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Control of Cracking in Concrete

State of the Art

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Control of Cracking in Concrete

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Transportation Research Board
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Foreword

Concrete is a quasi-brittle material with a low capacity for deformation under tensile stress. Mechanical loading, deleterious reactions, and environment loading can result in the development of tensile stresses in concrete. These tensile stresses all too frequently result in cracking that can adversely affect the performance of concrete. However, the potential for cracking can be minimized by appropriate precautions in design, materials and proportions, and construction practices. These precautions will ensure that concrete can be used satisfactorily for an extended period of time without any significant loss of aesthetics, service life, safety, and serviceability.

This circular discusses causes of cracking, testing, and ways of minimizing strains and stresses that can cause cracking in transportation structures: namely bridge structures, pavements, and footings. It is intended for anyone interested in controlling cracking for cost-effective and long-lasting transportation structures.

Many members of the Transportation Research Board's Basic Research and Emerging Technologies Related to Concrete Committee (AFN10), and many friends of the committee made generous contributions in the preparation of this document. Special appreciation is expressed to Jason Weiss and Celik Ozyildirim for their leadership in the development of this document.

—Mohammad Shamim Khan
Professional Services Industries, Inc.

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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.

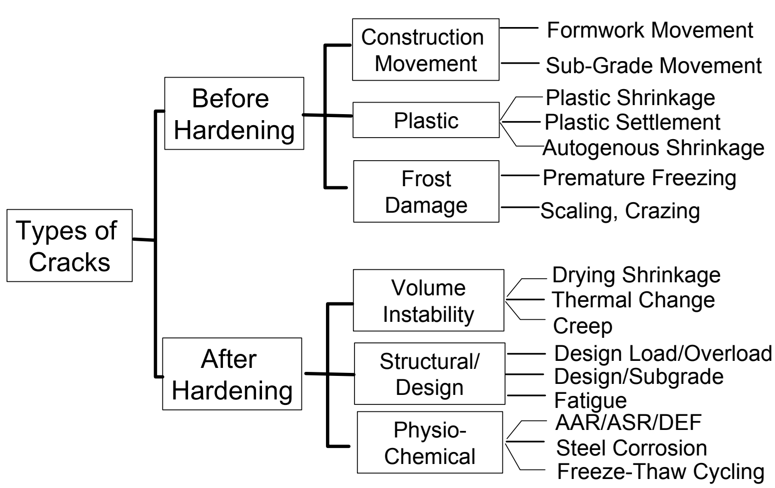

FIGURE 1 Common causes for cracking in concrete dtructures.

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 spacings	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, delamination, and debonding, 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.

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 non-linear. 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

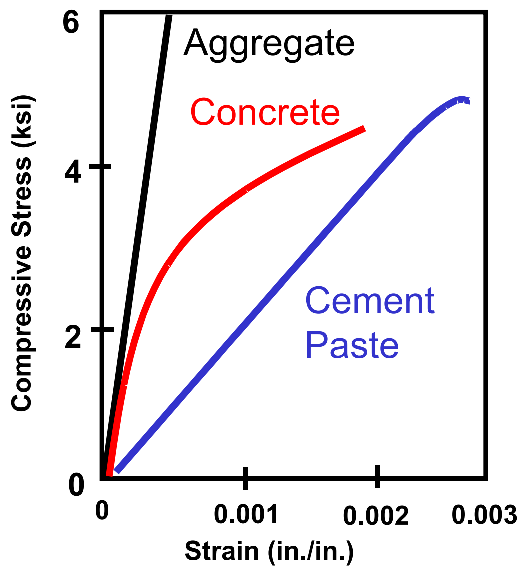

FIGURE 2 Stress strain response of behavior of concrete (1 ksi = 6.89 MPa).

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

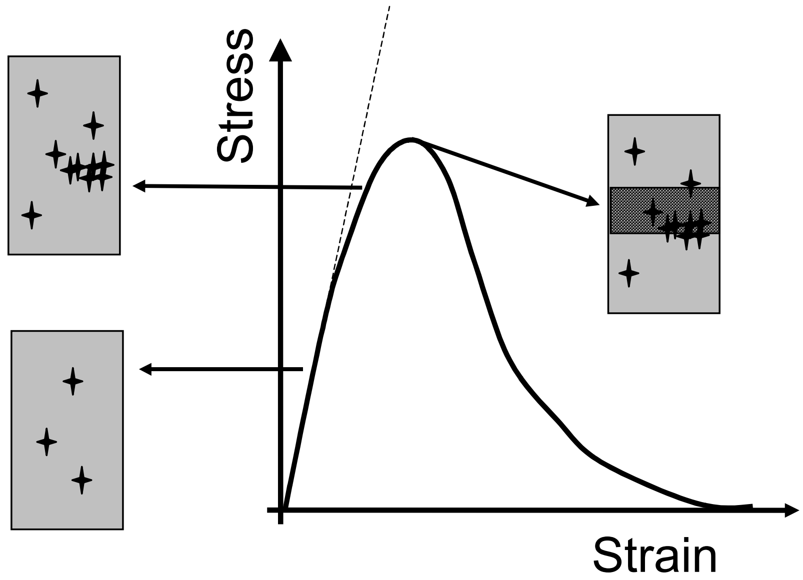

FIGURE 3 Strain softening aggregate and rock.

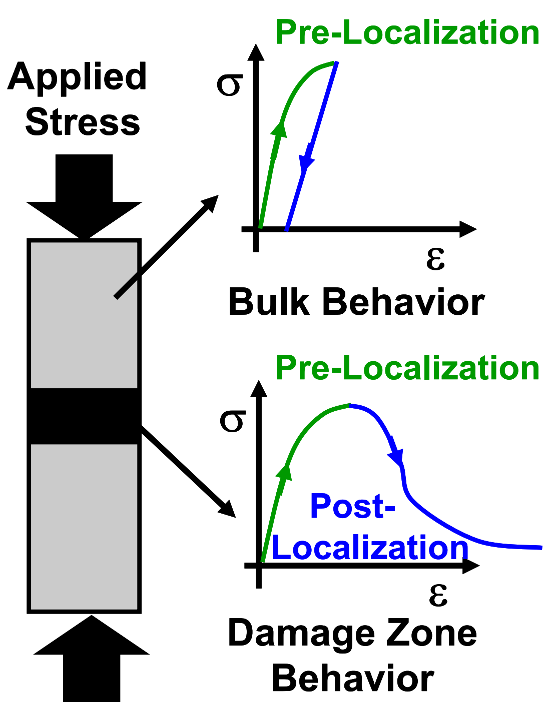

FIGURE 4 Composite model for the response of damaged concrete.

