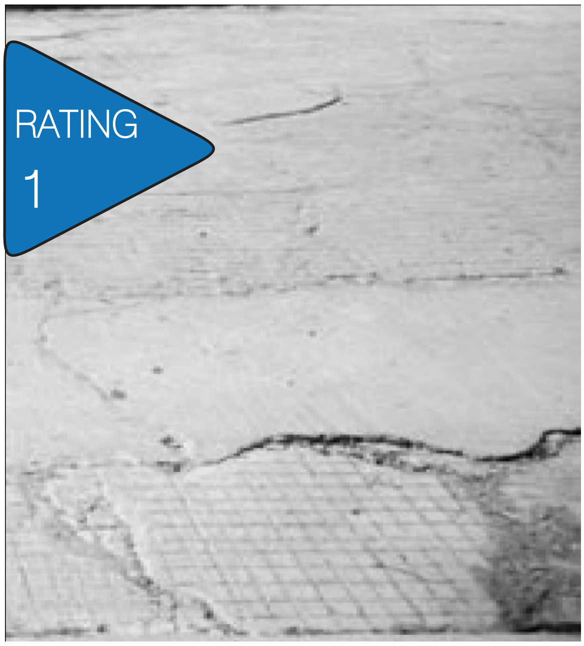
Control of Cracking in Concrete Road



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Introduction

The age-old axiom in concrete construction is that concrete cracks. While cracks may develop in concrete for a variety of causes, the underlying principle is the relatively low tensile strength of concrete. Visible cracking occurs when the tensile stresses exceed the tensile strength of the material. Visible cracking is frequently a concern since these cracks provide easy access for the infiltration of aggressive solutions into the concrete and reach the reinforcing steel or, other components of the structure leading to deterioration. This document reviews the causes of cracking, discusses various tests that can be performed to assess the susceptibility of a material to cracking, and provides several case studies.

It is important to understand why cracks develop in highway concrete structures and pavements. While some commonly think of external loading as being responsible for generating the majority of the tensile stresses in a material, much of the cracking in concrete can be traced to an intrinsic volumetric instability or the deleterious chemical reactions. The volume instability results in response to moisture, chemical, and thermal effects. In addition, various deleterious chemical reactions involving the constituents of concrete or embedded materials can play significant roles causing localized internal expansions.

The impact of cracking on durability, especially corrosion, is detrimental to many transportation structures. In particular, cyclic or tidal exposures initiate dry-wet cycles and provide a constant source of salts to enter the cracks, significantly exacerbating deterioration. Similarly, cracked concrete in contact with sulfate rich soil can lead to accelerated sulfate attack. The complex relationships between cracking and accelerated deterioration are unique to each situation and are not well understood. Thus considerable attention is needed from the research community to fully understand the principles involved and transfer them to the practicing engineering community for improved durability.

Figure 1 provides a listing of some of the common types of cracks and distinguishes these cracks based upon when they appear in concrete, before hardening or after hardening.

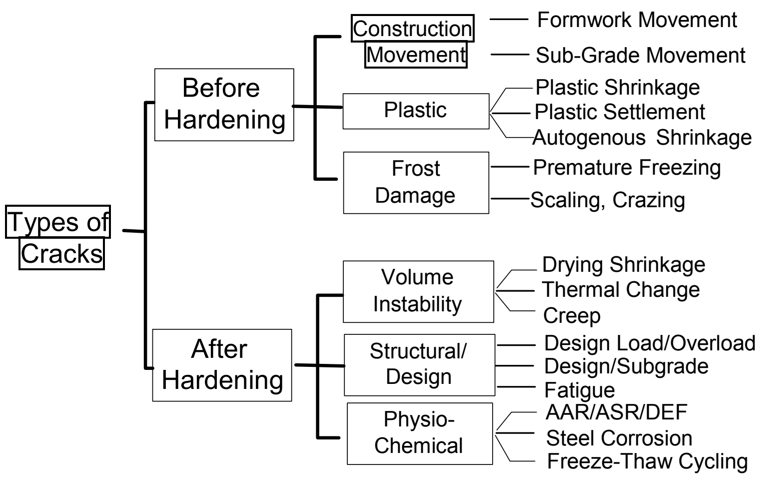


FIGURE 1 Common causes for cracking in concrete dtructures.

Cracks that occur before hardening, primarily due to settlement, construction movements, and excessive evaporation of water, are called plastic cracks. Plastic cracking can be predominantly eliminated through close attention to the mixture design, material placement, and curing. Cracks that occur after the concrete has hardened may be due to a variety of reasons. These cracks may be due to mechanical loading, moisture and thermal gradients, chemical reactions of incompatible materials (e.g., alkali-aggregate reactions) or environmental loading (e.g. freezing of water in unsound aggregate or paste). Table 1 provides a summary of cracks due to environmental conditions, and discusses when they are most likely to occur.

As civil engineers begin the process of rehabilitating more infrastructure elements and simulating long-term performance of the infrastructure using computer models, it is more critical than ever that they have a good understanding of the impact of cracks on performance. The role cracks play in the performance of transportation structures is somewhat controversial however.

TABLE 1 Classification of Cracks

Type of Cracking	Form of Crack	Primary Cause	Time of Appearance
Plastic settlement	Over and aligned with reinforcement, subsidence under reinforcing bars	Poor mixture design leading to excessive bleeding, excessive vibrations	10 min to 3 h
Plastic shrinkage	Diagonal or random	Excessive early evaporation	30 min to 6 h
Thermal expansion and contraction	Transverse	Excessive heat generation, excessive temperature gradients	1 day to 2–3 weeks
Drying shrinkage	Transverse, pattern or map cracking	Excessive mixture water, inefficient joints, large joint spacing's	Weeks to months
Freezing and thawing	Parallel to the surface of concrete	Lack of proper air void system, non durable coarse aggregate	After one or more winters
Corrosion of reinforcement	Over reinforcement	Inadequate cover, ingress of sufficient chloride	More than 2 years
Alkali–aggregate reaction	Pattern and longitudinal cracks parallel to the least restrained side	Reactive aggregate plus alkali hydroxides plus moisture	Typically more than 5 years, but weeks with a highly reactive material
Sulfate attack	Pattern	Internal or external sulfates promoting the formation of ettringite	1 to 5 years

For example, there are contradictory beliefs on how cracking influences corrosion and deterioration. Some believe that cracks accelerate corrosion and cause extensive damage by enabling the rapid penetration of chlorides, oxygen and water to easily reach the reinforcing steel, while others believe that corrosion in cracked concrete occurs in localized regions and does not therefore result in extensive damage.

Based on laboratory studies it appears that crack width has a significant influence on the corrosion process. For example, some have reported that when the cracks remained relatively small [< 1 mm (0.04 in.)], they had little impact on the corrosion process; however, larger cracks [>1 mm (0.04 in.)] increased the corrosion rate. Recent studies on reinforced concrete beam elements (Yoon et al., 2000) have shown that cracking, especially under sustained load which act to hold the crack open can produce accelerated corrosion and strength loss. Although there are controversial findings about the impact of crack width on corrosion rate, there exists a general agreement that cracking reduces the time to corrosion initiation. The localized corrosion at the cracked areas lead to further longitudinal surface cracking, delaminating, and deboning, ultimately resulting in a reduction in the strength capacity and stiffness of the structure. Studies investigating the performance of concrete bridge barriers documented that a porous layer of concrete is often present under the top reinforcement. Water and other contaminants penetrate through the cracks and move through the porous layer, initiating corrosion along the full length of the reinforcement (Park and Paulay, 1975; Attanayake and Aktan, 2004). Cracks forming in fracture critical portions of structural components and contributing to deterioration are a safety concern. Early tendon corrosion that initiates from moisture ingress through the cracks within the end zone interferes with the load path and reduces the beam capacity (Aktan et al., 2003).

While cracking is commonly observed in concrete structures, it is important to understand that all cracks may have different causes and different effects on long-term performance due to the confounding effects of design, traffic loads, and climatic conditions relevant to the structure. Cracking need not be alarming and can be addressed appropriately so that the life of the structure is not compromised. This document describes the various causes of crack formation in concrete transportation structures and the relevance of such cracks to concrete performance. Quality control testing methods are described that can be used to assess how susceptible different concrete elements may be to cracking. Strategies to minimize cracking and its effects are outlined and several case histories are presented.

Types of Crack

Figure 1 (Ref. 1) shows part of a structure with a variety of crack types. Most of the cracks actually shown could be broadly classified as non-structural cracks, the term taken to mean cracks which arise from causes not associated with the primary load carrying function of the structure. This category includes • Plastic settlement cracks

- Plastic shrinkage cracks
- Thermally induced cracks
- Drying shrinkage cracks
- Corrosion spelling
- Alkali aggregate cracks and cracks due to other chemical effects

Included in the category of structural cracks are bending and shear cracks referred to in the figure but not shown. Note that cracks in the non-structural category are often the ones of concern. In many instances these can be prevented by good construction practice.

