CONTROLLING CRACKS IN BRIDGE DECKS

By
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A Report on Research Sponsored by

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Structural Engineering and Engineering Materials
SM Report No. 129
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THE UNIVERSITY OF KANSAS CENTER FOR RESEARCH, INC.
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ABSTRACT

Laboratory studies incorporating crack-reducing technologies with those included in specifications for low-cracking high-performance concrete (LC-HPC) bridge decks and analyses of data from crack surveys and construction observations on more than 50 concrete decks are described.

The laboratory investigations include the combination of supplementary cementitious materials (SCMs), slag cement and silica fume, as partial replacements for portland cement, internal curing through use of pre-wetted lightweight aggregates (IC), and calcium oxide-based (CaO) or magnesium oxide-based (MgO) shrinkage-compensating admixtures (SCAs) or expansive additives with LC-HPC. A modified version of ASTM C157 in which length-change measurements begin 5½ ± ½ hr after casting concrete is developed and used to evaluate swelling and shrinkage including the combined effects of supplementary cementitious materials (SCMs), internal curing (IC), and MgO- and CaO-based shrinkage-compensating admixtures (SCAs) on shrinkage of concrete specimens designed according to LC-HPC specifications. The results show that the modified version of ASTM C157 helps to capture the early-age behavior of concrete mixtures. IC is effective in reducing 0 to 20-day drying shrinkage in concrete, but the opposite is observed regarding 20 to 180-day drying shrinkage. SCMs induce increased first-day expansion and reduce shrinkage. A further increase in first-day expansion and a reduction in shrinkage is obtained when IC is used in conjunction with SCMs. The SCAs evaluated in this study reduce the tendency to develop shrinkage strain. The CaO-based SCA induces the more rapid expansion of greater magnitude, while the MgO-based SCA expands more gradually. When the CaO-based SCA is incorporated in a mixture containing SCMs or SCMs and IC, expansion is further increased, an observation that cannot be made for mixtures incorporating SCMs with the MgO-based SCA.

Analyses of crack survey results and construction observations for more than 50 bridge decks are used to better understand the principal factors affecting cracking, evaluate the effects of construction practices on cracking, and assess the effectiveness of crack-reducing technologies, such as synthetic fibers and IC. The results indicate that bridge deck cracking increases with age. Paste content (volume of cementitious materials plus water) is the most dominant factor affecting cracking, and parameters such as slump, compressive strength, and air content have much less effect. Decks cast with concrete with paste contents exceeding 27.2% exhibit substantially greater cracking than those with lower paste contents, regardless of other factors. The incorporation of a crack-reducing technology such as IC in the decks cast with concrete having paste contents exceeding 27.2% cannot overcome the negative effect of the greater paste content on cracking. Individual contractors and poor construction practices, particularly poor consolidation and overfinishing the concrete, can significantly affect cracking even when the decks are cast with a low-shrinkage concrete (paste content limited to 27.2%) even where a crack-reducing technology is used. Bridge decks with precast partial-depth concrete deck panels with cast-in-place concrete toppings show excellent cracking performance if the topping has a low paste content and good construction procedures are used. Greater average crack widths correspond with greater crack densities.

Keywords: Bridge deck cracking, concrete, construction practices, crack control, crack-reducing technologies, crack width, internal curing, paste content, shrinkage-compensating admixtures, synthetic fibers

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CHAPTER 1 - INTRODUCTION

1.1 GENERAL

One of the major problems impacting the performance of bridges is the deterioration of the concrete decks. This can be a result of concrete distress caused by freeze-thaw damage, the alkali-aggregate reaction, cracking, and corrosion of reinforcing bars. Beginning in the 1970s, several techniques were instituted to deal with the phenomenon of chloride penetration to the reinforcement in concrete bridge decks, as the corrosion of steel reinforcement was found to increasingly affect performance. Increasing the clear cover to reinforcement and using epoxy-coated reinforcing bars and concrete with lower permeability were among those techniques (Russell 2004). However, a comprehensive field evaluation of 59 bridge decks with different properties by Lindquist et al. (2005) showed that regardless of other factors, susceptibility to corrosion significantly increased at crack locations compared to regions of uncracked concrete. Bridge deck cracking is considered to be a major durability problem by transportation agencies (Aktan et al. 2003).

Starting in 1991, the University of Kansas (KU) laid the foundations to address the bridge deck cracking problem. This effort first involved three bridge deck cracking studies over 11 years that included surveys of composite steel girder bridges in Kansas (Schmitt and Darwin 1999, Miller and Darwin 2000, Lindquist et al. 2005). One hundred thirty-nine crack surveys were performed, covering 160 concrete placements on 76 bridges. Thus, the majority of the decks were surveyed more than once. The results obtained from the crack surveys, combined with laboratory work performed at KU during that period, led to the development of specifications for low-cracking high-performance concrete (LC-HPC) bridge decks. These specifications included a combination of requirements addressing material properties, construction practices, and deck

configuration to accommodate changes in materials to improve the cracking performance and durability of concrete bridge decks. The work by Schmitt and Darwin (1995, 1999) and Miller and Darwin (2000) provided the impetus to initiate what became a two-phase, 13-year Pooled-Fund study with participation by 19 state departments of transportation (DOTs), the Federal Highway Administration, and industry. To date, 16 bridges in Kansas and six in other states have been built employing LC-HPC specifications. Nineteen of the bridges have an associated control bridge for comparison. Bridges constructed under LC-HPC specifications have demonstrated improved cracking performance when compared to control decks built by standard DOT specifications as demonstrated by annual cracking surveys (Lindquist et al. 2008, McLeod et al. 2009, Yuan et al. 2011, Pendergrass and Darwin 2014, Alhmood et al. 2015 Darwin et al. 2016).

This study addresses the bridge deck cracking problem through both experimental and field evaluations. In the experimental part, the effects of different crack-reducing technologies on cracking potential of candidate mixtures for bridge decks are evaluated. These technologies include the use of pre-wetted fine lightweight aggregate (LWA) for internal curing (IC), supplementary cementitious materials (SCMs) (slag and silica fume), and calcium oxide-based (CaO) and magnesium oxide-based (MgO) shrinkage-compensating admixtures (SCAs). These new methods will potentially be included in future LC-HPC projects. The results from the field evaluations, collected from fifty bridge deck placements having a similar deck and superstructure type (conventional monolithic concrete decks built with 100% portland cement supported by steel girders) will be used to assess the effects of different factors on long-term cracking performance and determine which factors have the most significant effects. These factors include mixture proportions, material properties, environmental conditions (temperature and time of placement during the day), and construction procedures. In addition, the results from a six-year field study of

twelve other bridge deck placements will be used to evaluate the effects of deck type, superstructure type, concrete mixture proportions, and crack-reducing technologies on long-term cracking performance.

This chapter reviews previous research and presents the objective and scope of this study.

1.2 CRACKING MECHANISMS

In concrete bridge decks, cracking occurs when the tensile stress exceeds the tensile strength of the concrete. Stresses can be caused by external loading or restraint of volumetric changes caused by temperature and shrinkage. The latter plays an essential role in bridge decks since a high degree of restraint is present, and because of their large surface to volume ratio, bridge decks are susceptible to shrinkage due to moisture loss. Bridge decks also have a high degree of restraint caused primarily by the composite action between the girders and concrete (Pendergrass and Darwin 2014). Stresses due to differences in thermal and shrinkage strains are higher for bridges with steel girders than those with prestressed concrete girders because steel and concrete have different coefficients of thermal expansion and steel is not affected by changes in moisture. Restraint in concrete bridge decks is also provided by the steel reinforcement and fixity at the ends (integral abutments).

1.2.1 Concrete Shrinkage

Based on previous research (Schmitt and Darwin 1995, Krauss and Rogalla 1996, Miller and Darwin 2000, Russell 2004) it is well understood that restrained shrinkage of concrete is a significant factor contributing to both early-age and long-term cracking in bridge decks.

The following sections will explain briefly different types of shrinkage: plastic shrinkage, drying shrinkage, autogenous shrinkage, and carbonation shrinkage.

1.2.1.1 Plastic Shrinkage

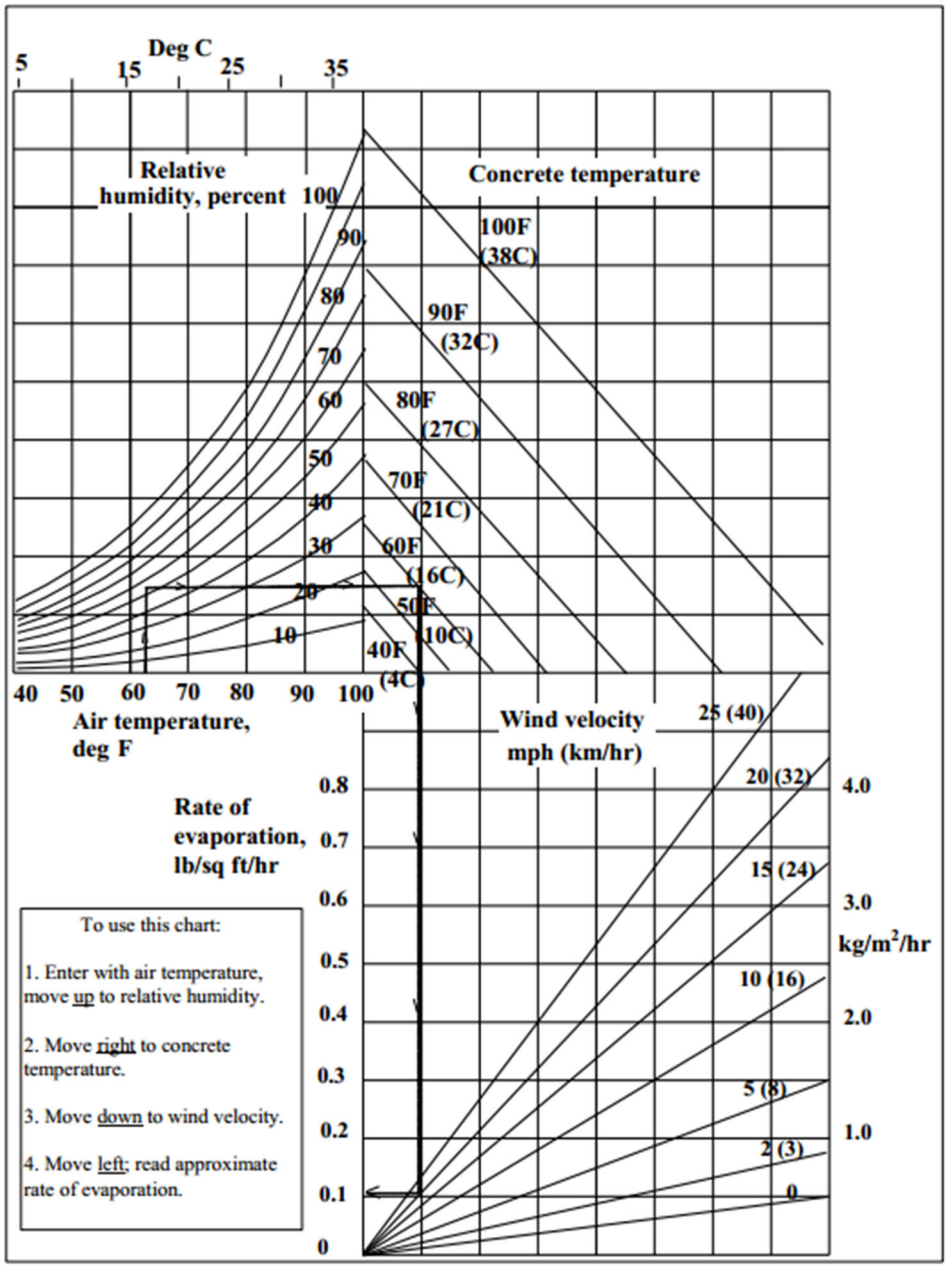
In fresh concrete (prior to set), the space between the cement particles is filled with water. As water at the surface evaporates, if it is and not replaced by the bleed water menisci are formed. As a result of the formation of these menisci, negative capillary pressure is developed causing a reduction in volume (Mindess et al. 2003). Because the concrete below the surface retains more bleed water and thus, does not shrink as much, it provides restraint, inducing tensile stresses in the surface concrete. Depending on the amount of water lost and the size of the pores, the tensile stresses may be enough to cause plastic shrinkage cracking in a material that has virtually no tensile strength. Plastic shrinkage cracks are oriented randomly, short (2-7 in., 50-180 mm), and shallow (depth of 2 to 3 in., 50 to 75 mm) (Krauss and Rogalla 1996). To control plastic shrinkage cracking, evaporation of water from the surface of the concrete must be minimized following concrete placement.

A decrease in the bleeding rate and an increase in the evaporation rate are major factors contributing to plastic shrinkage. The bleeding rate of concrete can be reduced for various reasons, most of which relate to the material properties and proportions of the concrete mixture. In general, mixtures with lower water-to-cementitious material (*w/cm*) ratios have lower bleeding rates. Also, the use of finely-ground cement, silica fume, air-entraining admixtures (AEAs), and superplasticizers can all cause a reduction in bleed water (Soroka and Ravina 1998, Russell 2004, Pendergrass and Darwin 2014).

High ambient and concrete temperatures, low relative humidity, and high wind velocity increase the evaporation rate of concrete. The monograph shown in Figure 1.1 is often used to estimate the loss of water. If the evaporation rate exceeds 0.2 lb/ft^2/h (1.0 kg/m^2/h), measures, such as reducing concrete temperature using ice, protecting concrete from direct wind, and preventing evaporation shortly after finishing (such as with wet burlap) can significantly reduce the potential

for plastic shrinkage. Keeping the forms and reinforcing steel wet just before concrete is placed can minimize water loss from the concrete to the forms and steel bars. If concrete contains pozzolanic cementitious materials, the evaporation rate should be limited to 0.1 $lb/ft^2/h$ (0.5 $kg/m^2/h$) (Mindess et al. 2003).