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-

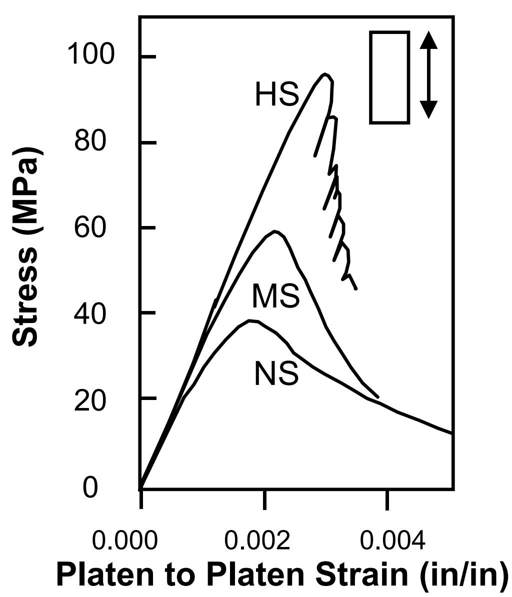

FIGURE 5 Influence of strength on the stress-strain response (1 MPa = 145 psi).

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 autogenous 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.

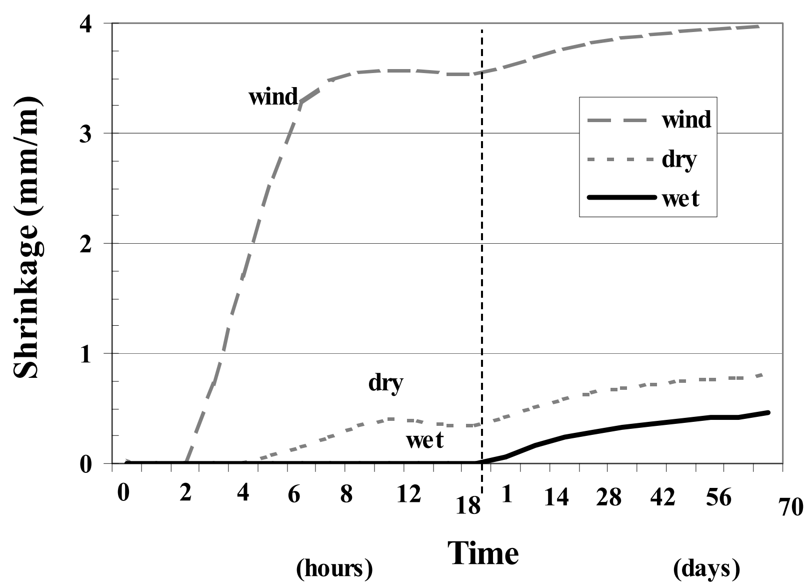

FIGURE 6 Combined early age and long-term shrinkage for three different curing environments (Holt and Leivo, 2000). [1 mm/m = 1,000 µm/m (0.001 in./in.).]

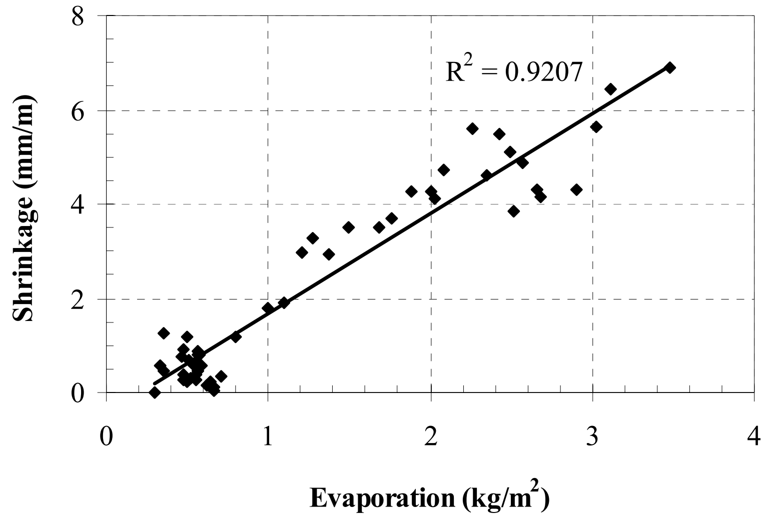

FIGURE 7 Early age shrinkage dependence on evaporation prior to setting for normal strength concretes (Holt and Leivo, 2000). [1 mm/m = 0.001 in./in., 1 kg/m^2 = 0.2 lb/ft^2.]

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. Autogenous 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, autogenous 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

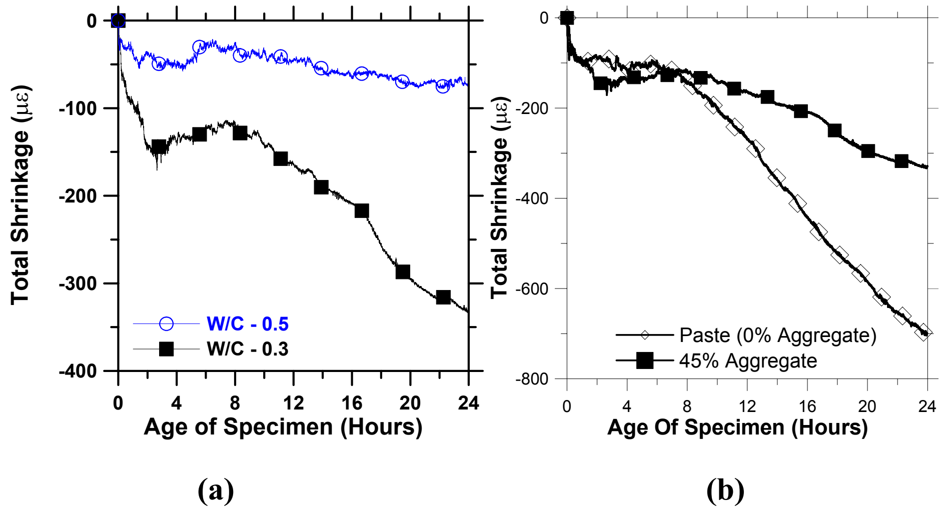

FIGURE 8 Early age shrinkage dependence on (*a*) water to cement ratio in a mortar with 45% aggregate and (*b*) aggregate volume (Pease et al., 2004).