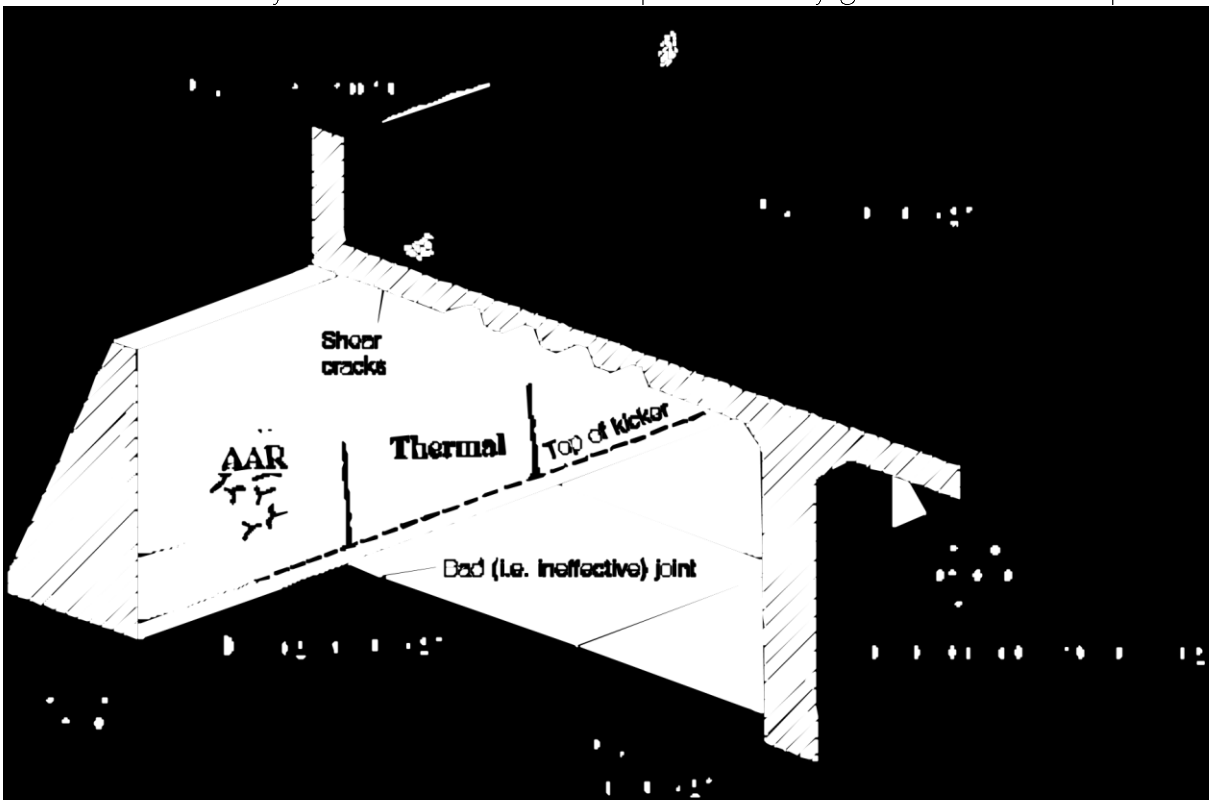


Figure 1: Examples of Crack Patterns (Ref 1)

Cracks initiate at defects in the presence of stress. Provided certain conditions are fulfilled, the crack may propagate. The propagation may be slow or fast. For example, load added to a simply supported beam can cause rapid development of flexural cracks. Shrinkage induced cracking in a slab can be slow to develop, taking many months for propagation across the full width of the slab. While the effect that produces the tensile stresses necessary for cracking may be different in each instance, the initiation and propagation of the cracks is governed by the same concrete properties and crack mechanics. Some crack mechanisms are reviewed in the section 4.

Causes of Cracking

Concrete structures do not frequently fail due to lack of strength, rather due to inadequate durability or due to improper maintenance techniques. The most common cause of Premature deterioration is attributed to the development of cracks (Mehta, 1992; Hobbs, 1999). Cracking can occur in concrete pavements and structures for several reasons that can primarily be grouped into either mechanical loading or environmental effects. It should also be noted that for most practical structures, reinforcement is used to bridge and hold cracks together when they develop, thereby assuring load transfer while adding ductility to a relatively brittle material. Therefore not all cracking causes concern. Reinforced concrete elements are frequently designed on the assumption that cracking should take place under standard loading conditions (Nilsson and Winter, 1985; Nawy, 2000). For example continuously reinforced concrete pavements (CRCP) are designed with longitudinal steel in an amount adequate to hold shrinkage cracks tight, while joints exist only at locations of construction transitions and on-grade structures. In this pavement type wherein shrinkage cracks develop over time and stabilize over the first 3 to 4 years, cracking in the transverse direction in specific patterns is not detrimental to the structure as long as the cracks remain tight and retain good load transfer. Therefore, cause of cracking should be carefully identified to determine which cracks are common and acceptable and which cracks merit repair or further investigation. Several guides currently exist to assist in determining the cause of cracking including the American Concrete Institute (ACI) committee reports "Guide for Making and Condition Survey of Concrete in Service" (ACI 201-92) and "Causes, Evaluation and Repair of Cracks in Concrete Structures" (ACI 224-R93).

Mechanical loads induce strains that can exceed the strain capacity (or strength capacity) of concrete, thereby causing cracking. Concrete may be particularly susceptible to cracking that occurs at early-ages when concrete has a low tensile capacity (Kasai, 1972). If the loads are applied repeatedly or over a long period of time, fatigue and creep can affect the strain (or strength) development that can lead to failure (Bazant and Celodin, 1991) or reduce stresses (Shah et al., 1998).

Although numerous factors influence whether concrete would be expected to crack due to environmental effects, it can be simply stated that cracking will occur if the stress that develops in response to internal expansion or the restraint of a volumetric contraction that results in stress development exceeds the strength (or fracture resistance) of the material. Internal expansion is primarily caused by chemical attack or freezing of the pore water while volumetric contraction is typically attributed to moisture changes, chemical reactions, and thermal changes.

MECHANICAL LOADING

Static Loading

Concrete is a composite material that is made by binding aggregates together with a cementitious paste. While the independent response of a cement paste and aggregate to an applied load is linear as shown in Figure 2, it can be seen that response of the composite concrete is highly nonlinear. This non-linearity can be attributed to the development of small cracks (microcracks) throughout the concrete matrix as load is applied (Hsu et al., 1963). Others have suggested that

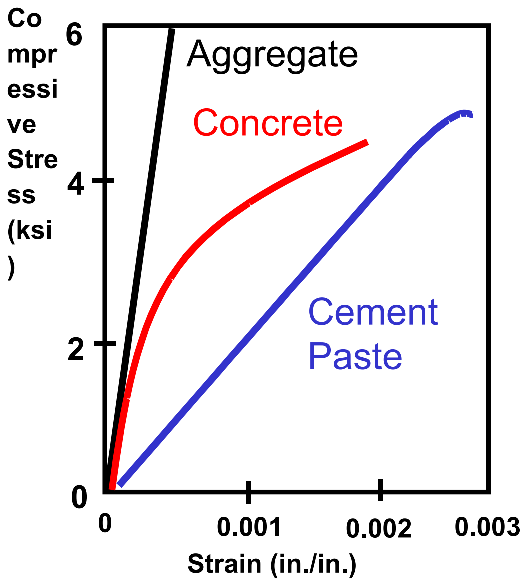


FIGURE 2 Stress strain response of behavior of concrete (1 ksi = 6.89 MPa).

this may be attributed to existence of a weak bond or interfacial transition zone between the aggregate and the paste matrix (Mehta, 1996). While these cracks occur over a wide range of load levels they can be attributed to the development of high local stresses that occur at the interface of the aggregates and paste (Shah and Slate, 1965).

The response of unreinforced concrete to mechanical loading must first be described to fully understand how reinforced elements react. Immediately upon loading, concrete typically is thought to develop some micro-cracking (Shah and Slate, 1965; Attiogbe and Darwin, 1987; Li et al., 1991), though it is frequently assumed to be negligible since little change is detected in the load-displacement response. The load-displacement response remains fairly linear until the load level reaches approximately 40% to 50% of the maximum strength. At this time the stress-strain response becomes less linear as an increase in micro-cracking occurs resulting in the decrease of the elastic modulus of the material. As the load level approaches 90% to 95% of the peak, the slope of the load-displacement curve is once again reduced as the cracks begin to coalesce and localize in one region of the specimen. This localized area will eventually become the location of a visible crack. Depending on how the specimen is loaded (i.e., load control, displacement control) the crack may result in sudden failure (load control) or continue to develop and grow after the peak load is reached (displacement control) resulting in large visible cracking. After the peak load is achieved the specimen begins to demonstrate strain-softening behavior resulting in a gradual decrease in load carrying capacity with increasing strain as shown in Figure 3 (Jansen and Shah, 1995).

During the post peak region of the stress-strain curve, the response of concrete can be idealized as two types of materials, the bulk concrete and damaged zone, which behave quite differently from each other (Bazant, 1976; Hillerborg et al., 1976; Shah and Jansen, 1993). A typical model for unloading of these parts is shown in Figure 4.

It should also be noted that the strength of concrete significantly increased over the last two decades through the increased use of lower water to cement ratio mixtures and the use of supplementary cementing materials. As the strength of these materials increases the material

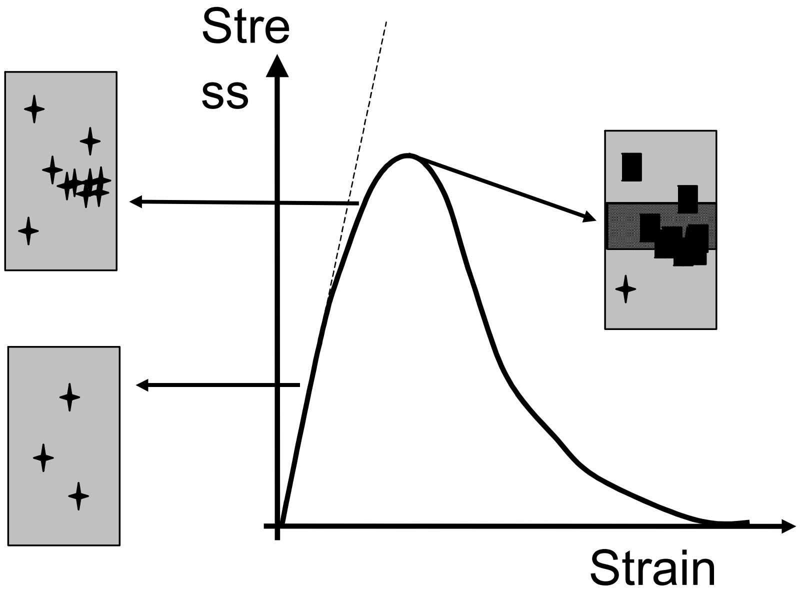


FIGURE 3 Strain softening aggregate and rock.