Surface evaporation can occur in both warm and cold ambient conditions, and the latter can be more damaging to concrete since concrete will remain plastic for a longer period and the relative humidity just above concrete can be very low in cold weather (Pendergrass and Darwin 2014).



Effect of concrete and air temperatures, relative humidity, and wind velocity on the rate of evaporation of surface moisture from concrete. This chart provides a graphic method of estimating the loss of surface moisture for various weather conditions. To use the chart, follow the four steps outlined above. When the evaporation rate exceeds 0.2 lb/ft^2/hr (1.0 kg/ m^2/hr), measures shall be taken to prevent excessive moisture loss from the surface of unhardened concrete; when the rate is less than 0.2 lb/ft^2/hr (1.0 kg/m^2/hr) such measures may be needed. When excessive moisture loss is not prevented, plastic cracking is likely to occur.

Figure 1.1—Evaporation rate nomograph (ACI Committee 308 1997)

1.2.1.2 Drying Shrinkage

Drying shrinkage occurs due to water loss from hardened concrete and is the primary cause of cracking in bridge decks, as confirmed by a majority of laboratory and field investigations (Russell 2004, Vargas 2012).

Cracking can occur when shrinkage in concrete is restrained. There is always restraint present in a structure; the amount and type of restraint will determine the induced stress in the member. Concrete in a bridge deck is restrained externally and internally. External restraint is caused by integral abutments and also the supporting girders. Internal restraint is provided by concrete itself when non-uniform drying takes place because of moisture loss at the surface. This non-uniform drying, which occurs whether or not the deck is externally restrained, will result in a shrinkage gradient through the depth of the deck, inducing tensile stress near the surface that is drying. Internal restraint is also provided by the steel reinforcement within the deck, the amount and spacing of which can affect the induced stresses and the width and density of cracks.

This section describes the three main phenomena that result in the development of internal stresses after water has evaporated from the cement paste matrix: capillary stress, disjoining pressure, and free surface energy.

1.2.1.2.1 Capillary Stress

The spaces between the particles in the cement paste matrix that were initially filled with excess water are referred to capillary voids or pores. When the pore water from the voids near the concrete surface evaporates, capillary stresses develop. The magnitude of the capillary stress is dependent on the radius of the meniscus and surface free tension of the pore water. Capillary stress is expressed as:

$$P_{cap} = \frac{2\gamma . \cos\theta}{r}$$

where P_{cap} = capillary stress (Pa).
γ = surface free tension energy of water (N/m).
r = meniscus radius (m).
θ = contact angle between pore fluid and solids (rad)

Schmidt and Slowik (2009) showed that in a system with pore sizes between 8×10^{-8} and 2×10^{-6} in. (2.5 and 50 nm), capillary stresses can reach a magnitude of about 7.3 psi (50 kPa), which can cause cracks. When pore sizes are larger than 2×10^{-6} in. (50 nm), the induced suction force is too low to cause shrinkage. When the pore sizes are smaller than 8×10^{-8} in. (2.5 nm), a meniscus will not form, and there will be no shrinkage (Larrard 1997). Capillary stress cannot develop when the relative humidity is less than 45% since menisci do not have enough stability to induce forces (Mindess et al. 2003).

1.2.1.2.2 Disjoining Pressure

Adsorbed water, the water held on the surface of a material by electrochemical forces, has significantly different properties than the absorbed water present in the same medium (ACI Concrete Terminology 2013). When the relative humidity increases, the thickness of the layer of adsorbed water between the adjacent calcium silicate hydrate (C-S-H) particles increases, resulting in a disjoining pressure that acts opposite to the attracting forces between the particles. When the RH drops, however, the adsorbed water between C-S-H particles evaporates, leading to a decrease in disjoining pressure. As a result, the attraction forces between C-S-H particles become prominent and draw the particles close to one another, which causes shrinkage. The magnitude of disjoining pressure depends on the relative humidity and its effect on shrinkage is not significant unless the RH is above 45 percent (Mindess et al. 2003).

1.2.1.2.3 Free Surface Energy (Tension)

Free surface energy contributes to drying shrinkage of concrete when the RH is below 45 percent. When the final layers of water evaporate from C-S-H particles, the free surface energy of the particles increases significantly. This increase causes the development of compression forces between particles that are dependent on the particle specific surface area and surface energy (tension). As a result of these compression forces, volume reduction and shrinkage take place (Mindess et al. 2003).

1.2.1.3 Autogenous Shrinkage

A reduction in the internal relative humidity of concrete not caused by the external moisture transfer can result in a reduction in the bulk volume of cement paste, referred to as autogenous shrinkage. The reduction in internal relative humidity is caused by the self-desiccation and occurs in pastes with low *w/cm* ratios (Mindess et al. 2003). Autogenous shrinkage does not play a significant role in the volume change of concretes with *w/cm* ratios greater than 0.42 because there is enough water available in the pores to support on-going hydration (Powers and Brownyard 1948). In general, autogenous shrinkage plays an increasingly important role as the *w/cm* ratio decreases. In these cases, the cement paste matrix becomes denser, limiting access of external curing water to the hydrating particles.

A number of techniques can be used to reduce autogenous shrinkage. Shrinkage-reducing admixtures (SRAs) have been shown to be effective in reducing both drying and autogenous shrinkage through the reduction of the surface tension of water (Bentz et al. 2001). Partial replacement of normalweight aggregate with pre-wetted LWA to provide IC water has also been used to reduce both autogenous shrinkage has been effective (Bentur et al. 2001) and drying shrinkage (Browning et al. 2011). In recent years, the use of superabsorbent polymers has also shown to be an effective method to reduce autogenous shrinkage (Jensen and Hansen 2001).

1.2.1.4 Carbonation Shrinkage

Carbonation shrinkage, which results from chemical reactions between carbon dioxide from the atmosphere and the hydration products in cement paste, C-S-H, and calcium hydroxide. Carbonation shrinkage affects the long-term shrinkage of concrete (Mindess et al. 2003).

1.2.2 Thermal Effects

An increase or decrease in internal concrete temperature will result, respectively, in the expansion or contraction of concrete. Restraint, such as present in bridge decks from end abutments, girders, and internal steel reinforcement, limits thermally induced volume changes of concrete, causing stresses to develop. Some studies have concluded that bridge deck cracking is highly related to thermally-induced volume changes (*Durability of Concrete Bridge Decks* 1970, Krauss and Rogalla 1996, Babaei and Purvis 1996). Placing concrete in a bridge deck with higher temperatures than that of the girders (hot concrete placed on cold days) will induce significant thermal stresses because of the differences in the initial temperatures of the two materials. To avoid this, it has been suggested that the air under the deck be heated on cold days to raise the temperature of the girders (*Durability of Concrete Bridge Decks* 1970, Babaei and Fouladgar 1997). In hot-weather precautions need to be taken to limit concrete temperature using ice or liquid nitrogen.

1.2.3 Concrete Settlement

Settlement or subsidence cracking occurs as fresh concrete settles around fixed objects, such as reinforcing bars. Settlement of the plastic concrete around reinforcing bars will result in the development of tensile stresses above the reinforcement and may result in the formation of cracks.

Settlement cracking increases as concrete slump and reinforcing bar size increase and as concrete cover above the bars decreases (Dakhil et al. 1975). Insufficient consolidation of concrete

has also shown to increase settlement cracking (Issa 1999). Recent studies have shown that the use of synthetic polymer fibers, SRAs, IC through use of pre-wetted LWA, and rheology-modifying admixtures reduce settlement cracking (Suprenant and Malisch 1999, Banthia and Gupta 2006, Henkensiefken et al. 2010, Kakooei et al. 2011, Al-Qassag et al. 2015).

1.2.4 External Loading

The self-weight of the concrete deck, dead loads from railings, and traffic loads can cause flexural cracks in concrete bridge decks. Flexural cracks are usually seen over the piers where a continuous superstructure configuration is used. For that reason, the use of a simply-supported superstructure has been recommended by some as a way to provide better cracking performance (Keller 2004). It has been demonstrated, however, that the contribution of the flexural stresses to the overall cracking performance of concrete decks is minimal compared to the effects of stresses due to restrained drying shrinkage and thermally-induced volume changes (Krauss and Rogalla 1996).

1.3 TYPES OF BRIDGE DECK CRACKING

Cracking in bridge decks can appear in various forms and is usually categorized based on its orientation relative to the longitudinal axis of the bridge. Cracks are commonly divided into four main types: transverse, longitudinal, diagonal, and pattern or map. Figure 1.2 shows examples of transverse, longitudinal, diagonal, and pattern/map cracking (Darwin et al. 2016). Cores taken from the cracked concrete sections on bridge decks show that some cracks can propagate through the depth of the deck. Cracks to the depth of the reinforcing steel can accelerate the corrosion by providing a direct path for moisture, oxygen, and deicing agents (Figure 1.3). Research demonstrates that even very narrow cracks (as small as 0.003 inches) are wide enough to permit the ingress of chloride ions, moisture, and other aggressive agents and that when cracks are present,

chloride ingress increases significantly both in vertical and horizontal directions when compared to an uncracked section (Rodriguez and Hooton 2003, Pease 2010). These findings are corroborated by studies by Miller and Darwin (2000) and Lindquist et al. (2005, 2006) on bridge decks where the chloride concentration in samples taken at crack locations exceeded the corrosion threshold of conventional steel reinforcement as little as nine months after the construction.

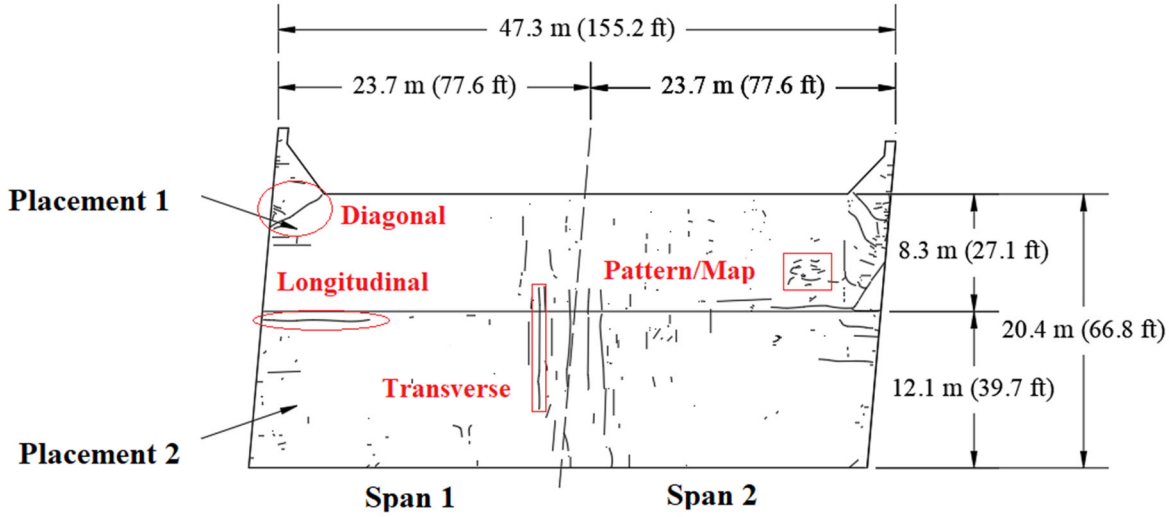


Figure 1.2—Major Bridge Deck Crack Types as shown on a crack map: Transverse, Longitudinal, Diagonal, and Map/Random Cracking (Darwin et al. 2016)

Figure 1.1—Full Depth Cracks: Virginia (left, Babaei 2007), Florida (right, Vargas 2012)

1.3.1 Transverse Cracking

Transverse cracks are the most commonly observed types of cracks in concrete bridge decks (*Durability of Concrete Bridge Decks* 1970, Krauss and Rogalla 1996, Ramey et al. 1997).

These cracks can appear soon after concrete is placed and are usually located parallel and above the transverse reinforcement (Schmitt and Darwin 1995, 1999, Saadeghvaziri and Hadidi 2002). Transverse cracks can significantly accelerate corrosion, even when epoxy-coated reinforcement is used (Darwin et al. 2011, O'Reilly et al. 2011). These cracks can be long, with some extending across the width of the roadway, and are often spaced 3 to 10 ft (1 to 3 m) apart (*Durability of Concrete Bridge Decks* 1970). Factors such as the settlement of concrete reinforcement and restraint applied to the concrete due to the closely spaced reinforcements mainly contribute to this type of cracking, with flexural moments having a minor effect.

1.3.2 Longitudinal Cracking

Longitudinal cracks are parallel to the traffic. Although this type of crack can appear in various bridge types, they most often appear in solid slab and hollow-core slab bridges *(Durability of Concrete Bridge Decks* 1970). It also has been observed that this type of crack can form along the steel girder line, which may result from rotation of the girders about their longitudinal axis (Curtis and White 2007). Frosch et al. (2003) observed full-depth longitudinal cracks caused by differential vertical movement of the adjacent prestressed box girders on bridges in Indiana. Short longitudinal cracks are also widely seen at the end of the bridges with integral abutments because of the restraint provided in the transverse direction against volume change (Schmitt and Darwin 1995, Miller and Darwin 2000, Lindquist et al. 2005, Pendergrass and Darwin 2014).

1.3.3 Diagonal Cracking

Diagonal cracks appear primarily in the corners of skewed bridges, although they are sporadically seen in other bridge types as well. Diagonal cracks usually start at a right angle at a support and propagate diagonally.

1.3.4 Map/Pattern Cracking

Map cracking can be described as a combination of random short cracks that look similar to the borders of countries on a map. These cracks can form in any deck type but are commonly seen on bridge decks with concrete overlays. Map cracks are shallow and narrow and have the potential to reduce the durability of the surface of bridge decks.

1.4 PREVIOUS WORK

1.4.1 Factors Affecting Cracking in Bridge Decks

Bridge deck cracking has been a problem throughout the country for decades. Many factors control cracking, and efforts have been underway for years to identify affecting factors and develop techniques that will reduce cracking (*Durability of Concrete Bridge Decks* 1970, Poppe 1981, Schmitt and Darwin 1995, Miller and Darwin 2000, Russell 2004). The major factors believed to control cracking of bridge decks include 1) material properties, 2) construction practices and environmental conditions, and 3) design specifications. The following sections describe these factors in more detail.