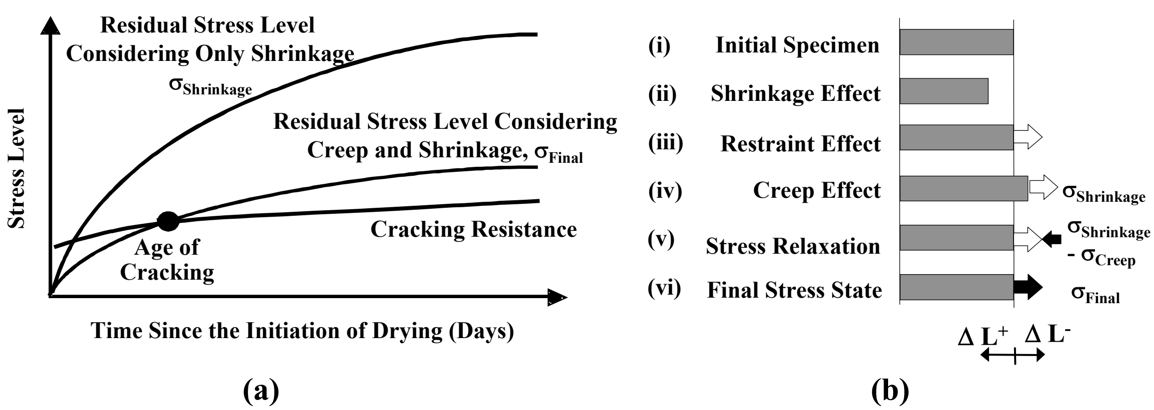

FIGURE 9 (*a*) Stress development and (*b*) conceptual description of relaxation.

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 autogenous 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

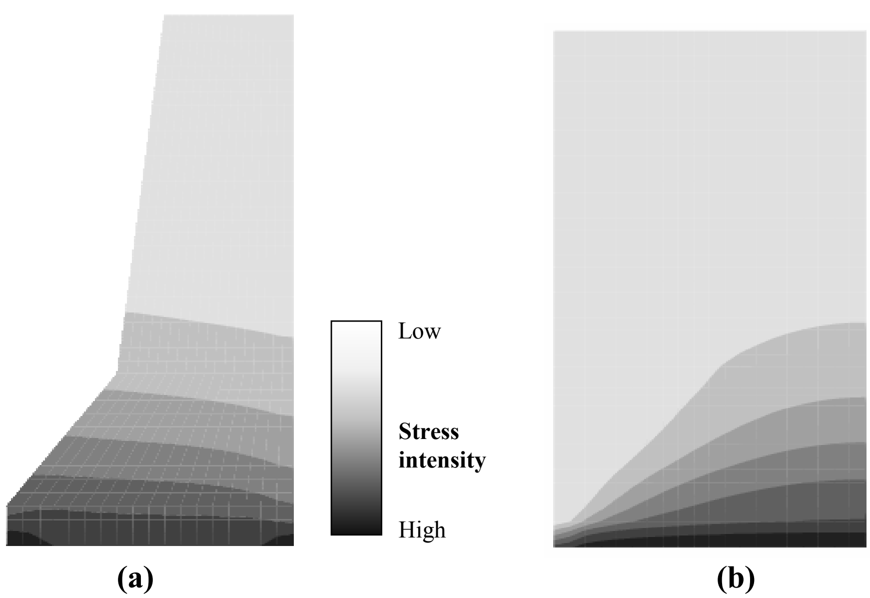

FIGURE 10 Longitudinal stress distribution along an RC barrier under uniform shrinkage: (*a*) height, and (*b*) length.

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 microstrain, 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

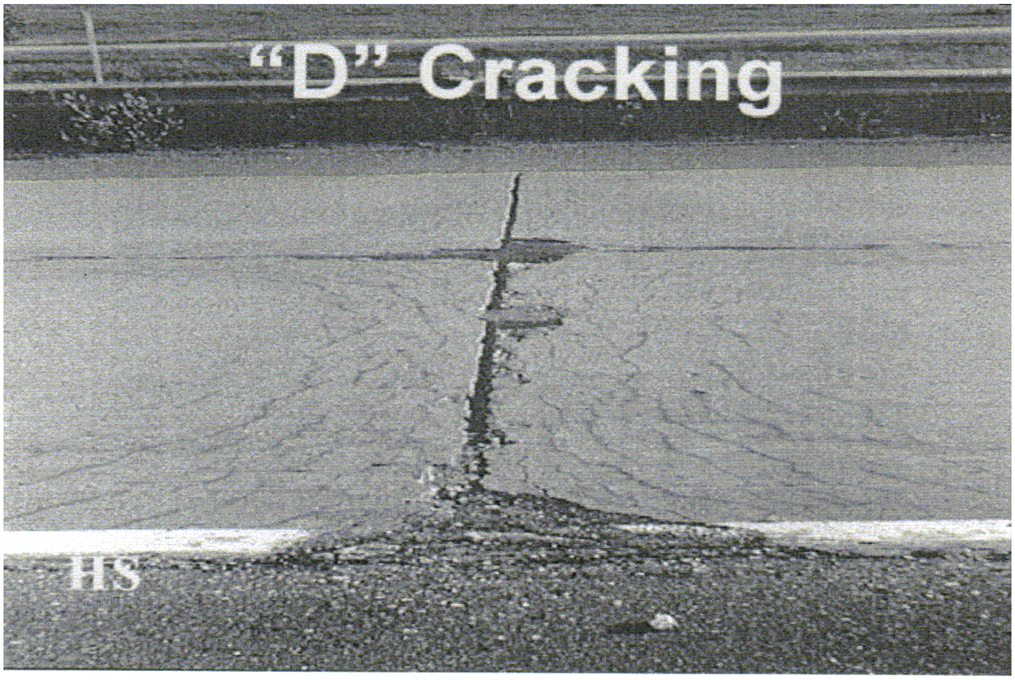

FIGURE 11 D-cracking of a pavement by freezing and thawing. Cyclic loading by traffic accelerated the deterioration.