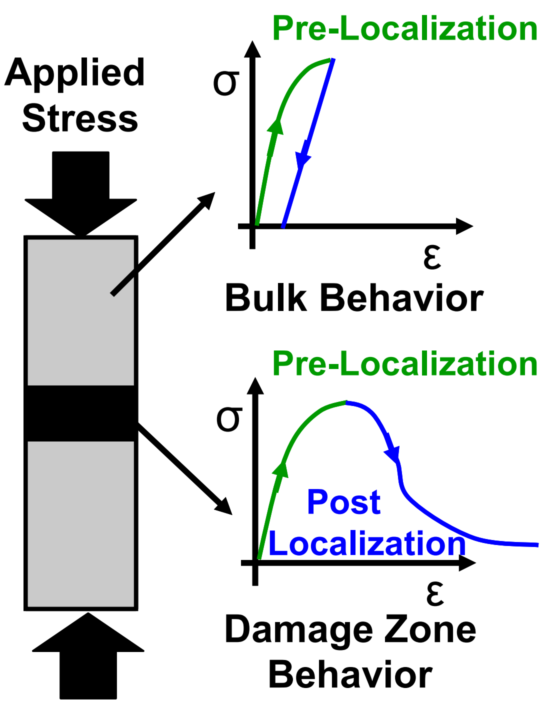


FIGURE 4 Composite model for the response of damaged concrete.

becomes increasingly brittle resulting in a steeper post-peak response as shown in Figure 5 (Jansen and Shah, 1995).

The cracks in a structure typically develop at the location that has the highest stress and the weakest bond. This can occur at a reduced section, a preexisting flaw, or an area of stress concentration. The study of how cracks develop and propagate in a structure is commonly referred to as fracture mechanics. Over the last four decades significant research has been performed to better understand the fracture processes in concrete. Fracture mechanics differs from continuum mechanics approaches in that it relates local stress levels (stress intensity) with the existence of a crack. The energy released with crack growth (creation of surface area) can be related to the change in local stress level. Developments in non-linear fracture mechanics research over the last three decades have shown that concrete is a quasi-brittle material and exhibits precritical crack growth (fracture process zone) and strain softening (post-peak stress transfer). Although the subject of fracture mechanics is beyond the scope of this document, additional information can be found in several recent books that summarize the main attributes of concrete fracture (Shah et al., 1995; Bazant and Planas, 1997; Van Mier, 1999).

Cyclic Loading (Fatigue)

Failure under repeated mechanical loading is referred to as fatigue or cyclic loading. Note that the load levels in the case of fatigue failures are not sufficient to result in failures under static conditions. While the mechanisms of fatigue failure are not completely understood there are two hypotheses concerning crack initiation and its evolution in plain concrete. The first hypothesis attributes the fatigue failure to the progressive deterioration of the bond between the coarse aggregate and the matrix. The second hypothesis attributes the fatigue failure in concrete to the coalescence of pre-existing micro-cracks in the matrix, resulting in a single localized macro-

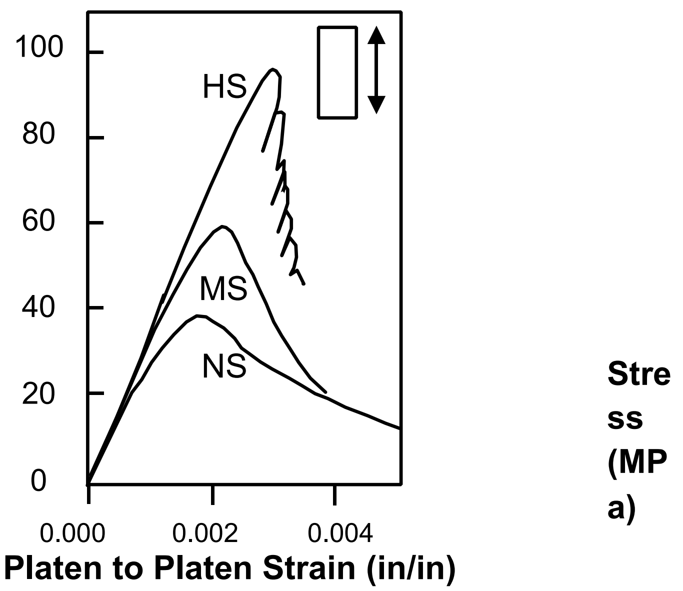


FIGURE 5 Influence of strength on the stress-strain response (1 MPa = 145 psi).

crack. Fatigue causes a crack to propagate through the matrix (typically starting along the interfacial zone between an aggregate and the paste). As cyclic loading proceeds, stresses are redistributed and the macro-crack width decreases but never closes completely.

It should be noted that concrete, like most other heterogeneous materials, exhibits a great deal of scatter. It can be seen that at stress levels of approximately 50% a plateau is typically observed. Reinforced concrete is more resistant to fatigue damage due to the presence of steel. For further information on fatigue behavior of plain and reinforced concrete the reader is referred to the ACI 215 committee report "Consideration for Design of Concrete Structures Subjected to Fatigue Loading."

VOLUMETRIC STABILITY

Settlement

Settlement cracking occurs in freshly mixed concrete as the concrete settles over time and encounters some restraint. The heavier particles 'sink' due to gravity until the concrete sets. Plastic settlement cracking has been frequently observed to occur at changes in cross section (i.e., over reinforcing bars or at change in section height). The practical significance of settlement cracking is in the construction of reinforced slabs, and bridge decks. The magnitude of tensile stress generated as a result of plastic settlement, along with the capillary stress and the auto genius effect, may be sufficient to initiate plastic cracking.

The role of settlement in plastic cracking has been studied for several decades. Powers (1968) measured the settlement of cement paste by manually monitoring the displacement of a steel pin resting on the surface of fresh concrete. The amount of settlement observed was related to specimen height, water-to-cement ratio (w/c) and concrete consistency (Powers, 1968). A uniform settlement (i.e., homogenous volume contraction) in a fresh concrete mixture does not lead to plastic cracking. Differential settlement however can lead to cracking. Differential settlement can be caused by either external boundaries or embedded rigid inclusions. Weyers et al. (1982) simulated the settlement behavior occurring due to embedded rigid inclusions using a model where rebar was positioned in a photoelastic material (gelatin) at variable cover depths and spacing. It was concluded that clear cover depth, rebar size and rebar spacing are the major factors affecting the magnitude of differential settlement with lager bars and smaller cover depths typically resulting in larger cracks.

Kayir and Weiss (2002) used a non-contact laser device to quantify the amount of settlement occurring in between the time of concrete placement and setting for mortar containing chemical admixtures. It was shown that the mixing and placement time significantly influence the amount of settlement that may occur. For example, when compared with the settlement measured immediately after mixing, the settlement in a material placed 40 minutes after mixing showed nearly a 50% reduction in settlement. Qi et al. (2003; 2005a) demonstrated that fiber reinforcement dramatically reduces settlement capacity of fresh concrete. Qi et al. (2005b) demonstrated a moving laser system to measure differential shrinkage over reinforcing steel or at changes in cross-sectional height.

Shrinkage in Fresh Concrete

Plastic shrinkage can occur at the surface of fresh concrete within the first few hours after placement. When the rate of evaporation of water from the surface of concrete exceeds its bleeding rate the surface begins to dry resulting in high capillary stress development near the surface [Cohen et al. 1989]. This can be attributed typically to high temperatures, low ambient humidity, high winds, and mixture ingredients and proportions [ACI 305]. Plastic shrinkage cracking is a problem for large flat structures, such as bridge decks and pavements, in which the exposed surface area is high relative to the volume of the placed concrete.

Cracks caused by plastic shrinkage can be quite wide on the upper surface 2 to 3 mm (0.08 to 0.12 in.), but their width often decreases rapidly below the surface. Plastic cracks typically do not exceed 10 mm but may pass through the full depth of the member; however the mechanisms leading to the formation of plastic shrinkage cracking does not explain full depth cracks. It is probable that the subsequent events including drying shrinkage and loading can cause the plastic shrinkage cracks to propagate.

Figure 6 shows the influence of the drying environment at early ages on the magnitude of shrinkage. The three different curing environments include wet (100% RH), dry (40% RH) and wind [40% RH with wind at 2.5 m/s (8.2 ft/s)]. Holt (2001) suggested that there is a higher risk of early age cracking when the early age shrinkage exceeds 1000 μm/m (0.001 in./in.).

This example shows that the construction environment is a major concern when assessing the risk of this early age cracking.

The amount of shrinkage that occurs is directly related to the loss of water from the concrete, greater evaporation leads to greater shrinkage. This correlation is shown in Figure 7 (Holt and Leivo, 2000; Leivo and Holt, 2001) for normal strength concretes with different proportions that are exposed to different curing conditions.

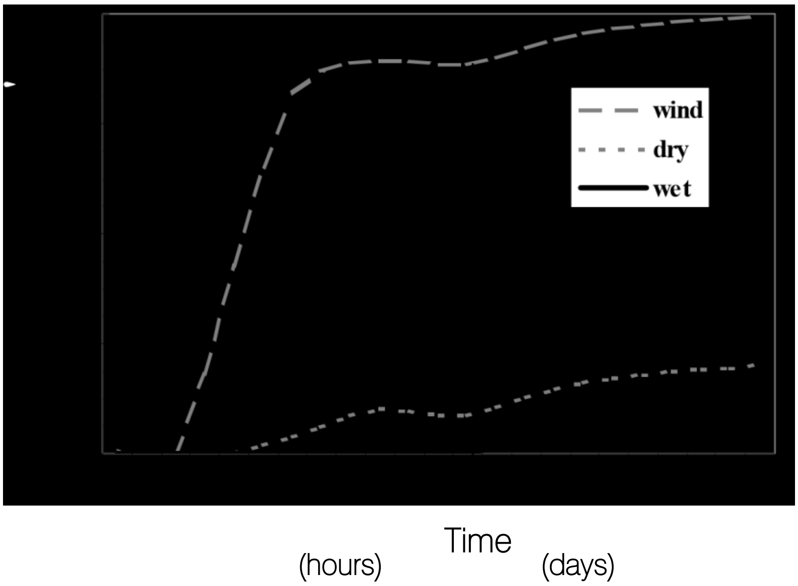


FIGURE 6 Combined early age and long-term shrinkage for three different curing environments (Holt and Leivo, 2000). [1 mm/m = 1,000 μm/m (0.001 in./in.).]