1.4.1.1 Concrete material properties

Concrete material properties play a significant role in bridge deck cracking. The fineness and type of cement used in concrete can influence cracking. Increased cement fineness is tied to increased shrinkage (Mindess et al. 2003), and the fineness of cement has increased significantly over the past 50 years (Tennis and Bhatty 2005). The volume of the paste (water and cementitious materials) has been shown to influence cracking performance significantly. In a field study conducted on 32 monolithic bridge decks, Schmitt and Darwin (1999) concluded that concrete decks with a paste volume higher than 27% exhibit significantly higher cracking than decks with lower paste contents. A comprehensive study (field and lab) conducted by the Pennsylvania DOT

also concluded that paste content plays a significant role in early-age cracking of bridge decks (Hopper et al. 2015).

"High-performance concrete" (HPC) has been used in bridge decks to improve durability. The term high-performance concrete, however, is primarily associated with lower *w/cm* ratios and higher strength. Although higher-strength concretes will have a higher tensile capacity and a denser and less porous microstructure that will provide an extra structural load carrying capacity for the bridge and lower permeability to deicing salts, the increased cement content and associated paste content can promote volume-change induced cracking in bridge decks. Hadidi and Saadeghvaziri (2005) have suggested that an increased compressive strength can be related directly to higher cracking. Schmitt and Darwin (2005) found a clear correlation between increased compressive strength and increased cracking. They related the increase to an increased modulus of elasticity and reduced creep. Shah and Weiss (2000) found that lowering the *w/cm* ratio makes the concrete more susceptible to early-age cracking. Although lowering the *w/cm* ratio increases tensile capacity, higher stresses will be developed due to higher stiffness and increased autogenous shrinkage. In a comprehensive study with the goal of reducing cracking in bridge decks, Hopper et al. (2015) concluded that placing a maximum limit of 5000 psi (34.5 MPa) on the 28-day compressive strength of concrete would be beneficial.

Absorption of the aggregates can also play a role in the shrinkage of concrete and cracking in bridge decks. According to a field investigation conducted on 116 HPC bridge decks in Ohio, all 64 bridges that exhibited little to no cracking contained aggregates with absorptions higher than 1%, and 75% of the 52 bridges exhibiting significant cracking used coarse aggregates with absorptions less than 1% (Delatte et al. 2007).

Concrete slump can also affect the cracking performance of bridge decks (Dakhil et al. 1975). Lindquist et al. (2005) concluded that slump, within the evaluated range of 1.5 to 3 in., has a measurable but relatively small influence on cracking of bridge decks. McLeod et al. (2009) and Yuan et al. (2011) observed that bridge decks constructed utilizing LC-HPC specifications, with lower slump than their matching control decks, had significantly less cracking. Research by the Pennsylvania DOT recommended a maximum slump limit of 4 in. (102 mm) to reduce settlement cracking on bridge decks (Hopper et al. 2015). In a study of the effects of synthetic fibers and rheology-modifying admixtures on settlement cracking of reinforced concrete specimens, Al-Qassag et al. (2015) observed that regardless of other factors, cracking increases with increasing slump.

1.4.1.2 Construction practices and environmental conditions

Construction practices and environmental conditions can significantly affect the cracking performance of reinforced concrete bridge decks to the extent that they can substantially neutralize the positive effects of crack-reducing technologies. Krauss and Rogalla (1996) identified a number of construction-related factors that affect cracking in bridge decks, including weather, time of casting, curing, finishing, consolidation, and placement length and sequence. They identified weather, time of casting, finishing, and curing as the most influential factors contributing to the cracking. Lindquist et al. (2005) and Pendergrass and Darwin (2014) also found a correlation between the contractor and the cracking performance of bridge decks.

1.4.1.2.1 Weather and time of casting

Weather conditions during concrete placement can significantly affect cracking performance. As discussed in Section 1.2.1.1, increased evaporation rate, which is a function of concrete and air temperatures, wind speed, and relative humidity, can cause plastic shrinkage cracking. A number of researchers have concluded that there is a direct relationship between

increased maximum air temperature and temperature range on the construction date and increased cracking (Schmitt and Darwin 1995, French et al. 1999, Bremner and Ries 2009, Yuan et al. 2011). Babaei and Purvis (1996) recommended that the temperature difference between the deck concrete at the time of casting and the girders should be kept within 22 ºF (12 ºC) to avoid early-age cracking due to thermal effects. Placing the deck during early evening or at night also can help avoid cracking due to temperature effects (Krauss and Rogalla 1996).

1.4.1.2.2 Curing

Early wet curing of concrete (10 to 20 minutes after finishing) can reduce plastic and drying shrinkage. Delayed curing also results in a poor-quality concrete since water is required in the early stages of hydration. The duration of the curing can also affect the shrinkage performance of concrete. Lindquist et al. (2008) observed decreased shrinkage in laboratory specimens when the curing period was extended from 7 to 14 days. Reynolds et al. (2009) also observed the benefit of an extended curing period (from 7 to 14 days) on reducing drying shrinkage for mixtures containing a 9 to 14% replacement of aggregate with pre-wetted, intermediate-sized LWA and a 30 to 60% replacement of cement with slag cement.

1.4.1.2.3 Finishing

Finishing procedures can affect cracking. Delays and overfinishing should be prevented, and if delays occur, it is recommended that wet burlap be used to cover the concrete to prevent drying until finishing can be continued (Pendergrass and Darwin 2014). Overfinishing can push the aggregate deeper into the concrete and work paste to the surface, resulting in plastic shrinkage cracking. Lindquist et al. (2005) noted that roller screeds tend to bring more paste to the surface than older tools, such as the vibrating screeds that were used in the 1980s. Concrete properties may differ, requiring changes in finishing techniques, a point that is not universally recognized. For

example concrete incorporating synthetic fibers, to reduce cracking can also change workability, pumpability, and placement characteristics.

1.4.1.3 Design

The structural aspects of the superstructure and the deck can affect cracking. Krauss and Rogalla (1996) and Ramey et al. (1997) suggest that cracking is more significant in bridges with continuous spans than bridges with a single span (Krauss and Rogalla 1996, Ramey et al. 1997). Lindquist et al. (2005), however, did not find a significant relationship between span type and cracking. Lindquist et al. (2005), Yuan et al. (2011), and Pendergrass et al. (2012) found that there was no increased cracking in the negative moment region of the bridge decks compared to other regions. Schmitt and Darwin (1995), Miller and Darwin (2000), and Lindquist et al. (2005) found no relationship between cracking and span length. Schmitt and Darwin (1995) found increased cracking around the abutment for bridges with the fixed-ended condition compared to those with the pin-ended condition. Lindquist et al. (2005) found that decks with overlays (silica fume or conventional) had higher cracking than monolithic decks. Partial-depth precast-prestressed concrete deck panels with a cast-in-place concrete topping have been used in Missouri, where it was found that cracking increased by a factor of two compared to full-depth cast-in-place decks (Wenzlick 2005). Hopper et al. (2015) found that bridges with steel girders experienced three times early-age cracking of prestressed girders. Properties (short-term and long-term) of prestressed girders can also affect the cracking behavior of concrete decks because creep, shrinkage, and camber of the girders can induce stresses in the concrete deck (Menkulasi et al. 2015).

As described earlier in Section 1.2.3, Dakhil et al. (1975) observed increased settlement cracking as bar size increased and cover decreased. Lindquist et al. (2005) found that decks containing No. 6 bars showed significantly greater cracking than those containing a combination of No. 5 and No. 4 or only No. 5 bars. They also found greater cracking in bridges with top

reinforcement spacing of greater than 6 in. (152 mm) compared to those with smaller spacing. The majority of state DOTs require a minimum of 2.5 or 3 in. (64 or 76 mm) top clear cover.

1.4.2 Effects of Material Properties and Crack-Reducing Technologies on Concrete Free Shrinkage

Shrinkage of concrete when it is not restrained is called free shrinkage. ASTM C157 (ASTM C157-17 2017), a test method to measure free shrinkage of concrete, is used to assess the cracking potential of candidate mixtures for bridge decks because of the correlation between concrete shrinkage and bridge deck cracking. One of the drawbacks with the test method in ASTM C157, however, is that the length-change measurements start one day after the concrete is cast. As a result, any deformation that takes place during the first day is not captured. One of the objectives of the current study is to modify this test method to address this drawback.

Symons and Fleming (1980) found a linear relationship between free shrinkage and paste content (for values of paste between 25 and 45 percent by volume) for concrete mixtures made both with ordinary portland cement and with different replacement levels of cement with fly ash. Deshpande et al. (2007) showed that when the volume of paste is increased, drying shrinkage increases. Lindquist et al. (2005) investigated the effects of different curing periods (7 and 14 days) on the free shrinkage of concrete mixtures containing different replacement levels of cement with Grade 100 and 120 slag and Class F fly ash. They found significant benefits of extending the curing period from 7 to 14 days in reducing free shrinkage of mixtures containing slag but no benefit for mixtures containing fly ash. West et al. (2010) observed that concrete mixtures made with a Type II coarse-ground cement experienced less shrinkage than the mixtures made with a finer Type I/II cement. They also observed a reduction of shrinkage for mixtures made with either cement type when the curing period was increased from 7 to 28 days. Yuan et al. (2011) saw the benefit of

increasing the curing period from 7 to 28 days in reducing the free shrinkage of mixtures containing fly ash.

Numerous technologies have been examined to minimize the shrinkage of concrete. Internal curing, providing water within the concrete through the use of pre-wetted LWA has been used efficiently to mitigate concrete shrinkage (Weber and Reinhardt 1997, Reynold et al. 2009, Browning et al. 2011, Pendergrass and Darwin 2014).

Supplementary cementitious materials (SCMs) such as slag, fly ash, and silica fume have been used in concrete industry for decades. Use of SCMs, as environmentally friendly materials, in concrete construction industry results in a substantial reduction in CO_2 emissions. SCMs have other benefits when used in concrete, such as reduced concrete permeability (Rose 1987, Maage and Sellevold 1987). The lower permeability is due to a change in the pore structure of the cement paste matrix. Yuan et al. (2015) found that partial replacement of cement with slag results in a reduction of concrete shrinkage.

Shrinkage-reducing admixtures (SRAs) function by reducing the surface tension of pore water, resulting in a reduction in capillary stress (a principal cause of drying shrinkage). SRAs are added in dosages ranging from 0.5 to 2% by the weight of binder. High SRA dosages (1.0 to 2.0 percent by weight of cement), however, may cause durability problems. Lindquist et al. (2008), Pendergrass and Darwin (2014), and Pendergrass et al. (2017) found that the use SRAs decreases the hardened air content of concrete, increases the size of entrained-air bubbles, and reduces the stability of the air void system within the concrete as a result of reducing the surface tension of water.

CaO and MgO shrinkage-compensating admixtures (SCAs) function by inducing early-age expansion in concrete through the formation of expansive hydration products, $Ca(OH)_2$ or

$Mg(OH)_2$. SCAs have been shown to substantially reduce the tendency of concrete to develop shrinkage strains (Mo et al. 2011). This benefit, however, can only be realized if adequate restraint is provided in the structure containing an expansive concrete. ACI Committee 223 (2010) provides guidelines on how to determine the appropriate amount of restraint for systems built with expansive concrete.

Researchers have evaluated the combined effects of IC and SCMs, both in laboratory tests and bridge decks. For concretes with moderate *w/cm* ratios, Browning et al. (2011) demonstrated the effectiveness of combining slag cement and IC on reducing shrinkage. Pendergrass and Darwin (2014) showed the effectiveness of combining slag, silica fume, and IC on reducing shrinkage of concretes. De la Varga et al. (2012) evaluated the effectiveness of using IC to counteract shrinkage of low *w/cm* concrete mixtures containing high-volume fly ash replacements. Bridge decks with binary (cement and slag) or ternary (cement, slag, and silica fume, or cement, fly ash, and silica fume) concrete mixtures combined with IC have been constructed in Ohio and Indiana (Dellatte et al. 2007, Barrette et al. 2015). Khayat and Mehdipour (2016) investigated the combined effects of a CaO SCA and SCMs in mortar and found that the expansion caused by CaO is more pronounced when high volume replacements of SCMs are incorporated in the concrete.

1.5 OBJECTIVE AND SCOPE

In this study, the effects of supplementary cementitious materials (slag and silica fume) as a partial replacement for portland cement, crack-reducing technologies such as internal curing through the use of pre-wetted lightweight aggregate, and CaO and MgO shrinkage-compensating admixtures (SCAs) or expansive additives, and different combinations of these technologies on shrinkage of a series of concrete mixtures using a modified ASTM C157 test method are investigated.

In addition to the laboratory work, the results of crack surveys conducted on 50 bridge deck placements having a similar deck and superstructure type (monolithic conventional deck built with 100% portland cement supported by steel girders) are used to evaluate the effects of different factors contributing to the long-term cracking, and further analysis is performed to identify the factors having the most significant effects. These factors include mixture proportions, material properties, environmental conditions (mainly temperature and time of placement), and construction practices. In addition, the results of a six-year field evaluation of 12 other bridge deck placements are used to evaluate the effects of deck type (monolithic, silica fume overlay, or precast concrete deck panels), superstructure type (steel or prestressed concrete girders), concrete mixture proportions (blend of cementitious materials), and crack-reducing technologies (synthetic fibers) on long-term cracking performance.