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 alkali-carbonate 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 non-swelling 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 cations that

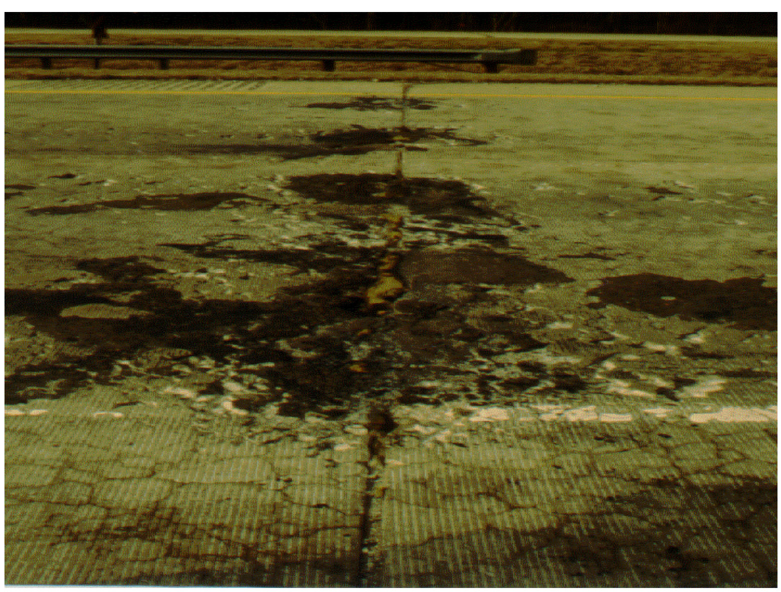

FIGURE 13 Alkali carbonate reaction, I-20 Louisiana.

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.

Autogeneous Shrinkage

Several test methods have been used to measure autogenous 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 autogenous 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 autogenous 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 autogenous 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 autogenous 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 autogenous shrinkage may be obtained. Jensen and Hansen (1995) developed a dilatomer for measuring the autogenous 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 microcracks, 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.

Ultrasonics

Other methods for measuring microcracks 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 microcrack 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 microcracks 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 BRIDGES

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, autogenous, and drying) and thermal stresses, which are influenced by such factors as bridge design, concrete mixture design, material properties, environmental conditions, and construction practices.

Bridge Design Factors

Bridge 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 autogenous 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). Autogenous 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 autogenous 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 crack-resistant 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.

CONTROL OF CRACKING IN PAVEMENTS

Cracks can form as a result of residual stress caused by gradients or restraint. The main sources of residual stress are: thermal expansion and contraction, thermal curling, and moisture related warping due to plastic and/or long-term drying shrinkage. It should be noted that these sources of stress development can never be completely eliminated. Plastic shrinkage cracks are a direct result of high rates of evaporation at a very early age and can be controlled by early curing. Thermal cracking occurs when the concrete slab restrained by its own weight and friction at the interface of the layer underneath cools to ambient from the temperature rise caused by heat of hydration. The cooling of the new concrete causes it to contract and cracking may occur. However, by proper selection of joint spacing and properly designing and placing the mixture these stresses can be accommodated and random cracking can be controlled.

The lower portion of a pavement never dries out due to the subsurface moisture, but the exposed surface does and therefore undergoes seasonal wetting and drying cycles. This produces a differential drying shrinkage, which tends to cause cracks to form at the surface, which typically do not penetrate deeply into the pavement section. Pavement slabs tend to be subjected to severe changes in temperature differentials as the top of the concrete heats and subsequently slowly cools relative to the bottom of the slab in phase with the surface temperature that rapidly undergoes daily changes in ambient temperature. Maintaining an optimal joint spacing in the design process can substantially control volumetric deformations and the resulting stresses induced by temperature and moisture gradients. Full depth cracks will form at mid span resulting from loading and environmental stresses.

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 autogenous 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.

CONTROL OF CRACKING IN FOOTINGS

Although concrete transportation structures are not generally thought of as being mass concrete, they can often be sufficiently large as to be classified as mass concrete. Mass concrete is defined in ACI 116R as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of cement and attendant volume change to minimize cracking." This is a somewhat unsatisfying definition, but necessarily so given the complex nature of thermal stress problems. In extreme cases, such as in construction of large dams, determining whether a structure must be considered as mass concrete is not difficult. However, in smaller structures this cannot be determined by any simple consideration of the size of the structure. For example, the rate with which heat is dissipated from a structure is a function of the inverse square of its smallest dimension. ACI 207.1R gives some examples of smallest dimensions and time required to dissipate heat: A 150-mm (6-in.) structure can thermally equilibrate in about 1.5 h; a 1.5-m (5-ft) structure would require about a week; a 15-m (50-ft) structure would require about 2 years; a 150-m (500-ft) structure (such as Hoover Dam) would require about 200 years. This suggests that structures with smallest dimensions of several feet are likely to be candidates for consideration as a mass concrete structure, depending on other variables. Other variables complicating this overly simple analysis are cement content, cement chemistry, restraint, placing temperatures, final temperatures, and temperature gradients. An analysis is warranted to have an approximation of the criticality of the size of the structure.

The difficulty with heat-of-hydration driven temperature rise is not the thermal expansion, rather the subsequent cooling to reach thermal equilibrium with the environment. If this volume-change cycle could occur without the structure being restrained, then no cracking would occur. But, if the structure is restrained (as described in the section on Testing and Crack Detection) residual stress can develop and cracking can be expected to occur. Restraint can develop as a result of the surrounding structure, differential movements in the structure, or internal reinforcement.

When concrete is placed against a rigid material to which there is adhesion, or at least friction, then this rigid structure can provide the necessary restraint to cause the cracking in the newly placed structure. During the warming-part of the cycle, the rigidity of the concrete is not well developed, so it typically has a high capacity for creep, and adhesive and/or frictional interaction with the rigid substrate is minor. During this expansion, the concrete is in compression. When the concrete is in its cooling phase, these surface forces are better developed, creep capacity is reduced, and the negative volume change against this restraint result in tensile strain in the structure. If these exceed the tensile strain capacity, then the concrete cracks, being initiated at the restrained surface and propagating into the structure. The base of dams, i.e., nearest to the foundation, is particularly vulnerable to cracking by this mechanism.