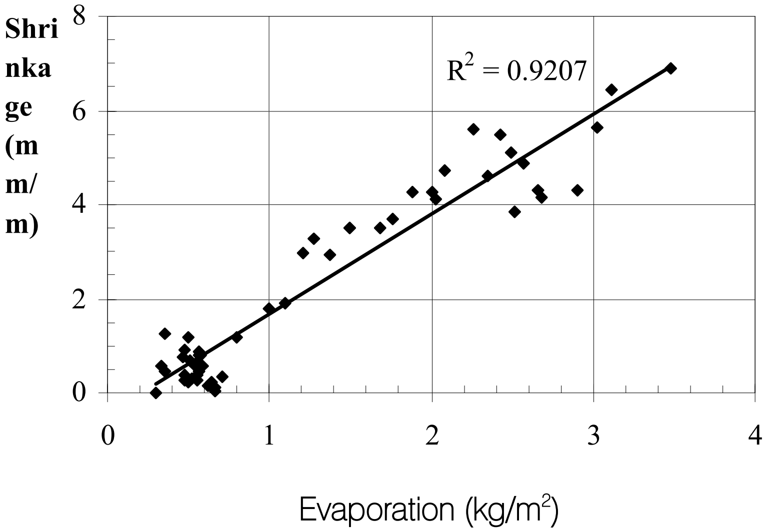


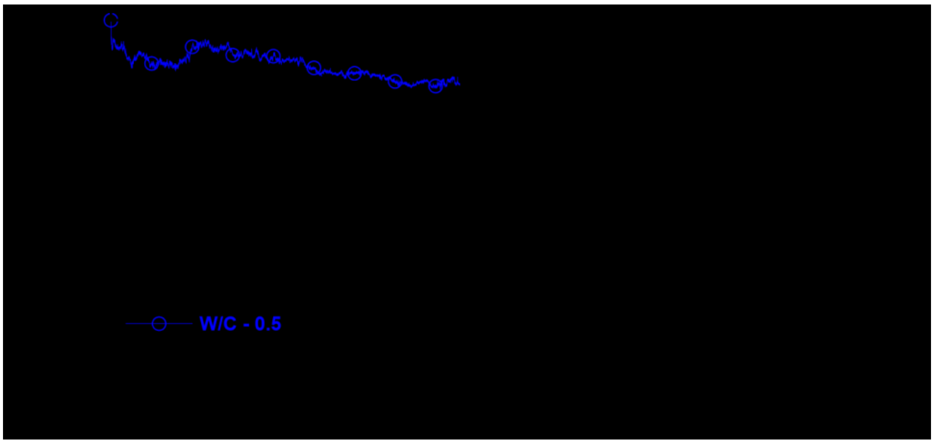
FIGURE 7 Early age shrinkage dependence on evaporation prior to setting for normal strength concretes (Holt and Leivo, 2000). [1 mm/m = 0.001 in./in., 1 kg/m^2 = 0.2 lb/ft^2.]

Recent evidence has shown that the potential for early-age cracking may increase with lower w/c concretes (Figure 8). This phenomenon termed as autogeneous shrinkage refers to the loss of moisture from the paste to allow hydration in a mix with low w/cm. Auto genius shrinkage increases dramatically when the w/c is reduced (below ~0.42). Figure 8a illustrates that a high amount of shrinkage occurs before initial set (3 h for the w/c = 0.3 mixture and 5 h for the w/c = 0.5 mixture), a slight expansion between initial and final set, and continued shrinkage after final set even under sealed condition. Figure 8b also shows that, as one may expect, the mixtures with lower aggregate contents exhibit the greatest shrinkage. This suggests that in addition to the evaporative effects that were described in the previous section, auto genius effects may substantially add to the shrinkage at early ages (in low w/c mixtures) thereby adding to the potential for plastic shrinkage cracking.

Shrinkage in Hardened Concrete

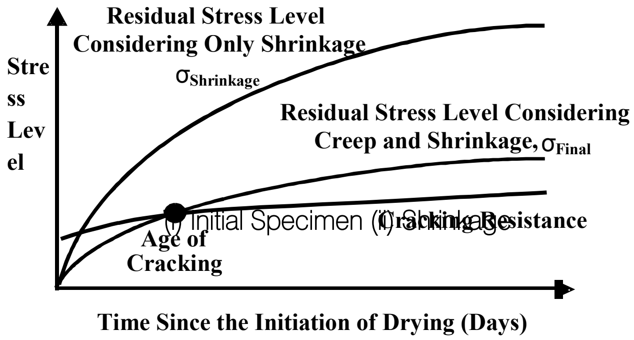
To better understand how volumetric changes of hardened concrete can result in cracking, Figure 9a compares the time dependent strength (cracking resistance) development with the time dependent residual stresses that develop. As a first analysis it can be argued that if strength and residual stress development are plotted as shown in Figure 9a, the specimen can be expected to crack when these two lines intersect. Similarly, it follows that if strength of the concrete is always greater than the developed stresses, no visible cracking will occur.

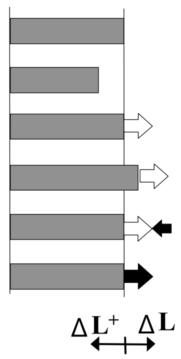
The residual stress that develops in concrete as a result of restraint may sometimes be difficult to quantify. This residual stress cannot be computed directly by multiplying the free shrinkage by the elastic modulus (i.e., Hooke's Law) since stress relaxation occurs. Stress relaxation is similar to creep, however while creep can be thought of as the time dependent deformation due to sustained load, stress relaxation is a term used to describe the reduction in stress under constant deformation. This reduction in stress is described in Figure 9b in which a



(a) (b)

FIGURE 8 Early age shrinkage dependence on (*a*) water to cement ratio in a mortar with 45% aggregate and (*b*) aggregate volume (Pease et al., 2004).





(a) (b)

FIGURE 9 (*a*) Stress development and (*b*) conceptual description of relaxation.

specimen of original length (i) is exposed to drying and a uniform shrinkage strain develops across the cross section. If the specimen is unrestrained, the applied shrinkage would cause the specimen to undergo a change in length (shrinkage) of ΔL^+ (ii). To maintain the condition of perfect restraint (i.e., no length change) a fictitious load can be envisioned to be applied (iii). However, it should be noted that if the specimen was free to displace under this fictitious loading the length of the specimen would increase (due to creep) by an amount ΔL^- (iv). Again, to maintain perfect restraint (i.e., no length change) an opposing fictitious stress is applied (v) resulting in an overall reduction in shrinkage stress (vi). This illustrates that creep can play a very significant role in determining the magnitude of stresses that develop at early ages and has been estimated to relax the stresses by 30% to 70%.

Although free shrinkage measurements are useful in comparing different mixture compositions, they do not provide sufficient information to determine if concrete will crack in service. Shrinkage cracking is dependent on several factors including the free shrinkage (rate and magnitude), time dependent material property development, stress relaxation (creep), strength, structural geometry, and the degree of structural restraint (Weiss, 1999).

An example of stress development as a result of volumetric change is the cracking analysis of reinforced concrete bridge barriers. In principle the barrier top is free to shrink and the bottom has to retain its geometry. If a barrier segment is subjected to uniform shrinkage the stress development shown in Figure 10 is observed. Cracks are expected to form near the barrier base where there is sufficient strain in a direction normal to the barrier base. In order to prevent cracking, the barrier segment length needs to be reduced such that the stress that develops near the base remains below cracking strength. Analytical studies and field observations indicate that barriers crack at a spacing equal to 1 to 2 times the barrier height. In that case to prevent any cracking barrier segments need to be cast at a length equal to its height (Aktan and Attanayake, 2004).

Thermal Contraction

Concrete temperature rises during the initial hydration and curing process due to the heat of hydration of cement. Temperatures typically peak after approximately 18 h; however, the temperature peaks depend on a number of factors, including the solar radiation and any application of an impermeable curing membrane. Subsequently, the hardened concrete begins to cool to the ambient temperature. This cooling process results in thermal shrinkage of the material which, like drying and auto genius shrinkage, can result in the development of residual stress. For example, during the cooling of bridge decks the longitudinal beams restrain the deck contraction. The magnitude of restrained thermal contraction in the deck depends primarily on