CHAPTER 2 - COMBINED EFFECTS OF INTERNAL CURING, SCMS, AND SHRNKAGE SHRINKAGE-COMPENSATING ADMIXTURES ON CONCRETE SHRINKAGE

2.1 GENERAL

Internal curing (IC) using pre-wetted lightweight aggregate has often been used to improve the cracking performance of concretes with low water-to-cementitious materials (*w/cm*) ratios (<0.42) where autogenous shrinkage and self-desiccation are of concern. Shrinkage-compensating admixtures (SCAs) are also used to improve the cracking performance of concrete. In this chapter, a modified version of ASTM C157, in which length-change measurements begin 5½ ± ½ hr after casting concrete, is developed and used to evaluate the effects of internal curing obtained through the use of intermediate-size pre-wetted lightweight aggregate, partial replacements of cement with the supplementary cementitious materials slag cement and silica fume (SCMs), incorporation of calcium oxide-based and magnesium oxide-based SCAs, and combinations of IC, SCMs, and SCAs using eleven concrete mixtures with a moderate *w/cm* ratio (0.45). The results show that the modified ASTM C157 method helps to capture the early-age behavior of concrete mixtures. IC provided by partial replacement of total aggregate with intermediate-size pre-wetted LWA is effective in reducing 0 to 20-day drying shrinkage in concrete made with moderate *w/cm* ratios. The opposite is observed regarding 20 to 180-day drying shrinkage where the mixtures containing IC experienced greater drying shrinkage than their pairs with no IC. Partial replacements of cement with slag cement and silica fume induce increased first-day expansion and reduce shrinkage. A further increase in first-day expansion and a reduction in shrinkage is obtained when internal curing is used in conjunction with slag cement and silica fume. The SCAs evaluated in this study reduce the tendency to develop shrinkage strain. The calcium oxide-based SCA induces the more rapid expansion of greater magnitude, while the magnesium oxide-based SCA expands more gradually. When the calcium oxide-based SCA is incorporated in a mixture containing SCMs or

SCMs and IC, expansion is further increased. The same observation cannot be made for mixtures incorporating SCMs with the magnesium oxide-based SCA.

2.2 INTRODUCTION

As discussed in Chapter 1, it is well established that concrete shrinkage is the main cause of cracking in bridge decks. Therefore, reducing concrete shrinkage can greatly improve the service life of concrete bridge decks. Numerous technologies have been examined by researchers to minimize the shrinkage of concrete, such as the use of pre-wetted lightweight aggregate (LWA) as an internal curing (IC) agent (Weber and Reinhardt 1997), the use of supplementary cementitious materials (SCMs) (Yuan et al. 2015), and the use of shrinkage-compensating admixtures (SCAs) (Mo et al. 2011). Researchers have also evaluated the combined effects of IC and SCMs, both in laboratory tests and bridge decks. Browning et al. (2011) demonstrated the effectiveness of combining slag cement and IC on reducing shrinkage of concretes with moderate water-to-cementitious materials (*w/cm*) ratios. Pendergrass and Darwin (2014) and Pendergrass et al. (2017) showed the effectiveness of combining slag cement, silica fume, and IC on reducing shrinkage of concretes with moderate *w/cm* ratios. De la Varga et al. (2012) evaluated the effectiveness of using IC to counteract shrinkage of low *w/cm* concrete mixtures containing high-volume fly ash replacements. Bridge decks with binary (cement and slag cement) or ternary (cement, slag cement, and silica fume, or cement, fly ash, and silica fume) concrete mixtures combined with IC have been constructed in Indiana (Barrett et al. 2015) and Ohio (Delatte et al. 2007) in recent years. Khayat and Mehdipour (2016) investigated the combined effects of a CaO-based expansive agent and SCMs in mortar and found that the expansion caused by CaO is more pronounced when high volume replacements of SCMs are used.

In this chapter, the effects of SCMs, IC, two types of SCAs (MgO-based or CaO-based),

and different combinations of SCMs, IC, and SCAs are evaluated based on the early-age deformation and long-term drying shrinkage of concretes with a *w/cm* ratio of 0.45. In the presence of water, MgO expands when converted to $Mg(OH)_2$. Likewise, CaO expands when converted to $Ca(OH)_2$. Since the mixtures evaluated in this study have a moderate *w/cm* ratio, it is theorized that the internal curing water provided by pre-wetted LWA at early ages is not used to eliminate premature self-desiccation due to the hydration process (autogenous shrinkage), as there is not a measurable early-age shrinkage on mixtures without internal curing. Instead, the incorporation of pre-wetted LWA, SCMs, and shrinkage-compensating admixtures may have other significant benefits. First, when MgO- or CaO-based shrinkage-compensating admixtures are used, the IC water may be consumed by the hydration of these additives, and as a result, a greater early-age expansion may be induced that helps counteract later-age drying shrinkage. Secondly, the IC water provided by pre-wetted LWA can be beneficial for later hydration of SCMs that can eventually reduce later-age drying shrinkage. Combining IC with shrinkage-compensating admixtures in concrete mixtures with SCMs and moderate *w/cm* ratios, which are not susceptible to autogenous shrinkage, differs from other studies where IC water is mainly consumed by the self-desiccation of concrete with lower *w/cm* ratios at early ages.

2.3 EXPERIMENTAL WORK

2.3.1 Materials

Type I/II portland cement was used for all mixtures in this study. Grade 100 slag cement and silica fume were used as partial replacements by volume of portland cement in some mixtures. The chemical compositions of cementitious materials used are summarized in Table 2.1.

Table 2.1—Chemical composition (percentage) and specific gravity of cementitious materials

Component	Portland cement	Grade 100 slag cement	Silica fume
SiO_2	20.5	43.46	94.49
Al_2O_3	4.97	8.61	0.07
Fe_2O_3	3.57	0.37	0.1
CaO	62.46	31.13	0.53
MgO	2.06	12.5	0.62
SO_3	2.49	2.24	0.11
Na_2O	0.35	0.21	0.09
K_2O	0.49	0.4	0.54
TiO_2	0.29	0.32	-
Mn_2O_3	0.11	0.35	0.02
SrO	0.26	0.04	0.01
Cl^-	-	-	0.05
LOI	2.6	0.37	3.21
Total	100.25	99.9	99.9
Specific Gravity	3.2	2.86	2.2

Two granite aggregates, with maximum sizes of 1 and ¾ in. (25 and 19 mm), were combined to achieve optimal gradations. The 1-in. (25-mm) granite had an absorption of 0.50% and a specific gravity (saturated-surface dry, SSD) of 2.61. The ¾-in. (19-mm) granite had an absorption of 0.58% and a specific gravity (SSD) of 2.60. River-run sand and pea-gravel were used as fine aggregate. The absorptions for sand and pea-gravel were 0.47% and 1.42%, respectively, and the specific gravities (SSD) were 2.62 and 2.63, respectively. Pea-gravel sized pre-wetted lightweight aggregate (LWA) was used to provide internal curing in some mixtures. The LWA was immersed in water for 72 hours prior to mixing. The absorption and pre-wetted surface-dry (PSD) specific gravity for LWA after 72 hours of soaking were 23.99 % and 1.71, respectively. A centrifuge was used to bring the soaked LWA to a PSD condition according to the procedure developed by Miller et al. (2014). The amount of internal curing water provided by the pre-wetted LWA was, on average, 6.6 lb per 100 lb of cementitious materials.

Two types of shrinkage-compensating admixtures (SCA) were used, both in solid (powder)

form. The active component of SCA 1 is magnesium oxide (MgO), which expands as it reacts with mixing water. SCA 1 also contains a shrinkage-reducing admixture (SRA) that lowers the drying shrinkage of concrete. The majority of SCA 2 is calcium oxide (CaO), which reacts with water and causes expansion when added to concrete mixtures. A tall oil-based air-entraining admixture (AEA) was used in all mixtures. A polycarboxylate-based high-range water-reducing admixture was added when necessary to obtain the desired concrete slump.

2.3.2 Concrete Mixtures

Eleven mixtures were used to evaluate the effects on shrinkage of pre-wetted LWA, SCMs, and SCAs. Mixture proportions are shown in Table 2.2. The mixture designated as "Control" contained only portland cement as the binder and no pre-wetted LWA, SCM, or SCA. The mixture containing a 10% volume replacement of total aggregate with pre-wetted LWA and only portland cement as a binder is designated as "IC." Mixtures containing 7.5% SCA 1 or 6% SCA 2 by weight of cement are labeled as "SCA 1" and "SCA 2," respectively. The mixtures containing a 10% volume replacement of total aggregate with pre-wetted LWA and 7.5% SCA 1 or 6% SCA 2 by weight of cement are designated as "IC-SCA 1" and "IC-SCA 2," respectively. The mixture containing a 30% volume replacement of cement with slag cement and a 3% volume replacement of cement with silica fume is designated as "SCM." The mixture containing a 10% volume replacement of total aggregate with pre-wetted LWA, a 30% volume replacement of cement with slag cement, and a 3% volume replacement of cement with silica fume is designated as "IC-SCM." The mixture containing 30% slag cement and 3% silica fume replacements of cement plus 6% SCA 2 is designated as "SCM-SCA 2," and the mixtures containing the pre-wetted LWA, 30% slag cement and 3% silica fume replacements of cement and 7.5% SCA 1 or 6% SCA 2 are designated as "IC-SCM-SCA 1" and "IC-SCM-SCA 2," respectively. A mixture containing SCMs

and SCA 1 was not tested.

Table 2.2—Mixture proportions (lb/yd^3).

Mixtures	*w/cm*	Cement	Grade 100 slag cement	Silica fume	Water	SCA	Sand	Pea gravel	Pre-wetted LWA	Coarse aggregate		AEA
										3/4 in.	1 in.	(fl oz/yd^3)
Control	0.45	520	0	0	234	0	1035	540	0	964	477	5.4
IC	0.45	520	0	0	234	0	1235	129	176	818	529	11.8
SCA 1	0.45	516	0	0	232	39	1027	529	0	957	473	4.1
IC-SCA 1	0.45	516	0	0	232	39	1226	128	175	812	525	8.4
SCA 2	0.45	516	0	0	232	31	1027	539	0	957	473	10.2
IC-SCA 2	0.45	516	0	0	232	31	1226	128	175	812	525	8.4
SCM	0.45	365	150	12	232	0	1097	363	0	752	695	10.0
IC-SCM	0.45	359	146	11	232	0	1226	128	175	812	525	7.1
IC-SCM-SCA 1	0.45	359	146	11	232	39	1226	128	175	812	525	9.1
SCM-SCA 2	0.45	359	146	11	232	31	1027	539	0	957	473	11.7
IC-SCM-SCA 2	0.45	359	146	11	232	31	1226	128	175	812	525	10.2

Concrete properties are summarized in Table 2.3. This study is part of a broader research project at the University of Kansas that aims to develop low-cracking high-performance concrete (LC-HPC) for bridge decks. Therefore, the mixtures had high air contents (between 6.75 and 9.25%, with the exception of one mixture containing 3.25% air and one with 10.5% air). Slump ranged from 0.75 in. (19 mm) to 4.75 in (121 mm). All mixtures had a *w/cm* ratio of 0.45. The strengths of the mixtures ranged from 3530 to 5600 psi (24.4 to 38.6 MPa).

Paste contents, based on an air content of 8%, were 23.5 or 24.0% of total volume, except for the SCM mixture, which contained 24.4% paste. The mixtures were designed to remain within a small range of paste contents by volume to minimize the effect of paste volume on shrinkage.

Table 2.3—Concrete properties.

Mixtures	Air Content, %	Slump, in.	Temp, °F	Unit Wt., lb/yd^3	Paste content, %	IC Water, lb/100 lb of binder	Strength, psi
Control	7.50	2	75	139.2	23.5	0	4440
IC	10.50	4¾	69	133.9	23.5	6.6	3260
SCA 1	7.50	1¼	74	-*	23.5	0	4540
IC-SCA 1	3.25	¾	72	146.7	23.5	6.6	5600
SCA 2	6.75	1¾	77	133.3	23.5	0	4290
IC-SCA 2	7.75	2	72	138.1	23.5	6.6	3530
SCM	9.25	3½	71	139.6	24.4	0	4190
IC-SCM	8.00	3¼	73	134.0	24.0	6.6	4940
IC-SCM-SCA 1	7.25	1½	74	136.7	24.0	6.6	5000
SCM-SCA 2	7.00	1¾	70	142.3	24.0	0	4340
IC-SCM- SCA 2	7.75	1¾	65	136.1	24.0	6.6	5440

* Data not available

2.3.3 Modified ASTM C157 Test

The method used to measure free shrinkage was based on ASTM C157-17, *Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete* (ASTM C 157 2017). The modified method differs from that in ASTM C157 in that the specimens are demolded and the first measurement is taken 5½ ± ½ hr after casting, rather than 24 hours after water is added to the mixture. Early demolding is used because demolding after 24 hours does not capture volume changes that occur during the first day. Early-age volume changes can be especially important for concrete that undergoes expansion. The 5½ ± ½ hr time of demolding approximates the time of final set. This time was selected after multiple trials and matches observations by Pease (2005) who measured final set times for mortars with 0.3 and 0.5 *w/c* ratios as 3 and 6 hours, respectively.

Three prismatic specimens with dimensions of 3 by 3 by 11.25 in. were cast from each mixture. After demolding at 5½ ± ½ hr, the specimens were stored in lime saturated water until 14 days after casting, after which they were dried in an environmentally controlled room with a temperature of 73° ± 3°F and relative humidity of 50 ± 4%. The initial lengths of specimens were measured immediately after removing the specimens from the molds. Three to four length

measurements were made within the first 24 hours, followed by daily measurements during curing and during the first 30 days of drying; measurements were then taken every other day between 30 and 90 days, and weekly thereafter.

Figure 2.1 and Figure 2.2 illustrate the usefulness of the modified method in measuring early age deformation – deformation that is more important for some mixtures than others. The figures show the deformations (based on an average of three specimens) for the mixtures designated as Control and SCA 2. The mixture proportions and concrete properties are listed in Table 2.2 and Table 2.3, respectively. Six specimens were cast for each mixture. Three specimens were demolded in accordance with the ASTM C157 24 hr after casting and three were demolded 5½ ± ½ hr after casting. Fig. 1 and 2 show the average deformations as a function of time after casting for the two mixtures. There is virtually no difference between the deformations for the Control mixture, regardless of the demolding time, either after curing at 14 days (Figure 2.1) or after 180 days of drying (Figure 2.2, 194 days after casting). This is not the case, however, for the SCA 2 mixture, which exhibits less swelling when demolded at 24 hours than when demolded 5½ ± ½ hr after casting. These results indicate the advantage of the modified procedure for mixtures containing constituents that may affect early age (first day) behavior. The results presented in the balance of this chapter were obtained using the modified method.