Internal restraint is another form of restraint that can develop. During the cooling part of the cycle, the surface of the structure will cool faster than the core of the structure if the only path for heat dissipation is through the surface. Thus the surface of the structure will shrink faster than the core, resulting in surface cracks.

Reinforcing steel can also be a source of restraint, but is usually not considered in thermal stress problems of mass concrete, except in the most detailed analyses. The location of reinforcing steel for structural purposes is usually not in a location to prevent or have much effect on cracking. Reinforcing steel is placed at critical locations to contain expected cracks from thermal stress. Temperature steel does not prevent the crack, but causes several small cracks to develop instead of a single large crack.

Most of the existing practices that control thermal stress cracking were developed to solve the serious cracking problems that occurred in early mass concrete dam construction. The same considerations apply to any mass concrete structure, but there are practical considerations that cause some of the practices to be emphasized differently with smaller structures. All of the practices are directed towards limiting the volume change by restricting the net temperature change from the peak value to the final stable value. The degree of control required depends on the tensile strain capacity and the creep capacity of the concrete. Control is achieved by limiting cement content, controlling the heat of hydration of cementitious materials, precooling the concrete, and temperature control after placing.

Cement–Binder

Since the heat of hydration of the cementitious materials is the principal heat engine driving the thermal-stress problem in mass concrete, reducing cement content can have a significant impact on the temperature rise. The use of low-heat cementitious materials has been a long-standing practice with mass-concrete construction. Type IV cement (ASTM C 150) was used in construction of some of the dams built in the 1930s and 1940s. The heat of hydration of Type IV cement is limited to 60 cal/g (108 Btu/lb) at 7 days. The heat of hydration of Type I cements is typically 80 to 90 calories per gram (144-162 Btu/lb) at 7 days. More recently it was found that Type II cements or Type II with some of the cement replaced by Class F fly ash are sufficient to reduce heat evolution to tolerable levels. Type IV cement is rarely available in the United States currently.

Compliance with the 70 cal/g (126 Btu/lb) limit in modern Type II cements can only be expected if the optional heat of hydration limit is invoked by the purchaser. Moderate heat of hydration Type II cements are not available in all areas. Some areas may have moderate heat and low heat cements made according to ASTM C 595 or ASTM C 1157 (using MH and LH designations). Making the cement requires a substantial change in the production process and usually requires a special production run. Most companies only make the material when large amounts are to be purchased.

A partial alternative use of scarcely available Type II cement is to specify Type V cement, where it is available. In order to get the high resistance to sulfate attack, this cement must have a C_3A content of less than or equal to 5%. C_3A is a major contributor to early heat of hydration. These cements commonly evolve only about 75 cal/g at (135 Btu/lb) in 7 days. While this still does not meet the optional 70 cal/g (126 Btu/lb) requirement of Type II cement, it represents a substantial improvement over the 80+ cal/g (144 Btu/lb) value of commercially available Type II cements.

Replacement of a fraction of the cement with fly ash has also been a common practice in mass concreting to reduce the early heat of hydration. Class F fly ash has been used for many years at levels of 25% to 35% (volume) replacement. A 30% (by volume) replacement typically results in a reduction in 7-day heat of hydration of about 10 cal/g (18 Btu/lb). Class C fly ash typically evolves more heat in the first few days of hydration than does Class F fly ash. This can be quite variable, with some quite comparable to Class F fly ashes, while use of others results in little or no reduction in early heat of hydration. The U.S. Army Corps of Engineers (USACE) has successfully used a Class C fly ash in construction of locks and dams on the Red River in Louisiana. Volume replacements were 40% to 50%, but this was a relatively nonreactive fly ash. A combination of Type II cement with 40% Class C fly ash gave superior early strength development as compared to the same cement with a 30% replacement with Class F fly ash, but the heat of hydration problem was still manageable.

Other studies have suggested that the use of large volumes of coarse GGBFS may result in lower heat of hydration. The USACE is currently exploring portland cement-slag-fly ash (Class F) mixtures. These mixtures can have very low heats of hydration at early ages.

Aggregates

Use of larger aggregates allows lower water and cement contents to be used. Selection of proper shape and well-graded material also enables reduction of the water and cement contents.

Construction Practices

Since the objective in mass-concrete construction is to keep the difference between the peak temperature and the final stable temperature to a small enough value that the tensile strain capacity is not exceeded, cooling the concrete so that the placing temperature is well below the final stable temperature will help to keep this differential under control. Concrete placing temperatures of 10°C (50°F) have been reported by using a combination of adding ice as mixing water and chilling the aggregates with cold water (ACI 207.1R). ACI 207.1R describes procedures by which the temperature of concrete can be predicted from the temperature and mass proportions of the materials.

Some dams are constructed with embedded piping through which cold water is circulated as a means of preventing excessive heat buildup in the structure. Sometimes this is limited to the part of the structure close to the foundation, which may present a strongly restraining condition. Use of insulation to minimize the temperature differential between surface concrete and interior concrete helps prevent surface cracking.

When cement contents cannot be reduced because of overriding requirements on the concrete, and hydration rates of the cementitious materials cannot be reduced because of lack of suitable materials, then precooling of the concrete and temperature management after placing must be the major controls to prevent cracking in mass concrete. However, Cannon et al. (1993) argue that the apparently common practice of using relatively high cement factors in reinforced mass concrete for footings is unnecessary. The argument, in short, is that concrete with lower cement contents can be designed with adequate strength to support the compressive loads and the reinforcing steel is designed to support the bending loads. They also make the point that cracking in footings and foundations may ultimately be less consequential than cracking in some larger structures, such as dams.

REPAIRING CRACKS

While books have been authored to provide an easy to use general guide to describe how concrete can be repaired (Emmons, 1992), the following section discusses crack repair in bridges, pavements and footings.

Bridge Structures

Loose material is removed from cracks by blowing the portion with compressed air or by hosing the area off with water. Just prior to the placement of the repair material, the area is dried.