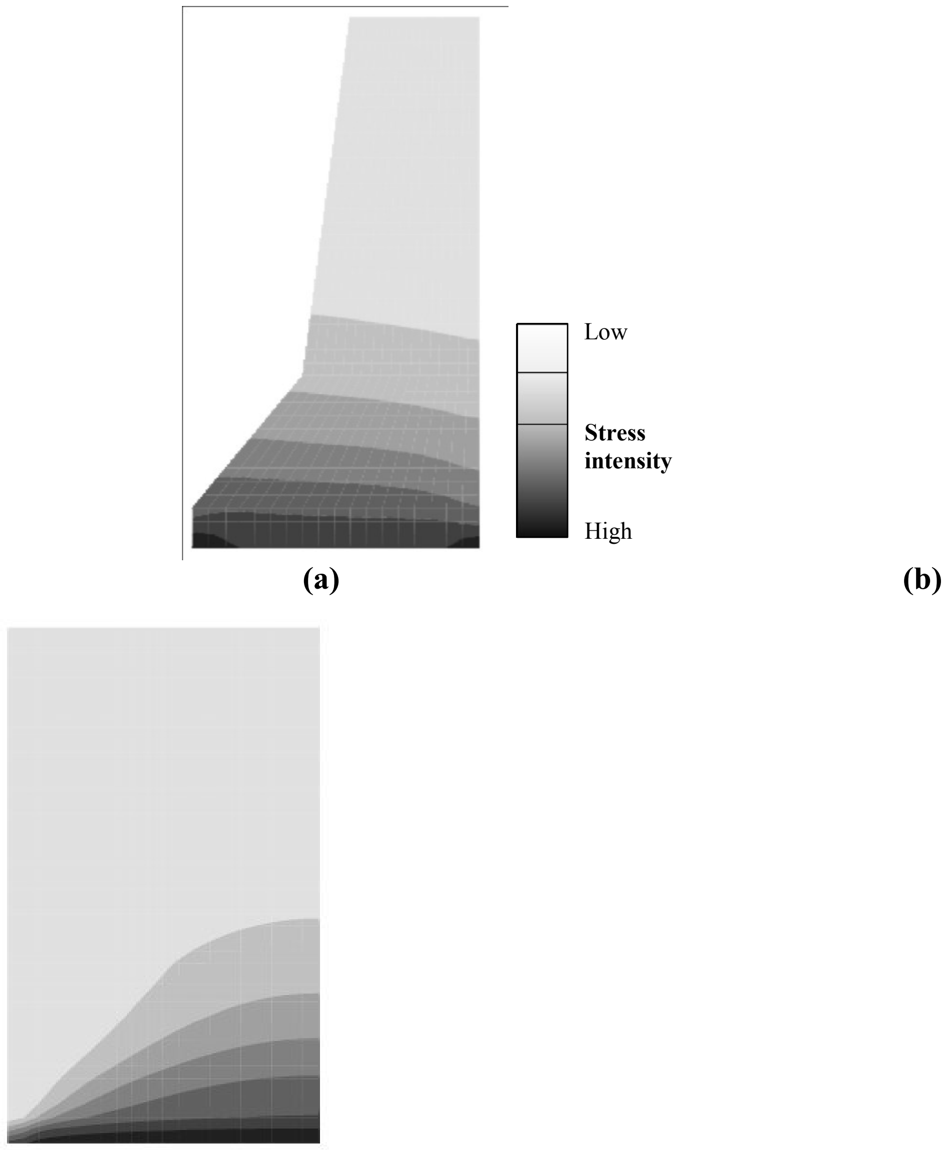


FIGURE 10 Longitudinal stress distribution along an RC barrier under uniform shrinkage: (*a*) height, and (*b*) length.

the difference between the peak concrete temperature and the corresponding temperature of the supporting beams. The same principles apply to pavement thermal expansions and contractions. The underlying base acts to restrain the concrete that might possibly be exposed to increased temperature from hydration and casting on hot days.

Unlike deck drying shrinkage which may take over a year, thermal shrinkage affects the concrete in a short period of time (a few days); thus concrete cannot creep and mitigate cracking. As a result, the restrained shrinkage required to trigger cracking will be less than that required to trigger cracking under drying shrinkage (Babaei and Purvis, 1995b). For typical concrete bridge decks cured under the normal weather conditions, the amount of restrained thermal contraction is usually in the order of 150 microstrain or less (Babaei and Purvis, 1995b). As long as the contraction is less than the threshold of approximately 225 micro strain, cracking is not expected. However, the thermal contraction is later superimposed on the drying shrinkage that may be large enough to cause cracking.

Krauss and Rogalla (1996) provided a system of equations to calculate the restraint in a composite reinforced concrete bridge deck subjected to uniform and linear temperature distributions. The equations consider multiple layers of reinforcement in the deck to account for the restraint effects of longitudinal deck reinforcement and stay-in-place (metal) forms. These equations are based on basic mechanics principles, and only consider the decks with simply supported girders.

ENVIRONMENTAL LOADING AND DURABILITY

Freezing and Thawing

Both laboratory and field experience have shown that properly air-entrained concrete with sufficient strength demonstrates resistance to cycles of freezing and thawing (PCA 2002). However, under extreme conditions even good quality concrete may suffer damage from cyclic freezing, e.g., if it is kept in a state of critical saturation or it interacts with other mechanisms of deteriorations such as distress caused by load.

Transportation structures are susceptible to freezing and thawing cycles that can cause internal cracking. It is commonly accepted that there are two basic forms of deterioration induced by freezing and thawing: internal cracking due to freezing and thawing cycles, and surface scaling, generally due to freezing in the presence of deicer salts. Freezing of water to ice and the accompanying expansion causes deterioration either of the hardened paste, aggregate, or both. Hydraulic and osmotic pressure develop in the pores when water freezes and expands. Water migrates to locations where it can freeze and ice develops in cracks and crevices that act to pry the cracks open wider. The magnitude of the pressure depends on the rate of freezing, degree of saturation, permeability of the concrete, and the length of the flow path to the nearest place for the water to escape. Concrete should be resistant to damage from freeze-thaw cycles if the concrete has gained sufficient compressive strength [approximately 27 MPa 94000 psi)], has approximately 9% entrained air by volume of mortar (ACI 201), has a spacing factor of less than 0.2 mm (0.008 in.), and has entrained air bubbles with a specific surface greater than 23.6 mm^2/mm^3 (600 $in.^2/in.^3$) (ACI 212).

Systems of vertical cracks visible on the surface and closely spaced at joints and pavement edges are called D-cracking as shown in Figure 11. D-Cracking occurs in concrete pavements near joints since the concrete is most likely to be water saturated. When the surface

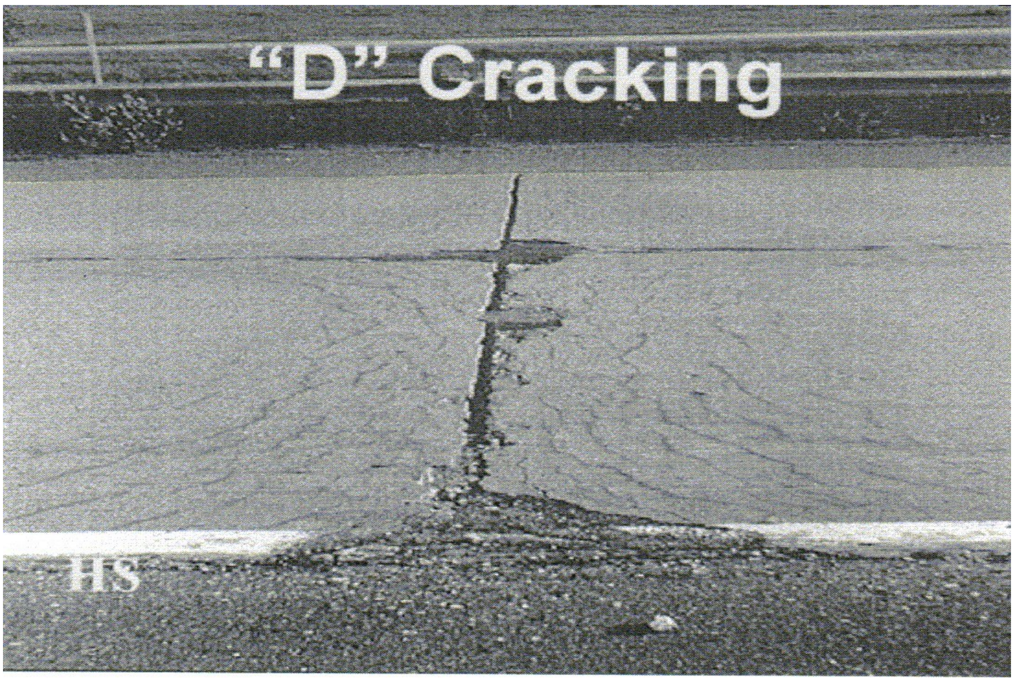


FIGURE 11 D-cracking of a pavement by freezing and thawing. Cyclic loading by traffic accelerated the deterioration.

shows D-cracking, the underlying concrete may likely be severely deteriorated. D-cracking occurs as a result of internal stress as discussed above but initiating within the non-durable aggregate.

Corrosion

Typically, reinforcing steel in concrete is protected from corrosion by the high pH of the pore water solution caused by the calcium hydroxide and the soluble alkalis (Bentur et al., 1997). Under these high pH levels, generally higher than 12.5, corrosion is resisted by the development of a passive layer of ferric oxide that develops on the reinforcing steel. This passive layer prevents corrosion from occurring. Carbonation and/or the ingress of chloride ions lead to lowering of the pH and the development of active corrosion. For corrosion to initiate, moisture, oxygen, and an electrolyte must be present. Corrosion is most deleterious in situations where the concrete is exposed to wetting and drying cycles. The corrosion products are expansive in nature and effectively cause a tensile pressure around the reinforcing steel. Once sufficient corrosion has occurred, splitting cracks typically develop and a loss of bond is observed. The thickness of corrosion products required to cause cracking is proportional to the cover thickness. For concrete with a cover thickness of 40 mm (1.6 in.) a corrosion product thickness of 50 μm (0.002 in.) is typically sufficient to cause cracking. These cracks frequently propagate to the surface resulting in concrete spalling or loss of bond. Recent research has illustrated that preexisting cracks can accelerate corrosion initiation and propagation while sustained load further accelerates corrosion and can lead to creep (Marcotte and Hansson, 2003; Yoon et al., 2002).

Alkali–Aggregate Reaction

Alkali–aggregate reactivity (AAR), as shown in Figure 12, is caused by certain aggregates reacting with alkalis from within the concrete or from outside sources, such as deicing salts, ground water, and sea-water. If the aggregates are siliceous, AAR is known as alkali silica reactivity (ASR) while if the aggregates are dolomitic carbonate rocks, it is known as alkalicarbonate reactivity (ACR), as shown in Figure 13. As a result of these reactions, expansion occurs, leading to longitudinal, map or pattern cracking, spalls at the joints, and overall deterioration.

The chemical reaction between soluble silica in the aggregates and the soluble alkali produces an alkali-silica gel that swells when external water is absorbed (Stark, 1980). The swelling of the gel may crack the concrete. Alternatively, cracks already present form thermal, shrinkage, freeze thaw deterioration, or loading effects can be filled with the gel, thereby inhibiting them from closing and causing even more cracking. The reactivity of aggregates varies. Aggregates containing opal, natural volcanic glasses, chalcedony (a variety of quartz present in chert), tridymite, and crystobalite react rapidly, whereas those containing strained and microcystalline quartz react slowly. The size and amount of reactive aggregates also play important roles in reactivity (Stanton in the 1940s).