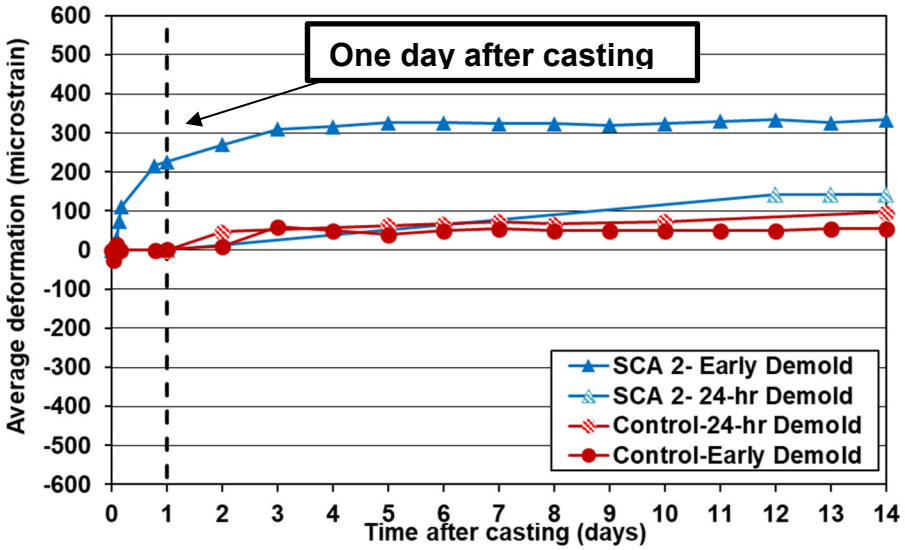


Figure 2.1—Deformations during curing period comparing a mixture that does not exhibit early age expansion (Control) and one that does (SCA 2) (shrinkage is negative; swelling is positive)

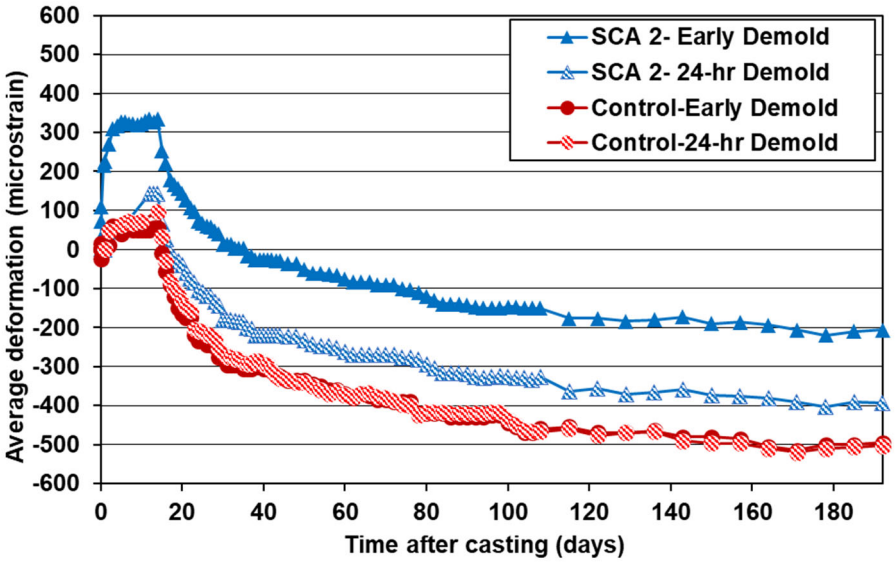


Figure 2.2—Deformations during curing period and during first 180 days of drying comparing a mixture that does not exhibit early age expansion (Control) and one that does (SCA 2)

Other test methods have been developed to evaluate early age deformation of cementitious mixtures, which can be exemplified by ASTM C1698, *Standard Test Method for Autogenous Strain of Cement Paste and Mortar* (ASTM C1698 2014) and that used by Zhang, Hou, and Han

(2012). In ASTM C1698, a corrugated polyethylene tube is filled with cement paste or mortar and then closed. After final set, the specimen is placed on a dilatometer bench for length measurements. This test method is widely accepted due to the simple operation and satisfactory precision. Because of the dimensions of the tube, however, this procedure is only capable of measuring the early age deformation of cement paste or mortar but not mixtures containing coarse aggregate. This prevents the test from evaluating concrete where coarse aggregate may affect shrinkage, such as for concretes made with highly absorptive aggregates (Browning et al. 2011). In addition, the method cannot be used to measure drying shrinkage since the specimen is enclosed in a polyethylene tube. This makes measurement of the long-term deformation of specimens that includes both the curing and drying periods impossible.

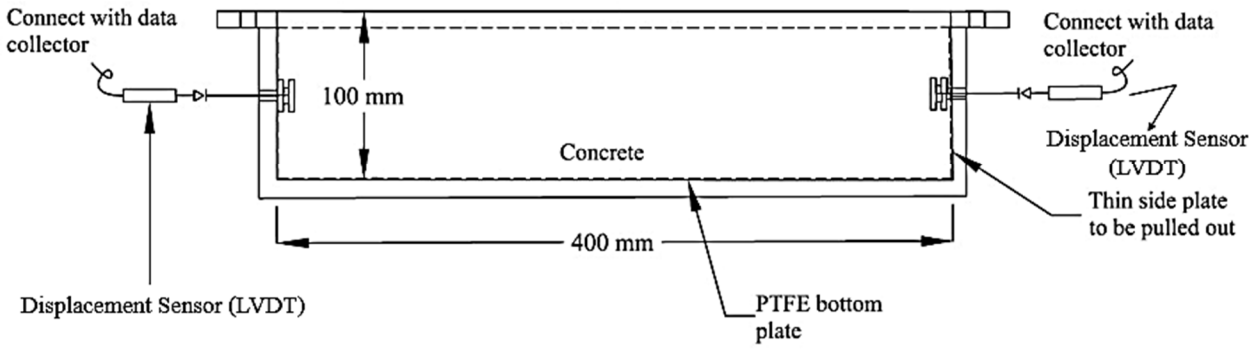


Figure 2.3—The test method used by Zhang, Hou, and Han (2012)

Figure 2.3 shows a schematic of the test setup used by Zhang, Hou, and Han (2012). The bottom of the mold is covered by thin vinyl sheets to reduce friction between the specimen and the mold. After initial set, the four side plates are removed, leaving 0.08-in. (2-mm) gaps between the specimen and the mold. This allows the mounting of an LVDT at both ends of the specimen to measure the deformation. This test method can be used to measure the early-age deformation of concrete specimens and allows for the integration of an automatic data acquisition system. However, for consistency of the length change measurements, the two LVDTs and the mold must

be in place throughout the testing period (curing and drying). Therefore, this method is not practical for simultaneous long-term drying shrinkage measurements of a large number of specimens.

2.4 EXPERIMENTAL RESULTS AND DISCUSSION

Table 2.4 summarizes the average deformation for the mixtures at the end of the first day, 14 days (end of wet-curing period), 34 days (20 days of drying), and 194 days after casting (180 days of drying). Table 2.4 also includes the drying shrinkage for the first 20 days and for 20 to 180 days of drying. Tables C.1 through C.11 in Appendix C include individual measurements taken from each specimen used to calculate the average values listed in Table 2.4.

Table 2.4—Summary of deformations (microstrain)*. Average of three specimens unless noted.

Mix Designation	During Curing Period		Deformation after		Drying Shrinkage	
	First Day	14 Days (no drying)	20 Days of drying	180 Days of drying	After 20 days of drying	Between 20 and 180 days of drying
Control	2	55	-295	-498	350	203
IC	97	100	-227	-527	327	300
SCA 1	64	210	-37	-173	247	137
IC-SCA 1	88	220	67	-213	153	280
SCA 2	227	333	3	-204	330	207
IC-SCA 2**	265	360	60	-240	300	300
SCM	50	63	-170	-347	233	177
IC-SCM**	68	80	-100	-340	180	240
IC-SCM-SCA 1	75	210	60	-110	150	170
SCM-SCA 2	358	413	120	-107	293	227
IC-SCM-SCA 2	402	523	353	107	170	247

* Shrinkage is negative; swelling is positive, ** Average of two specimens

In this chapter, three replicate specimens were tested for each mixture. At 180 days of drying, the ranges in deformation for specimens from the same mixture were no more than 56 microstrain, except for the SCA 2 mixture, which had a range of 220 microstrain. These values are well below 496 microstrain, the maximum range stated in ASTM C157 to be expected in 95 % of sets tested. The standard deviations (1s) for specimens from a single mixture were less than or equal to 39

microstrain, except for the SCA 2 mixture, which had a standard deviation of 117 microstrain. ASTM C157 has a 1s limit of 48 microstrain for results obtained using three specimens.

In this chapter, Student's t-test was used to determine whether the difference in deformation between two mixtures was due to random variation or due to an actual difference in behavior. When sample sizes are small and the population standard deviation is unknown, the t-test helps indicate whether the difference in the means of two samples, X_1 and X_2, represents a difference in the population means, μ_1 and μ_2. There are several ways to describe the outcome of a t-test. In this chapter, the results are compared based on the *p* value, which is the probability of obtaining an effect, in this case a difference in average deformations, at least as large as observed in the sample data, assuming that there is, in fact, no difference (Johnson et al. 2005). Traditionally, values of *p* less than 0.02 or 0.05 and sometimes 0.10 are treated as indicative that the differences between two means are statistically significant (that is, unlikely to have arisen by chance). Values above 0.20 are universally accepted as indicating that the difference between means is not statistically significant (that is, likely to have arisen by chance). In the comparisons that follow, the differences are statistically significant unless otherwise noted.

2.4.1 TOTAL DEFORMATION

2.4.1.1 Deformations during the first day of curing

Figure 2.4 shows the deformations for the mixtures during the first day. The order of the mixtures in the legend of this figure, and the others in this chapter, match the order of the curves. The mixtures expanded 50 microtrain or more, with the exception of the Control mixture, which exhibited an expansion of only 2 microstrain. The figure shows that internal curing, obtained through the use of intermediate-size pre-wetted LWA, increased expansion during the first 24 hours. In every pair of mixtures with and without IC, the IC mixture exhibited the greater expansion. Both SCAs caused first-day expansion, with the SCA 2 mixtures undergoing greater

expansion than the SCA 1 mixtures. The mixtures with SCA 1 exhibited a minimum expansion of 64 microstrain while those with SCA 2 exhibited a minimum expansion of 227 microstrain. Expansion increased more when SCA 2 was used in conjunction with SCMs or IC and SCMs. Incorporating IC and SCMs had less effect on the SCA 1 mixtures. Khayat and Mehdipour (2015) postulate that the extra expansion observed when SCMs are used in conjunction with a CaO-based SCA, such as SCA 2, is due to lower resistance of mixtures containing high volume SCMs at early ages. This hypothesis, however, does not hold for MgO-based SCA 1, suggesting that the reason for the extra expansion requires additional study.

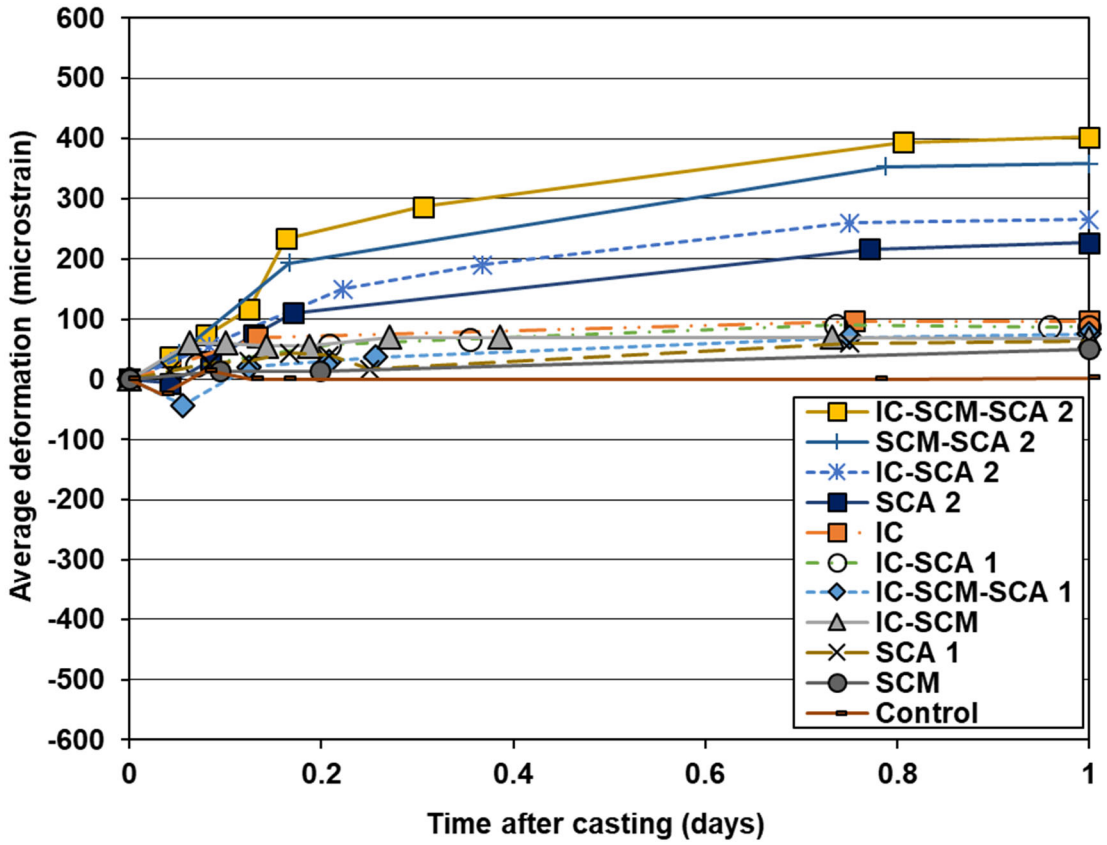


Figure 2.4—Deformations during first day of curing

In terms of other comparisons, the IC mixture had 95 microstrain more expansion than the Control mixture while the SCM mixture had 48 microstrain more expansion than the Control

mixture. The IC-SCM mixture exhibited 66 microstrain more expansion than the Control mixture.