According to the ACI, structural cracks are V-grooved to a depth of 1 in. and then blown clean. Then the groove is filled with a neat epoxy. Latex-modified concrete can be brushed into the groove instead of the epoxy when latex concrete is monolithically placed. Concrete cracks as tight as 0.2 mm (0.008 in.) may also be repaired by gravity-fill crack sealers. These sealers are either low viscosity epoxy, or high molecular weight methacrylate, or urethane. To perform this method the concrete must be at least 28 days old. The surface of the concrete must be dry and clean. To remove all dust, dirt, and debris, compressed air can be used. The surface temperature should not be less that 13°C (55°F). To ensure that the cracks are the most open, the resin is applied during the lowest temperature of the day. If the cracks are wider than 1 mm, they are filled with No. 50 sieve size silica sand before the polymer is used. The sealer should be applied directly to the cracks and a few minutes should be allowed to allow the sealer to seep down into the crack. If more sealer is needed, then additional applications are made until the crack is filled. Material may be spread and worked into the cracked area using a broom or squeegee. The excess material is then brushed off the surface before the polymer hardens. To improve skid resistance sand may be spread on the polymer-coated area on bridge decks. Formwork or sealing would be needed to contain the repair material in cracks extending to the bottom surface (ACI 345).

Pavements

Pavement cracks are always exposed to the elements and are affected by physical processes and chemical mechanisms. The physical processes include thermal contraction, drying shrinkage, wetting and drying, and freezing and thawing. Chemical mechanisms refer to the reaction of the aggregate within the concrete. The cracks start out being only continuous through an inch from the top or bottom surface (depending on the mechanism that initiates the cracks), but can increase to full depth cracks in rare cases when other forms of deterioration are present. In addition, fatigue cracks that initiate either from the top or bottom surface depending on the location of the critical load and the superimposition of stresses due to environmental factors, progress top-down or bottom-up respectively with application of additional traffic loads during the design life of the pavement.

The repair of plastic shrinkage cracks generally do not require major repair and can generally be sealed to protect the underlying concrete from infiltration of surface water, which has a direct effect on most deterioration mechanisms. Lack of sealing major cracks can lead to spalling and other distress following freezing and thawing.

Cracks where movement occurs are called working cracks. According to the American Concrete Institute, repair should cater for the anticipated movement. A suitably dimensioned recess should be cut along the line of the crack and then sealed with an appropriate sealant with a bond breaker. A surface seal made with a strip of formed sheet material may be appropriate in certain circumstances. The choice of sealant depends on the amount of movement forecast, and the limitations imposed by the size of the recess which can be cut, together with the situation, i.e., vertical or horizontal. There are three types of sealant in general use; mastics, thermoplastic and elastomers (ACI 345).

Mastics are generally viscous liquids, such as non-drying oils, or low melting asphalts, with added fillers or fibers. They are usually recommended where the total movement will not exceed 15% of the width of the groove. The groove should be cut so that it has a depth-to-width ratio of 2. Mastics remain plastic and will not withstand heavy traffic or solvents. In hot weather the mastic will tend to be forced out by the expansion of the adjacent structures and the surplus flattened and/or removed by traffic. Dirt and debris can become embedded in the material. Mastics are typically the cheapest of the sealants but their use should be restricted to vertical situations or those which are protected from traffic (Evans et. al., 2001).

Thermoplastics become liquid or semi-viscous when heated. The pouring temperatures are usually above 38°C (100°F). They include asphalts, rubber-modified asphalts, pitches and coal tar. The groove depth-to-width ratio is of the order of 1 and the total design movement is of the order of 200% of the groove width. Although these materials soften much less than mastics, they may extrude at high ambient temperatures and debris may become embedded. Some of these materials are degraded by ultraviolet light and thus may become hardened and lose elasticity after a few years of exposure to direct sunlight (Evans et. al., 2001).

Elastomers include polysulphides, epoxy polysulphides, polyurethanes, silicones and acrylics and may come as one part or two part materials. They can have considerable advantages over other types of sealants in that they do not have to be heated before application. In addition they typically exhibit favorable adhesion to concrete and are not susceptible to softening within the normal range of ambient temperatures. Elastomers have a much higher degree of elongation than other sealants and many of them are capable of over 100% extension but in practice this should be limited to ±25%. The groove depth-to-width ratio should be 0.5. It is important to take steps to prevent the materials from adhering to the bottom of the grove; it should adhere to the sides only (Cady, 1995; Evans et. al., 2001).

Footings

The approach to repairing cracks depends on the effect the crack has on the structure. If structural stability is the issue, then repair with a high-tensile-strength material is necessary. If leaking is the problem, then crack filling is required. The USACE guidance (EM 1110-2-2002, 1995) on crack repair first directs that a crack in mass concrete be analyzed to determine whether it is active or not and whether there is a structural-stability problem or a leakage problem that must be repaired. "Judicious neglect" is sometimes the chosen option.

Crack arresting techniques are useful for stopping the propagation of a crack when it is caused by restrained volume changes. This repair has been used in mass concrete structures to prevent a crack from propagating into an adjacent placement. The simplest form of this repair technique is to place a grid of reinforcing steel over the cracked area, then place concrete over the grid.

According to Crumpton and Stratton, cracks that require repair because of structural-stability problems are normally repaired either by adding reinforcing, stitching, or applying an external stress. The first method is to drill holes [commonly ¾ in. (19 mm)] perpendicular to, and through the crack [at least 18 in. (450 mm) deep]. These holes and crack plane are then filled with epoxy under low pressure [maximum 1.4 MPa (200 psi)], and reinforcing steel (commonly No. 4 or 5 bars) is inserted into the holes. A large crack in the landside wall of the Eisenhower Lock was repaired by a similar method but using steel cable. Holes were drilled from a gallery above the crack through the crack and into the concrete next to the foundation. The cables were inserted from the gallery and anchored into the lower part of the structure, then tensioning from the gallery. Stitching is a method used to repair surface cracks. Holes are drilled on both sides of the crack and anchoring "dogs" (staples) are inserted into the holes either with nonshrink grout, expanding mortar, or epoxy. The stitching should be variable in length and orientation so that loads are not transmitted to a single plane within the sound concrete. External stressing is a repair technique that may have some application in structural mass concrete. Threaded steel rods are mounted on the surface of the structure using steel mounting plates to anchor each end of the rod across the crack. The mounting plates are bolted into or through the structure. Tension is applied along the rod with turnbuckles or by tightening the anchoring nuts at the end of the rod. (Crumpton and Stratton, 1983)

Practices for the Prevention of Cracks

BRIDGES

Cracking in bridge decks is a significant issue in the highway construction industry, particularly with the increased use of high performance concrete. A great deal of research has been done to identify causes and prevention measures in an effort to extend the service life of bridge decks.