Early prevention methods had specified a total equivalent alkali content of cement below 0.60 percent to inhibit destructive expansion, but this limit does not provide the needed protection in all cases (Stark, 1980). Although possible with very rich mixes, ASR has not been evident when a limit of 0.40% was used (Tuthill, 1982). When the soluble alkalis achieve normality above about 0.6 times the normality in the pore water ASR is reasonably assured to occur. This is why specifying the alkalinity of cement does not always control ASR whereas the amount of alkali in the system does have a significant influence on ASR. When high amounts of alkalis are present, pozzolans (Class F fly ash, silica fume, metakaolin, and natural pozzolans),

(a) (b)

FIGURE 12 Alkali aggregate reaction: (*a*) a 355-mm (14-in.) airport pavement at Pease International Airport, Portsmouth, New Hampshire, and (*b*) a bridge end wall on I-95 in Kittery, Maine.

slag cements, or lithium salts are used to inhibit deleterious expansion. The pozzolans or slag are effective because (*a*) they tie up hydroxide ions, preventing the formation of expansive gel; (*b*) reduce the concentration of alkalis to a safe level by replacing portions of portland cement; or (*c*) lower the permeability of concrete, thus preventing the penetration of alkalis from outside sources. Lithium is effective when its concentration exceeds the equivalent alkali ratio of about 0.60 (2/3 is commonly used). Its effectiveness is hypothesized to be the creation of a nonswelling gel. Lithium salts have been shown to retard ASR expansion if ASR is already occurring in structures. Lowering the internal relative humidity of concrete to less than about 80 percent also stops ASR expansion. The elimination of moisture in above ground structures has been tried to extend their service life.

Some argillaceous, dolomitic aggregates can expand upon reacting with alkalis (Newlon and Sherwood, 1964). This is not a widespread phenomenon. Cracking may result from the expansion associated with de-dolomitiization.

Measures recommended to inhibit damaging reactivity are exclusion or dilution of the aggregate by a non-reactive one and use of cements with low alkali content (Newlon and Sherwood, 1964). Pozzolanic materials have been found to be ineffective in reducing ACR in some cases (ACI 201). At present, corrective measures are not available for mitigating this reaction in existing structures.

Sulfate Attack

Concrete may crack due to internal expansion resulting from sulfate attack. This type of deterioration is the result of two chemical reactions: the combination of sulfates with lime to form gypsum, and the combination of sulfates with hydrated calcium aluminates to form ettringite (Lea, 1971). The final reaction product occupies a larger volume than the original constituents. It is also postulated that crystallization of sulfate salts generates stresses that can cause disruption (ACI 201). Some sulfate salts, such as magnesium sulfate, contain cautions that

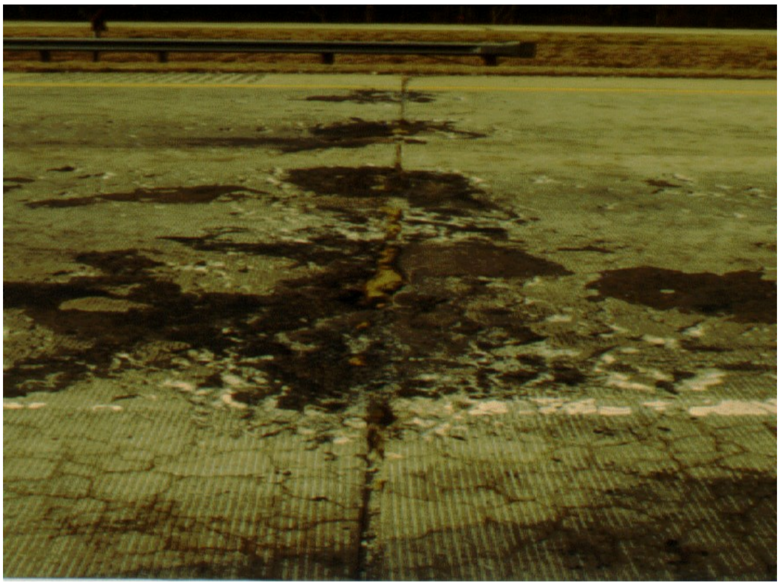


FIGURE 13 Alkali carbonate reaction, I-20 Louisiana.

lead to further expansions thereby exacerbating the effects of sulfate attack. To protect against sulfate attack, cements with low tricalcium aluminate (C_3A) content, pozzolanic materials that react with lime, and low-permeability concretes can be used (ACI 201).

Sometimes a delayed expansion may occur in mature concrete known as delayed ettringite formation (DEF) due to high temperatures (i.e., steam curing) during initial curing. The delayed expansion is generally associated with other deterioration mechanisms, especially ASR (Kosmatka et al., 2002).

Testing and Crack Detection

Several different test methods currently exist that will enable the influence of loading or environment related volumetric changes on the cracking potential of concrete to be determined. In addition, several methods for crack detection are available. In general, it is not possible to make precise predictions for the exact time when a structure will crack, however in many cases a correlation of cracking potential and a strength parameter, determined using a particular test method, does exist. The following section outlines the primary test methods that are available.

MECHANICAL LOADING

Static Loading

Several standard tests exist to determine the mechanical response of concrete. Specimens are tested in compression in accordance with ASTM C 39 to determine the peak strength or ASTM C 469 to determine the static elastic modulus. No standard test currently exists to assess direct tensile strength, however the flexural strength (ASTM C 78) or splitting tensile strength test of concrete (ASTM C 496) can be used as an estimate of tensile strength. It is generally agreed that the flexural strength is approximately 20% higher than the direct tensile strength. Additionally, ASTM C 1018 is commonly used to test the flexural toughness and first crack strength of fiber reinforced concrete. No standard currently exists in North America to assess the non-linear fracture properties of concrete; however some standards have been proposed by RILEM.

Cyclic Loading (Fatigue)

Currently no standard test method exists to determine the fatigue behavior of plain or reinforced concrete. Fatigue tests have been conducted in pure compression, tension or in bending. The fatigue behavior of concrete structures is in general a function of the magnitude of applied loads relative to the strength of the concrete sample. Tests have also demonstrated that the fatigue resistance is affected by stress range and loading frequency and to a certain extent by load history. For a summary of recent test methods the reader is directed to ACI 215.

VOLUMETRIC STABILITY

Settlement and Plastic Shrinkage Cracking

While various approaches have been used to assess plastic shrinkage cracking (Rodecea, 1990; Berke and Dalliare, 1994; Hammer, 1998; Schaels and Hover, 1988), no standard test method currently exists to quantify the potential for plastic shrinkage cracking. Many studies have chosen to adopt procedures that are similar to that proposed by Berke and Dalliare (Berke and Dalliare, 1994, Qi et al., 2003). The salient feature of this restrained slab geometry is that sufficient restraint is provided at the base of the slab by the base obstacles. Cracking is expected to occur above the stress riser and this cracking will combine effects of drying and settlement that may be similar to what occurs above reinforcing steel (Qi et al., 2005a).

Drying Shrinkage

To assess the free shrinkage of concrete ASTM C 157 can be used for specimens made in the laboratory or ASTM C 341 can be used for drilled or sawed specimens to measure the time dependent length change of square prisms. It should be noted however that free shrinkage alone is not sufficient to determine whether restrained cracking can be expected to occur (Weiss et al., 1998).

To assess the effect of restraint on the potential for cracking, several recent studies have been conducted in which the specimens were restrained from shrinking freely. Linear test specimens have been developed to either use passive restraint from a fixed steel frame (Springenshcmidt et al., 1985; Kim and Weiss, 2002) or active restraint from a closed-loop system where a tensile specimen is gripped in the testing frame to apply the necessary load so as to maintain no displacement in the specimen (Kovler, 1994; Altoubat and Lange, 1997; Altoubat and Lange, 2001). These specimens generally use flared grips to reduce stress relaxation or cracking at the ends of the specimens (Altoubat and Lange, 2002).

While the linear restrained specimens are preferred for data interpretation, the restrained ring test is frequently used as a simple laboratory test since it removes difficulties associated with providing sufficient end restraint. The ring test consists of a concrete annulus that is cast around a rigid steel core. As the concrete dries it attempts to shrink but this movement is prevented by the inner steel core. The restrained ring test was used as early as 1939 (Carlson and Reading, 1988) to assess the susceptibility of a concrete mixture to early-age cracking. Recently, an AASHTO provisional test standard (AASHTO PP 34) has been developed to provide a comparison of cracking ages for different materials. Similarly ASTM (ASTM C-1581) has been developed with a slightly thinner concrete wall and higher degree of restraint than the AASHTO specimen. The residual stress in the concrete can be calculated directly from the ring (Weiss and Furgeson, 1999; Attiogbe et al., 1997; Hossain et al., 2003) and this residual stress can be compared with the tensile strength to assess how susceptible a material may be to cracking.

Ring specimens use axi-symmetry to simulate an infinitely long slab that is easy to conduct in the laboratory without the difficulties encountered with end conditions of testing tensile specimens. It can be shown that due to the axi-symmetric nature of the specimen, friction between the ring and steel does substantially impact results. Geometry can be selected to reduce non-linear stress distributions in the radial direction by using a sufficient dimension of the radius when compared to the concrete thickness. More recently solutions have been provided to account for the moisture gradients that exist in the ring specimen when it dries from the outer circumference (Hossain et al., 2004; Moon et al., 2004). Due to its simplicity and versatility, the 'ring-test' has become more commonly used over the last decade to assess the potential for shrinkage cracking.

Thermal Expansion–Contraction

The coefficient of thermal expansion can be used to predict strains generated from differential concrete temperatures and from external restraint due to volumetric changes from temperature effects.

AASHTO TP60-00 is a test method to determine the coefficient of thermal expansion of concrete cylinders. Because temperature expansion and contraction values are highly dependent upon moisture content, the 100 mm diameter cylinders are measured for length change in an underwater rig. This rig allows the specimens to be kept moist at all times to provide meaningful test data.