The IC-SCA 1 mixture exhibited 24 microstrain more expansion than the SCA 1 mixture. The IC-SCM-SCA 1 mixture exhibited 11 microstrain more expansion than SCA 1 and 13 microstrain less expansion than the IC-SCA 1 mixture, but these differences are not statistically significant, indicating little effect of combining IC or IC and SCMs with an MgO-based SCA. The IC-SCA 2 mixture exhibited 38 microstrain more expansion than the SCA 2 mixture, a difference that is also not statistically significant. The SCM-SCA 2 and IC-SCM-SCA 2 mixtures exhibited, respectively, 131 and 137 microstrain more expansion than the SCA 2 mixture, indicating that incorporating the two SCMs with a CaO-based SCA will have a major effect on expansion. Future research will be needed to determine the individual roles of slag cement and silica fume.

These results suggest that the first-day expansion observed in some of the mixtures, if properly restrained, can help to reduce the stresses caused by drying. ACI 223R-10, *Guide for the Use of Shrinkage-Compensating Concrete* (ACI Committee 223 2010), provides detailed guidelines on how to determine the appropriate amount of restraint for systems built with expansive concrete.

2.4.1.2 Deformations during 14 days of curing

Figure 2.5 shows the deformations for mixtures as a function of time after casting during the 14-day curing period. As shown in the figure and Table 2.4, the mixtures without an SCA exhibited little expansion after the first day. In contrast, all mixtures containing SCA 1 (SCA 1, IC-SCA 1, and IC-SCM-SCA 1) and three out of four mixtures containing SCA 2 (SCA 2, IC-SCA 2, and IC-SCM-SCA 2), exhibited an additional expansion of at least 95 microstrain. The Control, IC, SCM, and IC-SCM mixtures exhibited additional expansions of 53, 3, 13, and 12 microstrain, respectively, after the first day. In contrast, the SCA 1, IC-SCA 1, and IC-SCM-SCA 1 mixtures exhibited additional expansions of 146, 132, and 135 microstrain, respectively, and the

SCA 2, IC-SCA 2, SCM-SCA 2, and IC-SCM-SCA 2 mixtures exhibited an additional expansion of 106, 95, 55, and 121 microstrain after the first day. In general, the SCA 2 mixtures exhibited a much greater percentage of the 14-day expansion during the first day after casting than the SCA 1 mixtures, which continued to expand gradually during the curing period.

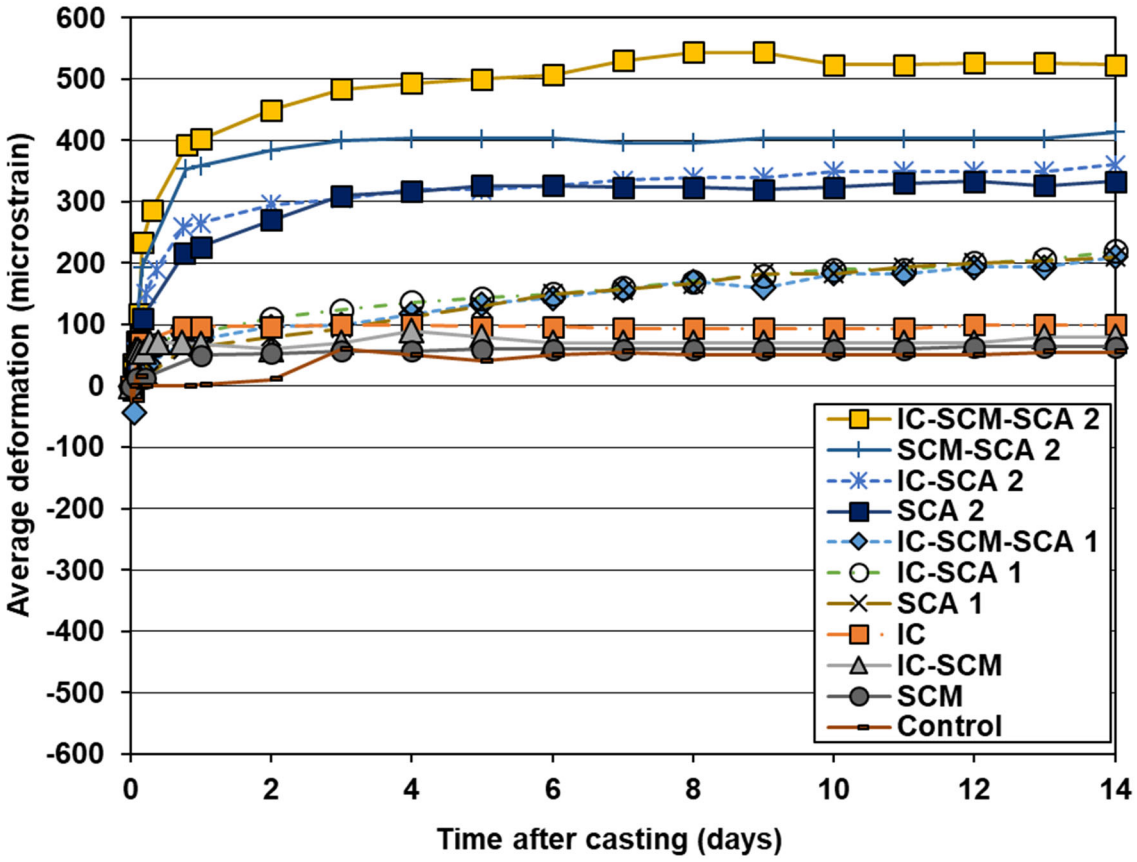


Figure 2.5—Deformations during 14-day curing period

2.4.1.3 Deformations through 20 days of drying

Figure 2.6 shows the deformations of the mixtures during the 14-day curing period and the first 20 days of drying as functions of time. The results indicate that incorporating IC, SCMs, and SCAs, individually or in combination, reduces the tendency to develop shrinkage strain during a period in which shrinkage takes place at the greatest rate and concrete is most susceptible to

cracking. This reduction is realized because, in general, when these technologies are used alone or in combination, they induce greater early-age expansion and, in all but one case, reduce drying shrinkage.

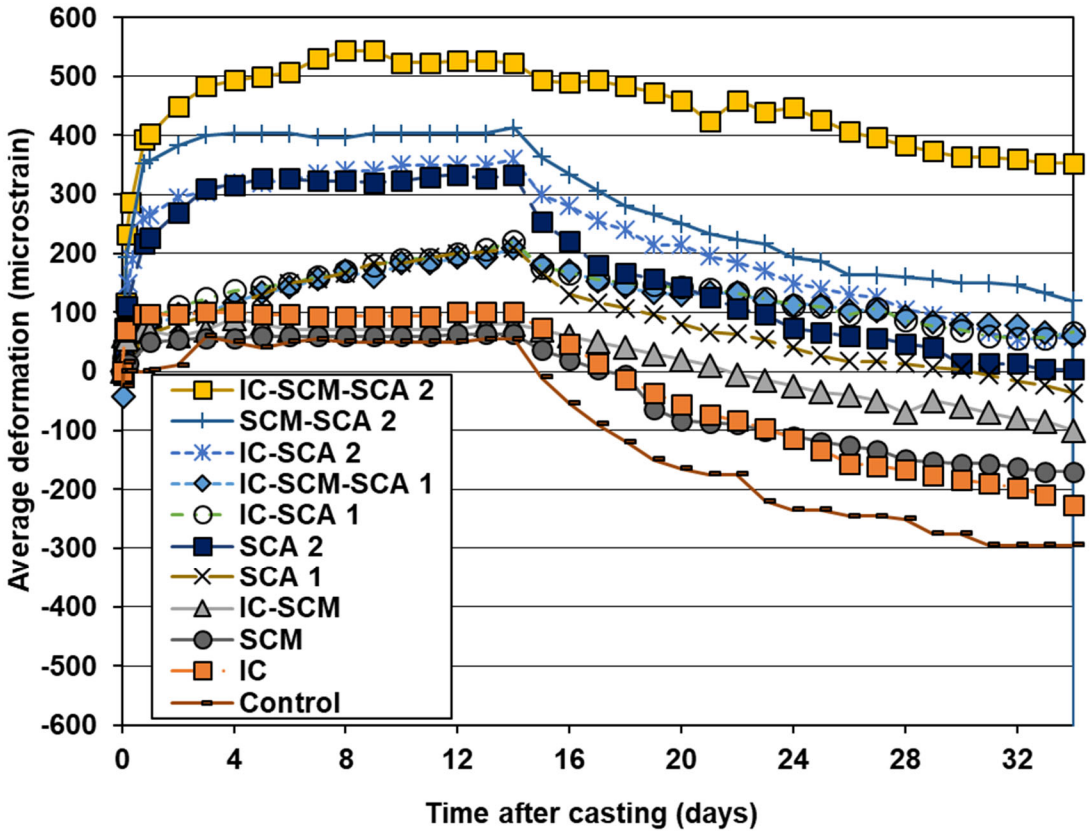


Figure 2.6—Deformations during 14-day curing period and first 20 days of drying

Looking only at the shrinkage that occurs after drying begins (discussed in more detail later in this chapter), the use of internal curing in the mixture with 100% portland cement resulted in reduced shrinkage compared to the Control mixture. Partial replacements of cement with slag cement and silica fume (SCM mixture) also reduced shrinkage, with a further reduction observed when internal curing and partial replacements with slag cement and silica fume were used together (IC-SCM mixture). In terms of total deformation, the mixtures incorporating SCAs uniformly exhibited the lowest tendency to develop shrinkage strains. In every case for pairs of mixtures with

and without IC, the internally cured mixture had the lower tendency to develop shrinkage strains.

The use of IC or IC and SCMs in the mixtures containing SCA 1 reduced the shrinkage during the first 20 days of drying. A similar trend was observed for the mixtures containing SCA 2 where, in all cases, the use of IC, SCMs, or IC and SCMs in mixtures containing SCA 2 reduced shrinkage during the first 20 days of drying.

The SCA 1 mixtures (SCA 1, IC-SCA 1, and IC-SCM-SCA 1) experienced less shrinkage than the matching SCA 2 mixtures (SCA 2, IC-SCA 2, and IC-SCM-SCA 2) once the drying started. The difference is likely tied to the fact that SCA 1 incorporates a shrinkage-reducing admixture, while SCA 2 does not.

2.4.1.4 Deformations through 180 days of drying

Figure 2.7 shows the deformations during the 14-day curing period plus the first 180 days of drying as functions of time. The Control and IC mixtures had the highest shrinkage, and the mixture with pre-wetted LWA, SCMs, and SCA 2 (IC-SCM-SCA 2) continued to exhibit a net expansion – the only mixture to do so. The mixtures containing SCA 1 continued to exhibit lower rates of drying than the corresponding mixtures containing SCA 2. Unlike through 20 days drying, in which all mixtures containing IC exhibited less net deformation than their comparable pairs with no IC, after 180 days of drying the mixture with IC caught up to the ones with no IC suggesting that incorporation of IC increased the drying shrinkage after 20 days.

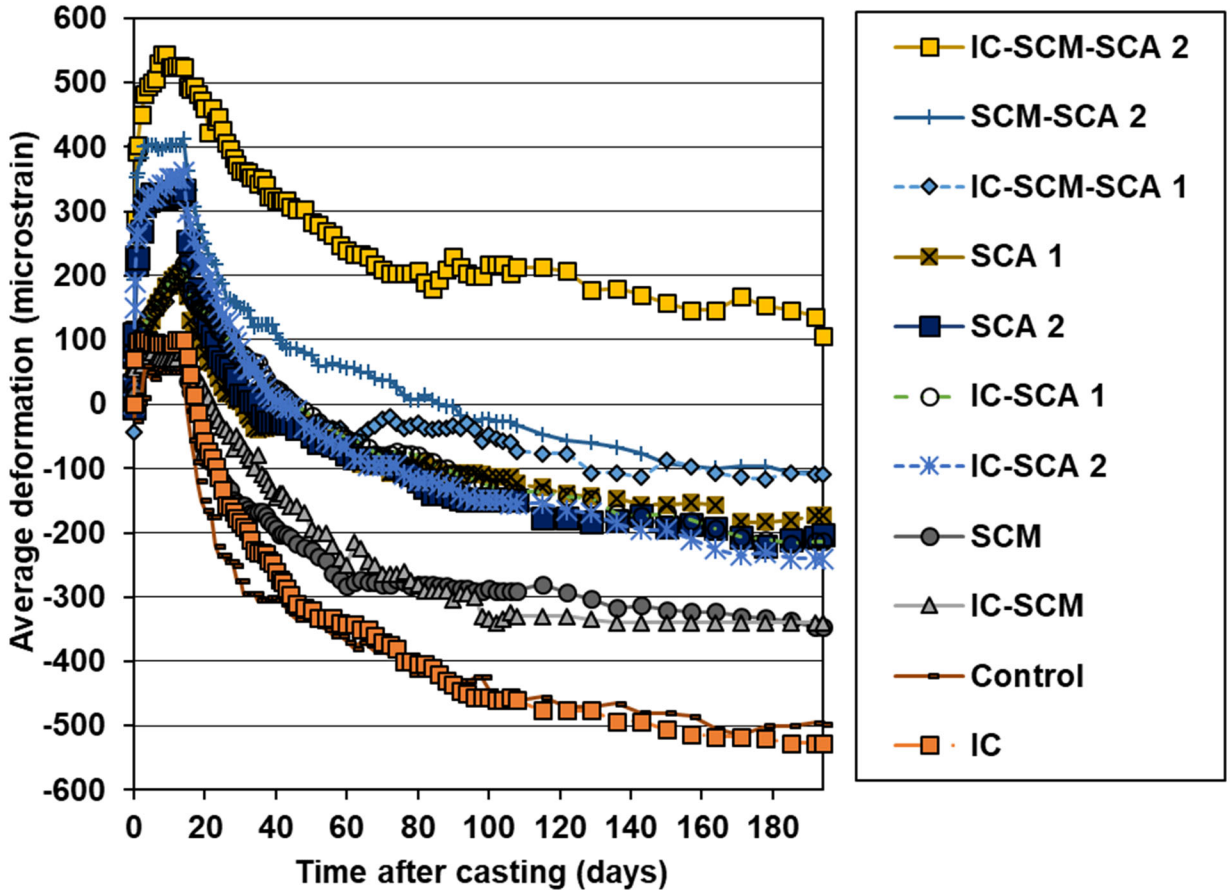


Figure 2.7—Deformations during 14-day curing period and 180 days of drying

2.4.2 DRYING SHRINKAGE

2.4.2.1 First 20 days of drying

Figure 2.8 shows the shrinkage for the eleven mixtures in the study that occurred during the first 20 days of drying. The Control mixture experienced the greatest drying shrinkage during this period. The 20-day drying shrinkage was reduced when a portion of cement was replaced with slag cement and silica fume and reduced further when IC was used in combination with slag cement and silica fume. The three mixtures containing both IC and SCMs (IC-SCM, IC-SCM-SCA 1, and IC-SCM-SCA 2) experienced less drying shrinkage than seven out of the eight mixtures without both IC and SCMs (Control, IC, SCA 2, IC-SCA 2, SCA 1, SCM, and SCM-