According to Krauss and Rogolla, and Weiss, concrete used in bridge decks ideally should have a low elastic modulus, high creep capacity, low coefficient of thermal expansion, low permeability, low heat of hydration, high toughness, low drying shrinkage, and low autogenous shrinkage (Krauss and Rogolla, 1995; Weiss et al., 1999). One of the most critical properties in the mixture proportions of bridge deck concrete is the w/c; maintaining it reasonably low (i.e., around 0.40) provides best results. However, mixtures with an extremely low w/c may exhibit substantial autogenous shrinkage, especially during the first 24 hours. Autogenous, drying, and thermal shrinkage is reduced when these concretes contain an aggregate volume as high as is practical.

While the nature of the material assures that concrete will experience some degree of cracking, there are things that can be done during the design and construction of a bridge deck to help prevent the development of cracks or mitigate the severity of any that do develop. The following practices for materials and construction are offered as information only and do not guarantee crack free surfaces for all possible combinations of concretes.

In addition, the design of the structure can have a large effect on the presence of cracks. Flexible structures, structures with large skews and shallow cover depth would be more prone to cracking.

Material Selection

Details on the desirable material properties are provided in the section on Control of Cracking.

Aggregates

The use of soft aggregates (sandstone) result in increased drying shrinkage, while the use of hard aggregates (quartz, dolomite, and limestone) results in decreased shrinkage. In the absence of aggregate performance information, it is good practice to choose an aggregate with limited absorption. However, cracking in concrete is related to many factors, and aggregates with high absorption such as the lightweight aggregates can be used in some instances to minimize cracking due to internal cracking (Lura et al., 2004).

Cement–Binder

Minimizing the cement content has a positive direct effect on reducing and controlling cracking. Using less cement decreases the heat generated from hydration resulting in less thermal shrinkage. Concrete with less cement exhibits less drying and autogenous shrinkage. Research suggests that some types of cement may contribute more to shrinkage of concrete than others because of their composition. For example, cements with high alkali contents may show a rapid shrinkage with a greater overall magnitude. Also, Type II cement has lower heat of hydration than Type I. Addition of pozzolanic materials such as fly ash and slag will reduce heat of hydration and consequently results in reduction in concrete temperature rise and therefore thermal shrinkage.

Admixtures

Certain admixtures can be used to control shrinkage. Some water reducing and retarding admixtures reduce the rise in temperature of the concrete and may be used to reduce the potential for thermal shrinkage cracking. Retarders are particularly useful when the ambient temperature is expected to reach or exceed 24°C (75°F). Water reducers are used because they result in reduction in the amount of mix water and resulting drying shrinkage. Shrinkage reducing admixtures may also be used for some applications to reduce shrinkage by as much as 50% (Weiss and Berke, 2002).

Construction Practices

Construction issues can be classified into two types: physical construction issues that are within the control of contractor, and environmental issues that are not. Both categories can be addressed through a combination of specifications, good concrete placement practices, project partnering, inspection, proper planning, and contractor awareness. The following section primarily describes construction issues related to cracking and how they can be dealt with at the project level.

Temperature Exposure

Experience has shown that it is a good practice to minimize thermal gradients during placement and curing of the deck. This includes potential differences in temperatures between the newly placed deck and the substrate girders. If the deck is placed during the daytime heat, the girders will be warmer and will cool late in the day and through the evening while the concrete is reaching its peak temperature as a result of its heat of hydration. A deck placed at night or in the early morning hours will experience its heat rise concurrent with that of the supporting girders, thereby limiting the thermal gradient.

Summer heat is not the only issue with girder temperature during deck construction. Cold-weather concrete specifications typically call for the concrete temperature to be maintained at a certain desirable level for the duration of the curing period. This is most often accomplished through the use of insulated curing blankets and/or the construction of a temporary heated enclosure. The use of curing blankets, while providing for heat retention during curing, allows the girders to be exposed to ambient air temperatures and can result in a potentially severe temperature gradient. Likewise, many enclosures are often built above the deck level but allow the girders underneath to be exposed to ambient temperatures. Some configurations do provide heat from underneath but do not provide much protection from the environment. Heat retention underneath can be provided by either a complete “wrap-around” enclosure of the deck or, at a minimum, by draping tarps over the outside of the bridge to prevent wind from blowing under the bridge. Site-specific conditions dictate the appropriate measures to be taken.

Babaei and Purvis (1995) recommended limiting the restrained thermal shrinkage to 150 microstrain by maintaining the concrete deck/girder temperature differential to no greater than

12°C (22°F) for 24 h after the deck has been placed. This can be accomplished by selecting an appropriate time of the day to place the concrete and, in cold season, utilizing proper temperature controls during and after placement of the deck concrete.

In conjunction with the concrete deck/girder temperature differential cited above, Babaei and Purvis (1995) recommended limiting the drying shrinkage as follows: The 4-month specimen drying shrinkage (in accordance with ASTM C-157) should be under 700 microstrain (or 400 microstrain 28-day shrinkage). The allowable deck/girder differential temperature can be increased provided the drying shrinkage is decreased. Further details are provided in (Babaei and Purvis, 1996).

Environmental Exposure During Placement and Curing

The trend toward use of supplementary materials for bridge deck applications has resulted in higher quality bridge decks but also led to the use of concrete that is more sensitive to environmental exposure and potentially prone to plastic shrinkage cracking. This is a result of these concretes generally exhibiting little to no bleed water, thereby making them prone to plastic shrinkage. There are several things that can be done to reduce the length of time concrete is exposed to evaporation and the potential formation of plastic shrinkage cracks.

Utilizing the ACI nomograph (ACI 305) to determine the theoretical evaporation rate, several states have established limits generally ranging from 0.5 $kg/m^2/h$ to 1.0 $kg/m^2/h$ (0.1 $lbs/ft^2/h$ to 0.2 $lbs/ft^2/h$) when decks may be placed. These limits are particularly important for concretes containing supplementary cements. If the evaporation rate is high, practitioners often consider delaying the placement. If concrete decks are placed at or near these limits, protection measures are necessary.. State specifications often refer to the use of either fog sprays to provide high humidity above the deck or wind screens to provide protection from the drying effect of the wind. If fog sprays are to be used, the fogging nozzle is to provide a constant mist that does not allow water to accumulate on the deck. Wind screens are generally not a practical alternative for bridge applications as they do not lend themselves to construction immediately prior to deck placement, and the contractor may not be amenable to placing a wind screen that may not be needed. Additionally, wind screens may actually increase the effective wind at deck level due to the vortices that form over the top of the barriers. Topically applied evaporation retarders may also be used.