Auto generous Shrinkage

Several test methods have been used to measure auto genius shrinkage, however there is no generally accepted standard used in the US. Some have considered tests similar to those of drying shrinkage (ASTM C157 or C341) however the sides of the specimens are sealed to prevent moisture loss (generally using two layers of aluminum tape). It should be noted that the standard shrinkage tests can neglect shrinkage that occurs prior to the initial test, thereby providing a misleading measure of auto genius shrinkage (Aitcin, 1999; Sant et al., 2006). To overcome this limitation other test methods have been developed. The Japanese Concrete Institute developed a standard test for autogeneous shrinkage in mortar and concrete. A mold has end plates with holes through which gage points can be inserted and embedded enabling shrinkage measurements at early ages beginning with time of setting (Tazawa, 1998). While this procedure is relatively easy to implement, difficulties can exist in removing external restraint and determining the exact time at which measurements should begin. Aiticin and co-workers (1998) demonstrated the use of internal strain gages as a method to measure auto genius shrinkage thereby minimizing the potential complications of determining the time of set. Some have questioned whether the stiffness of the internal gage may influence the magnitude of the measured shrinkage. Other procedures have been used to measure auto genius shrinkage in mortar and cement paste. For example, Boivin et al. (1999) and Hammer et al. (1999) have placed paste in a membrane and suspended this from a scale in a water bath. By measuring the change in buoyancy, the auto genius change in volume could be computed. It has been illustrated (Lura and Jensen, 2005) that if the membrane used for these measurements is not impermeable, substantial errors in the measured auto genius shrinkage may be obtained. Jensen and Hansen (1995) developed a dilatomer for measuring the auto genius shrinkage of paste using a corrugated tube. This method has an advantage of being easily repeated. Sant et al. (2006) demonstrated that the membrane, corrugated tube, and non-contact measurement methods provide results that are consistent with one another.

CRACK DETECTION

Cracks may be either macrocracks, detectable by visual inspection, or micro cracks, which can be detected only with microscopes or non-destructive testing. Another distinction is between discrete cracks, for which each has to be located and counted individually, and distributed fine cracks, for which calculations of an area may be more important.

Discrete Crack Detection

To find an alternative to the detection of individual cracks by visual inspection, a significant amount of effort has gone into development of automated analysis software for pattern

recognition of cracks in digital images (Koutsopoulos and El Sanhouri, 1991). In earlier work, the digital images were obtained by scanning analog photographs. As the resolution, i.e. number of pixels, of digital cameras has improved the practice is now to take direct digital images of the area under investigation. This reduces the work involved and avoids the image degradation introduced by the scanning process.

In the image analysis process, the software examines each black pixel and its neighbors to decide if it belongs to a given crack. When a crack is detected, it is then characterized by a set of parameters including location, length, width and direction (Mahler and Kharoufa, 1990). There are two major considerations in the sensitivity of this process: one is the probability of detection and the other is the probability of false positives. An algorithm with a low probability of detection will miss a significant number of cracks. An algorithm with a high number of false positives may detect a high percentage of actual cracks, but may also mistake other features for cracks.

After a crack has been detected and characterized, it may then be assigned to a particular class. Several classification systems have been proposed for particular applications (Koutsopoulos and El Sanhouri, 1991; Ritchie et al., 1991). It is important to distinguish between systems that are simply descriptive, and those that are diagnostic, i.e. those that assign causes to each crack. The problem with diagnostic classifications is that more than one cause of damage may produce the same crack appearance.

Microcrack Measurement Techniques

Conventional methods for measuring microcracks include optical microscopy, scanning electron microscopy and radiography. These have been reviewed by Slate and Hover (1984). They are all destructive, requiring the drilling of cores from the concrete followed by sectioning of the specimens, and the results are two-dimensional. More recently three-dimensional methods using computed tomography based on conventional X-ray or synchrotron radiation have been introduced. These can image entire specimens. The true crack area can be measured, rather than its two-dimensional projection. However, the overall size of the specimen is limited to less than 100 mm (4 in.) in thickness for useful resolution. Moreover, these cannot be applied in the field.

Ultrasonic’s

Other methods for measuring micro cracks are based on ultrasonics (Kesner et al., 1998; Jacobs and Whitcomb, 1997). These methods do not count individual cracks, but rather measure a bulk ultrasonic property of the concrete, usually attenuation. This can then be calibrated against radiographs to give micro crack density (Kesner et al., 1998). Ultrasonic methods offer the possibility of making measurements in the field on real structures. Their drawback is that features other than micro cracks in the concrete can contribute to attenuation.

Acoustic Emission

Acoustic emission describes a field of testing that has been popular recently in crack detection because of its non-invasive nature (Ouyang and Shah, 1991; Ohtsu, 1994; Ohtsu, 1996). Recent research has indicated that it is possible to quantify cracking using acoustic emission. The sensors detect acoustic activity when the specimen undergoes cracking, and they are amplified.

Applying threshold levels to the activities helps in detecting events produced by cracking as well as background noise (Puri and Weiss, in press). While some applications of acoustic emission have been performed in the field, majority of the applications have been performed in the laboratory.

Control of Cracking

Control of cracking in concrete Road

Long-term exposure and loading increase the magnitude of cracks, principally their width, in both reinforced and plain concrete. Microcracks also increase in both sustained and cyclic loading. However, microcracks formed at service load levels do not seem to have a great effect on the strength and serviceability of reinforced and prestressed concrete (ACI 224). ACI 224 presents the reasonable crack widths at the tensile face of reinforced concrete for typical conditions. However, the values are intended to serve only as a guide.

In the United States and Europe, equations are given in codes to limit service-load cracking. Ensuring acceptable cracking at service loading depends on proper detailing, such as provisions of minimum reinforcement, proper selection of bar diameters, bar spacing, and reduction of restraint (ACI 224). Nawy has demonstrated that as spacing is decreased through the use of a larger number of bars, a larger number of narrower cracks are formed. As the crack width becomes narrow enough within tolerable values, corrosion effects are reduced considerably (Nawy, 2001).

In recent years there has been an increasing awareness of cracking in bridge decks. Bridge deck cracking has been recognized as a major and costly problem for highway structures in that it often accelerates corrosion, increases maintenance costs, and shortens the service life of the deck. Several factors are known to affect deck cracking including bridge design, concrete mixture design, mixture materials, and placing, finishing and curing practices. Studies have shown that the primary source of deck cracking is attributed to a combination of shrinkage (plastic, auto genius , and drying) and thermal stresses, which are influenced by such factors as bridge design, concrete mixture design, material properties, environmental conditions, and construction practices.

Design Factors

design-related factors can have a substantial affect on deck cracking. Girder type, size and spacing are all known to be influential. For example, steel girders can create conditions more conducive to deck cracking as opposed to concrete girders that are stiffer. Also of significance, but to a lesser degree, is the size and spacing of bridge girders. Larger sized girders placed at closer spacings tend to induce greater residual stresses (when shrinkage and thermal strains are restrained) in decks and therefore increase the potential for cracking (Krauss and Rogolla, 1996).

Concerning deck thickness, thinner decks tend to promote higher stresses and are expected to exhibit increased cracking. Bridge decks constructed with increased thickness experience less shrinkage and thermal stresses, therefore, decreased cracking. It should be noted that this correlation can be affected by girder type, size, and its compatibility with the deck, which could then result in inconsistent effects on cracking (Krauss and Rogolla, 1996). Settlement cracking in decks, at the reinforcing bar locations, due to settlement of the concrete during the plastic stage, is influenced by the amount of cover over reinforcement. Increasing concrete cover over reinforcing bars should reduce the occurrence of settlement cracking. Furthermore, tests on the corrosion rate of concretes exposed to plastic shrinkage and settlement conditions showed a substantial increase in the time to corrosion initiation when the cover was

increased (Qi et al., 2005). Incidental issues such as leaking joints and plugged drains facilitate the saturation of bridge members by salt solution which makes them prone to chemical reactions and damage from cycles of freezing and thawing, resulting in undesirable cracking. Differential settlement of false-work for multiple span cast-in-place structures is also critical, and allowable false-work deflection can be calculated and specified on the plans.

Materials Selection and Proportioning

The role of concrete materials selection and proportioning and its influence on deck cracking cannot be emphasized enough. Mixtures with a high water-to-cement ratio (w/c) i.e., > 0.45 tend to have a relatively high porosity and can exhibit substantial drying shrinkage and reduced protection of the reinforcing steel from chlorides. This has led many to use mixtures with low w/c. Recent work however has illustrated that the propensity for cracking can increase as the strength of concrete increases and especially if insufficiently cured. This is due to the following five factors

1. Early-age auto genius shrinkage,
2. Higher material stiffness,
3. Increased brittleness,
4. Reduced creep, and
5. Increased shrinkage rate (Weiss et al. 1999).

Low w/c concrete also bleeds less and is therefore more susceptible to plastic shrinkage cracking. Some researchers have indicated that extended moist curing increases the modulus of elasticity and reduces the creep making the concrete more prone to cracking (Burrows, 1998). Auto genius shrinkage (shrinkage without water loss or temperature change) increases with decreasing w/c (when below 0.42) and can be quite substantial since significant strains can be measured before the concrete reaches an age of 24 hours. It should also be noted that sealing the concrete to prevent moisture loss is not sufficient to prevent auto genius shrinkage. Best results have been achieved when the w/c is targeted in the range of 0.38 to 0.44.

Concrete mixtures made using higher cement contents are very conducive to cracking by producing higher heat of hydration, greater shrinkage, higher modulus of elasticity, and lower creep. Frequent use of high strength concretes in the construction industry tends to encourage increased cement contents increasing the cost of the mixture and increasing cracking. With proper planning during materials selection and mixture proportioning, a crack-resistant concrete having lower cement content, which still meets durability and performance specifications, can be produced.

Cements

Controlling initial concrete temperatures and peak temperatures during hydration reduces thermal stresses and subsequent cracking. Furthermore it should be noted that the source of cement may have a large effect on drying shrinkage (Babaei and Purvis, 1995a; Burrows, 1998; Chariton and Weiss, 2002). Cements with high alkali content, high C_3S and C_3A contents, low C_4AF, and high fineness have high strength gain and are found to have higher cracking tendencies (Burrows, 1998: Jeunger and Jennings, 2002). Type III cements are therefore used with caution for deck applications. In an effort to control temperatures, Type II or Type IV cements, because of their low heat of hydration, often considered in lieu of Type I, especially when warmer ambient conditions exist. It has been shown in one study that when Type II cement replaced Type I cement (same source), temperature rise decreased from 9°C to 6°C (16°F to 11°F) and drying shrinkage decreased from 488 microstrain to 367 microstrain (i.e., 25% decrease) (Babaei and Purvis, 1995a). The slowest-setting cement can be expected to have reduced drying shrinkage and cracking.