SCA 2). In every pair of mixtures with and without IC, the use of IC, in all cases, reduced the 20-day drying shrinkage, an effect that was more pronounced for mixtures containing SCMs and the mixture containing SCA 1. The reduction of the early-age drying shrinkage through the use of IC or SCMs alone or in combination, when coupled with early-age expansion, resulted in a reduction in the tendency to develop shrinkage strain in these mixtures, as shown in Figure 2.5 through Figure 2.7 and Table 2.4, indicating the effectiveness of combining these technologies.

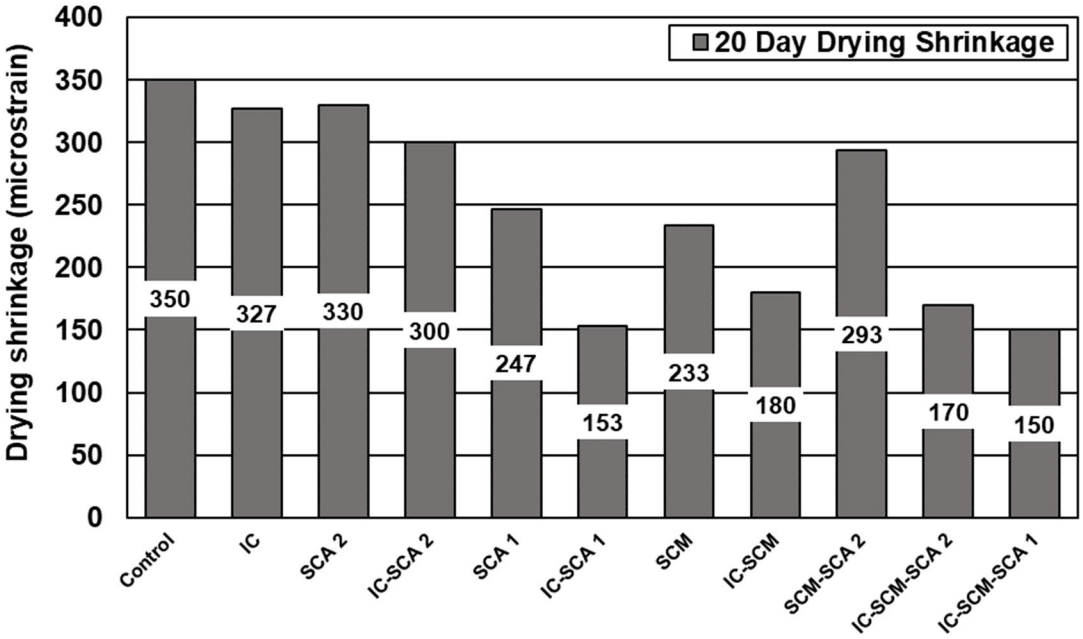


Figure 2.8—Shrinkage after 20 days of drying.

Figure 2.8 shows that the use of IC always resulted in reduction of drying shrinkage compared to similar mixtures without IC. This is likely due to the release of water from pre-wetted LWAs to the surrounding paste over time, which counteracts the loss of water due to drying. The reduction of drying shrinkage observed when cement is partially replaced by SCMs (SCM mixture) may be due to the fact that the hydration of slag cement starts at a later stage (De La Varga et al. 2012). Therefore, the SCM mixtures show lower early age drying than the mixture containing 100% portland cement (Control mixture).

2.4.2.2 20 to 180 Days of drying

Figure 2.9 shows the shrinkage that took place between 20 and 180 days of drying. The results show that in every pair of mixtures with and without IC, the mixture with IC experienced greater 20 to 180-day shrinkage. This observation was opposite to the one observed in 0 to 20-day drying shrinkage period.

SCA 1, likely due to the incorporation of an SRA, helped to reduce the 20 to 180-day drying shrinkage; the SCA 1 mixture exhibited the lowest 20 to 180-day drying shrinkage among all mixtures. In addition, among mixtures containing IC and SCMs (IC-SCM, IC-SCM-SCA 1, and IC-SCM-SCA 2), the mixture containing SCA 1 (IC-SCM-SCA 1) had lower 20 to 180-day drying shrinkage by at least 70 microstrain. On the other hand, SCA 2 was not effective in reducing the 20 to 180-day drying shrinkage; in every comparable pair, incorporation of SCA 2 always resulted in either a similar or greater shrinkage (Control vs. SCA 2, IC vs. IC-SCA 2, and IC-SCM vs. IC-SCM-SCA 2).

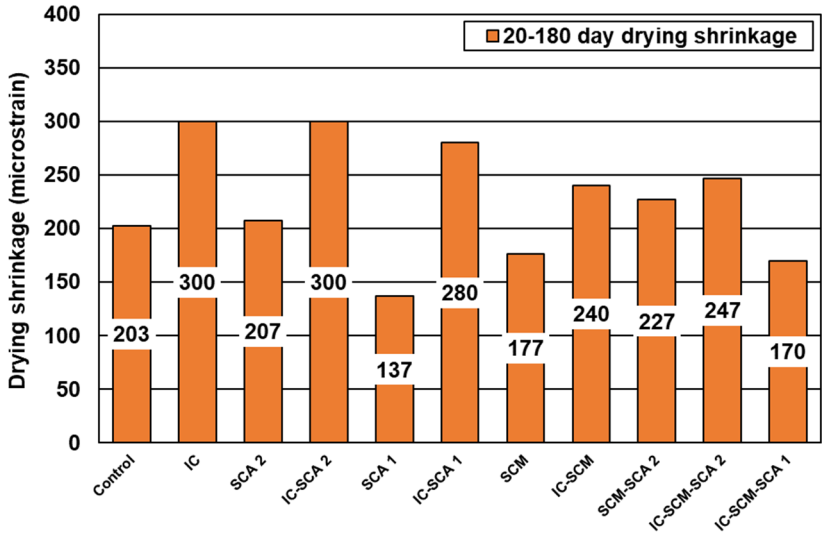


Figure 2.9—Shrinkage between 20 and 180 days of drying.

2.5 SUMMARY AND CONCLUSIONS

A modified version of ASTM C157, in which length-change measurements begin 5½ ± ½ hr after casting concrete, was used to evaluate the effects of internal curing (IC) obtained through the use of intermediate size pre-wetted lightweight aggregate, partial replacements of cement with the supplementary cementitious materials slag cement and silica fume (SCMs), incorporation of calcium oxide-based (CaO-based) and magnesium oxide-based (MgO-based) shrinkage-compensating admixtures (SCAs), and combinations of IC, SCMs, and SCAs on the shrinkage of eleven concrete mixtures with a moderate *w/cm* ratio (0.45).

The following conclusions are based on the results and analysis presented in this chapter:

1. The modified ASTM C157 method developed in this study helps to capture the early-age behavior of concrete mixtures.
2. IC provided by partial replacement of total aggregate with intermediate-size pre-wetted LWA is effective in reducing 0 to 20-day drying shrinkage in concrete made with moderate *w/cm* ratios. In every pair of mixtures with and without IC, the mixture with IC exhibited greater early-age expansion and less shrinkage during the first 20 days of drying. The opposite was observed in later-age drying shrinkage, 20 to 180-day, where mixtures containing IC experienced greater shrinkage than their comparable pairs with no IC.
3. Partial replacements of portland cement with slag cement and silica fume increase first-day expansion and reduced shrinkage. A further increase in first-day expansion and a reduction in shrinkage is obtained when internal curing is used in conjunction with slag cement and silica fume.
4. The incorporation of the CaO- or the MgO-based SCAs evaluated in this study results

is a reduction in the tendency to develop shrinkage strain. This is achieved because both the SCAs induce an expansion during the curing period. The CaO-based SCA induces a more rapid expansion of greater magnitude, with most of the expansion taking place within the first day after casting while the MgO-based SCA causes an expansion that steadily increases throughout the curing period and is lower in magnitude. The MgO-based SCA also contributes to lower drying shrinkage because it incorporates a shrinkage-reducing admixture.

5. When the CaO-based SCA is incorporated in a mixture containing SCMs or SCMs and IC, a larger expansion and, consequently, a lower overall shrinkage strain is observed. The increase in expansion is greater than observed for CaO-based SCA alone or the CaO-based SCA used in conjunction with pre-wetted LWA. The same observations cannot be made for mixtures incorporating SCMs with the MgO-based SCA.

CHAPTER 3 - PRINCIPAL FACTORS AFFECTING CRACKING IN CONCRETE BRIDGE DECKS

3.1 GENERAL

The cracking of concrete bridge decks is a nation-wide problem. In this chapter, the results of over one hundred cracking surveys conducted over a period of three decades on 40 monolithic composite concrete bridge deck placements supported by steel girders are used to evaluate the factors contributing to bridge deck cracking and determine the factors that have the greatest influence. The parameters considered are paste content (volume of cement and water), slump, compressive strength, and air content as material properties and temperature information and time of the placement during the day as environmental factors.

Statistical analysis is employed to evaluate the factors affecting cracking by considering the correlations between the factors. Results show that cracking increases substantially when the paste volume exceeds a threshold value of 27.2%, regardless of other factors. Increased strength and slump have small but measurable effects on increased cracking, regardless of other factors. In addition, cracking is greatly increased when concrete is placed on days with greater variations in temperature. Bridge decks placed and finished between midnight and noon exhibit lower cracking than those placed and finished between early morning and late night.

3.2 INTRODUCTION

One of the main problems affecting the performance of highway bridges is the deterioration of concrete decks. This can be a result of concrete distress caused by freeze-thaw damage, the alkali-aggregate reaction, cracking, or corrosion of reinforcing bars. According to the American Society of Civil Engineers (ASCE 2017), about 10% of U.S. bridges are considered structurally deficient, requiring repair or replacement. As the corrosion of steel reinforcement has been found to be a major factor affecting the durability of bridge decks (McKeel 1985, Prefetti 1985, Virmani and Clemeña 1998, Russell 2004), several techniques have been used to reduce chloride penetration to the level of reinforcement, including increasing the clear cover to reinforcement, using epoxy-coated reinforcing bars, and using concrete with lower permeability (Russell 2004). By measuring the chloride contents at the level of reinforcement in cracked and uncracked concrete sections of bridge decks, however, Lindquist et al. (2005) showed that regardless of other factors, susceptibility to corrosion significantly increases at crack locations compared to regions with uncracked concrete. Furthermore, recent research has demonstrated that even bars with epoxy coating are not completely protected against disbondment and corrosion in cracked concrete (Darwin et al. 2011). Corrosion of reinforcing steel, which is greatly accelerated by the presence of cracks, is costly as the estimated cost for the nation's backlog of bridge rehabilitation was $123 billion in 2017 (ASCE 2017).

Researchers at the University of Kansas (KU) initiated studies in 1991 to address bridge deck cracking. These efforts involved three studies to evaluate cracking, two of which also involved chloride ingress. chloride ingress of the 59 bridges included in the studies, 13 had monolithic conventional concrete decks with 100% portland cement (CONV), 16 had conventional concrete overlay decks, 11 had 7% silica fume overlay decks, and 19 had 5% silica fume overlay

decks (Schmitt and Darwin 1995 and 1999, Miller and Darwin 2000, Darwin et al. 2004, Lindquist et al. 2005 and 2006). The crack survey results suggest that cracking in bridge decks is caused mainly by concrete shrinkage. It was concluded that cracking increased with age, higher cement content, water content, and thus volume of paste (cement + water), concrete slump, and concrete compressive strength. Among the CONV bridges, those constructed in 1980s experienced less cracking than those constructed in 1990s. For the silica fume overlay decks, cracking exceeded that observed in the older CONV decks, but those constructed in 1997 and 1998 did show less cracking than those constructed in 1990 and 1991. The improved performance was in all likelihood due to improved efforts to reduce evaporation before the start of curing. For the CONV decks, it was found that cracking increased with increasing maximum air temperature and temperature range on the day of deck placement. The chloride testing indicated that, regardless of deck type, samples taken from older bridge decks had greater chloride contents than those from younger decks, and samples taken at cracks had much higher chloride contents than those taken in uncracked concrete.