To limit environmental exposure once the placement has begun, concrete usually is not deposited more than a few feet in front of the finishing machine. Concrete usually is deposited across the width of the deck and as near to its final location as practical.

With a properly designed mixture and a finishing machine that is set up correctly, the machine should be able to provide the proper finish on all areas other than those typically not accessible such as near the bridge rail reinforcement or at the beginning of a bridge on a significant skew. Hand finishing usually is minimized and the use of bullfloats generally discouraged. Not only does excessive finishing lead to longer environmental exposure but can also lead to future scaling problems.

Once the finishing is complete, wet burlap or cotton mats are typically placed as soon as possible but not more than 10 to 15 min after the finishing machine is done. This requires the contractor to be prepared to place the burlap and keep the placement operation “tight.” Some may object to placing the wet cure so soon out of concern for marring of the deck. While deep indentations are not desired, and can be minimized with proper care while laying the burlap onto the deck, surface marring is not a determent to the immediate placement of the burlap. Minor cosmetic damage is tolerable to provide a high-quality, longer-lasting deck.

Another measure that may be incorporated to mitigate the effects of environmental exposure is the use of curing compound. If compound is used, it is applied with a sprayer other than a "garden sprayer," and applied immediately after finishing. The compound is placed in two passes, the second pass being perpendicular to the first and applied at the manufacturer's recommended coverage rate. If a compound is used, it is viewed as additional protection and not as a "safety factor" to allow for delayed start of the wet curing.

Placement Sequence

For years multi-span decks were constructed by initially placing concrete in the positive dead load moment zones with a subsequent placement in the negative moment region. This allowed for some camber to come out of the girders and lessened the amount of uplift on the deck in the negative moment region.

With the use of self-propelled finishing machines and set-retarding admixtures, it has become increasingly common, and economically desirable, to construct a multi-span deck in a single placement. While this is allowed in the specifications, there is usually accompanying language requiring the concrete to be in a plastic state in previously placed spans. This, theoretically, is to allow the camber to come out before it can initiate any cracking by uplifting the set concrete. In practice, this is often difficult to accomplish. To be done properly, the retarder dosage is varied throughout placement in an attempt to achieve uniform deck setting. There is also a chance that the contractor and inspection staff will not agree on the meaning of "plastic" concrete and that the placed concrete will experience uplift and the accompanying stresses.

While constructing a multi-span deck in more than one placement will create cold joints, these joints can be prepared and treated to mitigate their effect on long-term deck durability. Another alternative is to continue to allow single continuous placements while assuring that the contractor and the concrete supplier fully understand the need for the concrete to remain plastic and utilize the retarder appropriately. Discussion often occurs at the pre-placement meeting so that all parties understand and agree on how the plastic condition will be maintained for the necessary duration.

Vibration–Consolidation

One of the basics of good concrete practice is to assure proper consolidation, yet it is often the most overlooked facet of bridge deck construction. Proper consolidation helps to assure uniformity in the deck and the avoidance of areas that may serve as crack initiation/propagation points. Construction personnel and inspectors often pay little attention to proper vibration technique and thoroughness. Additionally, often times there is only one, or perhaps two, vibrator(s) present during the placement. This may be sufficient for smaller placements but will not be enough to provide proper vibration/consolidation if the placement rate is too fast. One potential way to address this is being done by the New York State Department of Transportation (DOT). They now require a minimum of two (plus one back-up on site) vibrators to be used for all deck placements with additional ones needed based on the placement rate. The Kansas DOT requires multiple vibrators spaced at 0.3-m (1-ft) intervals and held in a mechanical system capable of uniformly consolidating the entire bridge deck concrete.

Concrete Girder Age

Differential creep and shrinkage between the deck and girders can also lead to the development of cracks. While creep and shrinkage of concrete girders is considered during the design phase, these issues are often not considered during construction. If a concrete deck is going to be placed on a prestressed concrete girder, the project can be scheduled so the girders are not cast well in advance of when deck construction operations are ready to begin. This will reduce the differential creep and shrinkage that occurs when the girders have been cast a significant amount of time prior to deck placement. Due to the potential effect on the contractor's schedule, this is something to be addressed in the project specifications, as a contractor cannot be reasonably expected to significantly alter his schedule after the project has been awarded.

PAVEMENTS

In properly designed pavements, material selection and construction practices can affect the crack occurrence.

Material Selection

Concrete is generally used with as large an aggregate volume as possible. In many cases this may consist of using as large an aggregate as feasible. Further the heat generated by the concrete during hydration is kept as low as possible. Water reducing admixtures are typically used to reduce the mixture water in concrete. This can be accomplished by using moderate or low-heat cements or through the use of supplementary materials like fly ash or slag.

Construction Practices

Paving concrete is cured to limit excessive moisture or thermal gradients. These concretes are adequately consolidated, the joint spacing set small enough to control excessive stresses, and sawed at an optimal time to provide stress relief.

FOOTINGS

Both the ACI and the USACE have developed major sets of guidelines on control of thermal-stress cracks. ACI 207.1R focuses principally on dam construction. Much of the information presented for mass concrete for dams are applicable to footings. ACI 224R is a general document on control of cracking in concrete structures, not specific to mass concrete. The ACOE have analytical protocols for dealing with thermal analysis of mass concrete. ETL 1110-2-542 (1997) describes three levels of thermal studies and gives some examples.

Material Selection

Concrete is generally used with as large an aggregate volume as possible. In many cases this may consist of using as large an aggregate as feasible. Furthermore, the heat generated by the concrete during hydration is kept as low as possible. This can be accomplished by using moderate or low-heat cements or through the use of supplementary materials like fly ash or slag.

Construction Practices

Mass concrete is properly cured to limit excessive moisture or thermal gradients. Recommendations for the control of temperature and thermal gradients in mass concrete are found in ACI 207.1R and ACI 207.2R.

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