Supplementary Cementitious Material

Supplementary cementitious materials, such as fly ash, slag, and silica fume, are frequently used in mixtures to enhance early and long-term performance characteristics. Fly ash and slag typically reduce the rate of strength gain, lower the heat of hydration, reduce the rate of stiffness development and thereby typically reduce the potential for cracking. Silica fume can increase the rate of strength development, increase the heat of hydration, reduce bleeding, and create conditions that are favorable for cracking. Some strongly discourage the use of silica fume in bridge deck applications; however others have reported that silica fume is not a cause of premature cracking.

Water Content

As the water content in the mixture increases the drying shrinkage is expected to increase. ACI 224 Report (ACI 224R) shows that for a typical concrete specimen, 134 kg/m^3 (225 lb/yd^3) water content results in about 300 microstrain drying shrinkage. The drying shrinkage increases at a rate of about 30 microstrain per 5.9 kg/m^3 (10 lb/yd^3) increase in water content. However a study was performed that included 12 bridges in Pennsylvania with crack intensities ranging from none to 87 m/100m^2 (265 ft/1,000 ft^2) with mixture water contents varying from 158 to 173 kg/m^3 (267 to 292 lb/yd^3) (Babaei and Purvis, 1995a). The results of this study indicated that an increase in water content increases the drying shrinkage by approximately 75 microstrain, indicating that mix water content alone was not the prime cause of the significant difference in the performance of the bridge decks with respect to transverse cracking.

Aggregates

Both aggregate quantity and quality should be carefully examined when designing a crackresistant deck mixture. Increasing aggregate content will allow a reduction in the paste content while reducing the mixture component that is most susceptible to shrinkage and thermal stresses. Because less cement is required, a mixture with reduced cement paste content provides for a more economic mixture. In addition, increasing the maximum size of the aggregate tends to increase the volume of aggregate that can be used. Therefore, aggregate of the largest size possible is usually used (provided the aggregate is not reactive or prone to freezing and thawing problems) with the aggregate grading optimized (well graded). This allows mix workability to be maintained with lower paste content, creating less potential for stresses and cracking to occur. Most recommendations specify aggregate at a 38-mm (1½-in.) maximum size or the smaller of one-third the deck thickness or three-fourths the minimum clear spacing between reinforcing bars (Krauss and Rogolla, 1996).

Absorption of an aggregate (coarse and fine) is closely related to its porosity, and the porosity influences the stiffness and compressibility. Generally, concretes made with high absorption aggregates tend to be more compressible, and thus yield higher shrinkages. Also, aggregates with high absorption may themselves shrink an appreciable amount upon drying. Soft fine aggregates contribute to drying shrinkage, but not as much as soft coarse aggregates. Based on the information provided in ACI 224 Report (ACI 224R), drying shrinkage can increase from 320 microstrain to 1,160 microstrain (about 250% increase) when the aggregate absorption is increased from 0.3% to 5.0%. Quartz, limestone, dolomite, granite, feldspar, and some basalt are generally classified as low shrinkage producing aggregates. On the other hand, sandstone, slate, trap rock, and some types of basalt often produce high shrinkage concretes. Aggregate restraint potentially has an important role in the performance of the bridge decks with respect to transverse cracking.

Admixtures

Depending upon the type, admixtures provide a means of improving the workability, placement, and performance of a concrete mixture. Admixtures can have both a positive and negative effect on deck cracking. When designing a mixture, one should always be familiar with the admixture type and its compatibility with other mix constituents in an effort to avoid unexpected cracking.

Water-reducers have been found to be desirable since they enable a reduction in mix water and paste content while still maintaining mixture workability, and thus, minimize drying shrinkage and cracking. Retarders are often used in bridge deck applications to allow for continuous placement of the deck concrete. Retarders offer a delayed set time, which then aids in placement and makes the concrete less susceptible to cracking due to deflection of the formwork during the placement. This also results in lower temperatures during hydration and helps control thermal stresses. On the other hand, with an extended set time, a mix with a retarder runs the risk of plastic shrinkage cracking. Another chemical admixture that has recently entered the construction markets is shrinkage reducing admixtures (SRAs) (Nmai et al., 1998; Shah et al., 1998). This chemical works by reducing the surface tension of the pore water and thus lowering plastic (Lura et al., 2006) and long-term shrinkage (Weiss and Berke, 2002; Pease et al., 2005).

Fiber Reinforcement

Due to the low tensile strength and fracture toughness of cementitious materials fiber reinforcement has been suggested as an effective method to mitigate early-age cracking in concrete (Ramakrishnan and Coyle, 1983; Balaguru and Shah, 1985; Gryzbowski and Shah, 1990; Gopalaratnam et al., 1991; Shah et al., 2004). Fibers increase the toughness of concrete (Gopalaratnam et al., 1991) which manifests itself in a reduction in the crack width in restrained concrete. Higher volumes of fibers have been shown to be particularly useful in delaying the time to cracking, transferring stress across a crack, and reducing the width of crack (Shah et al., 2000; Shah et al., 2004).

Construction Practices

To reduce the potential for plastic shrinkage cracking in bridge decks it has been found to be critical to limit the water evaporation from fresh concrete by proper construction techniques. This is best achieved by proper curing during the early hours immediately after concrete placement that reduces the rate of evaporation. Ideally, fresh concrete of a bridge deck should have no water evaporation from the concrete surface from the time of mixing for at least 24 h. The rate of evaporation of water from plastic concrete is a function of relative humidity, air temperature, air speed, and temperature of the placed concrete. A nomograph, developed many years ago by Lerch at PCA, is commonly used to gage how fast the evaporation rate will be to provide an indication for cracking potential (ACI 305). The use of wind breakers, controlling ambient temperature by shielding the fresh concrete from solar radiations etc are some of the measures that can be adopted to restrict evaporation loss. See ACI 308R, Guide to Curing Concrete, for more information.

Cracking Reduction Designs

Proper joint spacing in plain concrete pavements and the proper amount of steel in the continuously reinforced concrete pavements are essential. Sufficient thickness and proper drainage are important.

Material Selection and Proportioning

Concrete mixtures made using aggregates with increased stiffness and lower volumes of paste can also be effective in reducing shrinkage (Mindess and Young, 1981). Reducing the potential for cracking through proper mixture design and curing procedures cannot be overemphasized.

Cement

Cements with high alkalinity have higher tendencies of drying shrinkage than others. Minimizing the affect of alkalinity has been found to be a viable solution to preventing ASR. This is accomplished by substituting pozzolanic materials for cement, using cements with low alkali contents and very little salts.

Water

The more water that is available to evaporate from the concrete, the higher the tendency to shrink on drying and the lower the capacity to resist tensile stress. Consequently, the water content of a concrete has the most significant effect on its long-term drying shrinkage. Slip-form pavers tend to limit the use of mixtures with excessive w/c. However, experience has shown it is best not to use mixtures with excessively low w/c since these may exhibit auto genius shrinkage, not to limit the water content to lower slump to achieve zero variation of surface flatness is also not recommended.

Aggregate

Minimizing the amount of cement in a concrete mix will also minimize the amount of shrinkage. It is typically good practice to maximize the amount of aggregate by using the largest size possible (provided the aggregate is non reactive) and utilizing proper aggregate grading. Aggregates containing calcium in general have lower coefficients of thermal expansion and those containing quartz have higher coefficients. Concretes made using aggregates of high stiffness tend to show less cracking and concretes with lower coefficients of thermal expansion should crack less.

Admixtures

Retarders are often used, particularly, when the ambient temperature is expected to reach 24°C
(75°F) or more. Air entraining admixtures do not significantly influence the age of cracking (Krauss and Rogolla, 1995). Water reducers are used because they result in reduction in the amount of mix water and drying shrinkage. Shrinkage reducing admixtures (SRAs) reduce the surface tension of the concrete water and thus lower long-term shrinkage (Weiss and Berke, 2002). While SRAs have been shown to be effective, their cost generally limits their use in conventional full depth pavements though they may be useful for overlays, patches, or white-toppings. Expansive cements have also been used on several projects to minimize the effects of shrinkage cracking, to increase joint spacing, and limit curling (Keith et al., 1996). While this method has been shown to work well, special precautions must be taken including extra curing time, additional steel reinforcement, and stringent reinforcement placement requirements.

Construction Practices

Curing

High temperatures obviously occur during the summer time; however there may be larger temperature differentials between the air and concrete during the fall and spring. Thus, proper curing is needed at all times. The application of windbreaks to eliminate the effect of windy days is not practical on most paving sites, so the contractor is left with no alternative other than to wet cure the concrete, cover it with wet burlap or polyethylene sheet before the bleed water disappears, or apply a curing compound. Plastic cracking can be controlled in most cases by early application of a curing compound. Such early application is a common construction procedure during the placement of pavement concretes. Experience has shown that resin-based curing compounds should not be applied until the concrete has lost its free surface water as noted by the surface sheen, after the bleed water has disappeared. This may or may not be possible depending on the way the contractor has the paving train set up and how the concrete bleeds and dries out in a repeatable or uniform manner.

Saw Cutting

Well-designed contraction joints and appropriately timed joint cutting is essential to relieving early stress development. This is especially true for the case of thin concrete applications to control cracks that may develop. In addition, the use of proper curing practices that minimize evaporative losses or supply additional water can substantially reduce shrinkage.

MORE ABOUT CONTROL OF CRACKING IN OVERLAYS

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 unbounded overlay is generally best where severe cracking is present or where it can later develop in the base slab. Systems that are essentially unbounded have been constructed satisfactorily where the overlay is placed over an asphalt layer. The asphalt itself acts as a deboning 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, spelled, or potholed surfaces, a thin leveling and deboning 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 delimitations 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, delaminating of the overlay, has been found to occur only at cracks in the overlay or at boundaries, normally at very early ages. These delimitations 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 shot blasting, scrabbling, hand chipping, or sandblasting, and hydraulically by high-pressure water blasting (hydro demolition). Scarifying methods that impact the surface can cause cracking in the substrate that can result in delaminating. 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 deboning 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/m3/h (0.2 [lb/ft3] /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 micro cracks 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 cracker resting 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 pre cracking and post cracking 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 post cracking 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 544.2 R; 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 spelling 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 boomed on the surface after the curing and drying period but before traffic is allowed on the overlay. The penetrate 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 delaminating. 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.

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