These findings, combined with laboratory work performed at KU during that period and in consultation with the Kansas Department of Transportation (KDOT), led to the development of specifications for low-cracking high-performance concrete (LC-HPC) bridge decks (Lindquist et al. 2008). The LC-HPC specifications include requirements for material and mixture properties (aggregate and concrete specifications) and construction practices (construction specifications) to construct bridge decks with minimal cracking. The specifications limit the cement content to between 500 and 540 lb/yd^3 and the water-to-cement ratio (*w/c*) to be 0.44 or 0.45 to make sure that the paste volume is less than 25%. Concrete slump is limited to 1½ to 3½ in., compressive strength to 3500 to 5500 psi, and air content to 6.5 to 9.5%. The LC-HPC specifications were used

to initiate what became a two-phase, 13-year Pooled-Fund study with participation by 19 state Departments of Transportation (DOTs), the Federal Highway Administration, and industry. To date, 19 bridge deck placements in Kansas and six in other states have been built employing the LC-HPC specifications and have demonstrated improved cracking performance compared to control decks built following standard DOT specifications (Lindquist et al. 2008, McLeod et al. 2009, Yuan et al. 2011, Pendergrass and Darwin 2014, Alhmood et al. 2015, Darwin et al. 2010, 2016 and 2017). Appendix A includes the latest LC-HPC specifications.

In addition to the KU studies, a number of small-scale experimental studies have been conducted in recent years to evaluate the cracking behavior of bridge decks, mainly showing how different technologies help to reduce cracking. Most of these studies have focused on a single parameter. Richardson et al. (2013) demonstrated the effectiveness of Type K cement in reducing cracking by comparing cracking on two 7 × 10 ft reinforced concrete simulated bridge decks, one with concrete containing Type K cement and the other with concrete containing 100% portland cement, where the deck with Type K cement experienced lower tensile strains than the deck constructed with 100% portland cement. Khayat and Mehdipour (2016) compared the effectiveness of internal curing, shrinkage-compensating admixtures (SCAs), and supplementary cementitious materials (SCMs) in reducing cracking using 6 × 6 ft reinforced concrete slabs placed in wooden forms. They found that SCAs and a combination of SCMs and IC were effective in reducing the rate and magnitude of shrinkage.

Bitnoff (2014) and Hopper et al. (2015) evaluated the cracking performance of bridge decks based on field surveys. In these studies, either the sample size of the bridges was small or only one survey was conducted on the bridges having different ages. For four bridges with partial-depth decks panels with cast-in-place concrete toppings, Bitnoff (2014) observed that most of the

cracks appeared along the panel joints. Based on a single survey of 40 bridge decks with ages ranging from 0 to 90 years, Hopper et al. (2015) found that decks with higher strengths and higher cementitious materials contents exhibited higher cracking and decks supported by prestressed girders showed less cracking than those supported by steel girders.

In the previous analyses of the factors affecting cracking of the CONV bridge decks by Schmitt and Darwin (1995, 1999), Miller and Darwin (2000), and Lindquist et al. (2005, 2006), correlations between the factors were not considered, which affected their conclusions. Lessons learned by KU researchers after analyzing the most recent surveys on the LC-HPC bridges (Darwin et al. 2016, and 2017 and 2018 survey results described in this study), observations made during the construction of multiple bridge decks in Kansas between 2014 and 2017, and in-depth evaluation of the archived notes, pictures, and videos taken during the construction of the LC-HPC bridge decks (between 2006 and 2011) have been used to identify the construction practices and extreme traffic loads that can significantly affect cracking of bridge decks, which are discussed in Chapter 5. The effects of some of the construction practices and the extreme traffic loads on cracking were either not known to Lindquist et al. (2008), Yuan et al. (2011), and Pendergrass and Darwin (2014), who analyzed the cracking behavior of CONV (up to 78 months) and LC-HPC bridge decks (up to 42 months). As a result, cracking data for all LC-HPC bridge decks were included in the database, whether or not the decks were subjected those construction practices and extreme loads, affecting their conclusions. In this study, analyses to establish the principal causes of cracking are based on 96-month cracking data from the CONV bridges (previously analyzed only up to 78 months) and 96-month cracking data from the LC-HPC bridges (provided by surveys conducted between 2014 and 2018) by considering the correlation between the factors and excluding the bridge decks that involved either poor construction practices or extreme traffic loads.

3.3 DATA COLLECTION METHOD

Crack surveys can be used to evaluate the cracking performance of concrete bridge decks. The method used in this study was developed by Schmitt and Darwin (1995). The surveys are performed on days that are mostly sunny with an air temperature of at least 60° F. After traffic control is provided, typically one lane at a time, a 5 × 5 ft grid is drawn on the deck. Surface cracks are marked using lumber crayons or chalk, and a scaled plan (crack map) that also has the grid is used to record the cracks. A detailed description of the crack survey procedures is provided by Darwin et al. (2016). This method has been used by KU and others to evaluate the cracking performance of bridge decks constructed with and without a variety of crack-reducing technologies (Bitnoff 2014, Hopper et al. 2015, Polley et al. 2015, Cavalline et al. 2017). The crack map is scanned and converted into an AutoCAD file, and the crack lengths are measured using built-in AutoCAD commands. The crack density for each bridge is calculated by dividing the crack length by the deck area. Prior to 2014, a Fortran-based program was used at KU to calculate the crack densities using the scanned scaled maps. A sample crack map is shown in Figure 3.1. Appendix B includes the latest crack survey specifications.

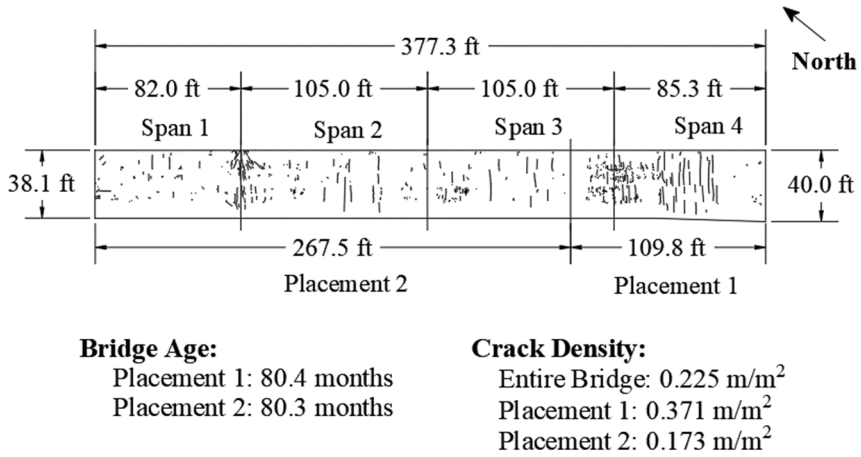


Figure 3.1–A sample crack map of an LC-HPC bridge surveyed in 2014

3.4 BRIDGE DECKS

The bridge decks included in this study are described next.

3.4.1 CONV Bridge Deck Placements

The bridge decks identified in this study as CONV were constructed between 1984 and 1995 in Kansas. Survey results for 35 individual placements are available, but nine placements are not used for analysis because some or all of the required material properties are not available or the deck was only surveyed once. The 26 placements used for analysis were surveyed by both Schmitt and Darwin (1995, 1999) and Lindquist et al. (2005, 2006). Miller and Darwin (2000) surveyed three of the placements. The concrete used for the 26 placements have cement contents between 602 and 658 lb/yd^3, water contents between 241 and 281 lb/yd^3, paste volumes (water + cement) between 25.65 and 28.78% of the concrete volume, water-to-cement ratios (*w/c*) between 0.40 and 0.44, measured slumps between 1½ to 3 in., measured air contents between 4.5 and 6.5 %, and measured compressive strengths between 4200 and 7430 psi. The decks were constructed following Kansas DOT specifications. Table 3.1 lists the material properties, along with the air

temperature range and high air temperature during the day of placement for the CONV bridge decks.

Table 3.1–Properties of the CONV bridge deck placements

Bridge Placements	Paste content (%)	Compressive Strength (psi)	Temp Range (°F)	High Air Temp (°F)	Air Content (%)	Slump (in.)
3-046 Ctr. Deck	25.7	5630	24	50	6	1.5
3-045 E. Ctr. Deck	26.4	6270	31	61	6	1.75
70-095 Deck	27.2	5510	18	57	5.93	1.75
70-104 Deck	27.2	4170	20	73	5	1.75
70-103 Left	27.2	4750	31	70	5.4	1.75
70-103 Right	27.2	5110	31	61	5.85	1.88
99-076 p* 2	27.9	7400	38	82	5	1.96
56-142 Positive Moment	26.5	4760	34	78	6.1	2
3-045 West Deck	26.4	4790	18	46	5	2
3-045 W. Ctr. Deck	26.4	5640	13	62	5	2
3-046 West Deck	26.4	5260	15	43	6	2
70-107 Deck	27.2	6820	21	57	5.4	2.15
56-142 Negative Moment	26.5	5130	24	65	6	2.25
89-208 Deck	27.1	7430	21	89	5	2.25
3-045 East Deck	26.4	6190	16	49	4.5	2.25
3-045 Ctr. Deck	26.4	6140	19	54	5.5	2.25
3-046 East Deck	26.4	5760	16	52	6	2.25
99-076 p3	27.9	6700	40	88	5.25	2.25
99-076 p5	28.7	6250	25	53	4.75	2.25
99-076 North (East Ln.)	28.7	5750	18	60	6	2.25
99-076 p4	28.7	6100	34	62	5.75	2.38
75-045 Deck	27.9	5640	22	88	5.76	2.41
99-076 North (West Ln.)	28.7	5380	18	55	5.5	2.5
75-044 Deck	27.9	6430	4	66	5.63	2.54
56-148 Deck	27.2	6170	23	97	6.5	2.58
89-204 Deck	28.8	6370	21	77	5.2	3

*p=placement

3.4.2 LC-HPC and Extra Control Bridge Deck Placements

As part of a 13-year Pooled-Fund study at KU on the "*Construction of Crack-Free Bridge Decks*," 17 bridge decks, cast in 22 placements, following the LC-HPC specifications were planned for construction in Kansas. The specifications were not followed on three placements; therefore, those three placements are not considered to represent LC-HPC bridge decks. The 19 bridge deck placements that were constructed following the LC-HPC specifications, many with a comparable control (sister) deck that was constructed following the existing Kansas DOT specifications, were built between 2005 and 2011. Of the 19 LC-HPC placements, 17 are supported by steel girders, the focus of this study, and two (LC-HPC-8 and 10) are supported by precast prestressed concrete girders. Thirteen out of the 17 placements are used in this study. Placement 1 of LC-HPC-4, the south lane of LC-HPC-11, placements 1 and 2 of LC-HPC-12, and LC-HPC-13 are excluded because they were subject to special conditions, either during construction or during service, which resulted in excessively high cracking. As explained by Pendergrass and Darwin (2014) and Darwin et al. (2016), the first placement of LC-HPC-4 had mixture proportioning issues during construction resulting in production of concrete not meeting LC-HPC specifications; the south lane of LC-HPC-11 has been subjected to excessively heavy truck loads throughout its service life; the two placements of LC-HPC-12 were subjected to high flexural and torsional loads during construction; and the concrete on LC-HPC-13 was not consolidated properly.

The 13 placements used in the analysis are designated LC-HPC-1 through 7, 9, 11, 15, 16, and 17. Data for the 13 placements, listed in Table 3.2, include crack densities, material properties, temperature range and high air temperature on the day of construction, and time of placement during the day. Detailed crack surveys were performed on the LC-HPC and associated control bridge decks between 2004 and 2018, the results of which up to 2016 are documented through a

series of reports and papers. A list of these documents can be found in the bibliography of the final report on the study (Darwin et al. 2016). The 13 LC-HPC placements included in this study have cement contents between 500 and 540 lb/yd^3, water contents between 225 and 243 lb/yd^3, volume of paste (water + cement) between 22.8 and 24.6%, *w/c* ratios of 0.42, 0.44 or 0.45, measured slumps between 3 and 4 in, measured air contents between 6.4 and 9.5%, and measured compressive strengths between 3800 and 6400 psi.

Table 3.2 also includes data for a bridge designated as the "Extra Control," a monolithic concrete deck supported by steel girders that is a non-LC-HPC bridge constructed in 2005 in Kansas and surveyed eight times by the KU researchers.

Table 3.2–Properties of the LC-HPC and Extra Control bridge deck placements

Bridge Placements	Paste Content (%)	Compressive Strength (psi)	Air Temp Range (°F)	High Air Temp (°F)	Air Content (%)	Slump (in.)	Construction Time
LC-HPC 1-p*1	24.6	5210	33	84	7.9	3.75	6:00 am-9:30 am
LC-HPC 1-p2	24.6	4980	35	78	7.8	3.25	7:30 am-10:30 am
LC-HPC 2	24.6	4600	25	78	7.7	3	6:00 am-9-:30 am
LC-HPC 3	24.4	5990	26	65	8.7	3.25	2:00 am-6:30 am
LC-HPC 4-p2	23.4	4790	20	79	8.8	3	1:30 am-6:00 am
LC-HPC 5	23.9	6380	15	56	8.7	3	2:00 am-10:00 am
LC-HPC 6	24.4	5840	26	60	9.5	4	5:30 am-12:30 pm
LC-HPC 7	24.6	3790	25	88	8	3.75	2:00 am-8:30 am
LC-HPC 9	24.2	4190	25	69	6.7	3.5	9:30 am-6:30 pm
LC-HPC 11**	23.4	4680	39	87	7.7	3	6:00 am-11:10 am
LC-HPC 15	22.8	4440	25	70	9	3.25	7:15 am-8:40 pm
LC-HPC 16	22.8	5040	20	57	6.4	3.75	11:00 am-9:30 pm
LC-HPC 17	24.6	5160	35	86	7	3.25	7:00 am-9:20 pm
Extra Control	25.7	5510	22	78	5.9	3	Not Available

*p=placement.
**=only north lane is included.

3.5 CALCULATION OF CRACK DENSITIES

Crack densities for the CONV bridge decks (Schmitt and Darwin 1995, 1999, Lindquist et al. 2005, 2006) were initially calculated using a Fortran-based program. A procedure using AutoCAD software, as briefly explained in "Data Collection Method," is developed and used to calculate crack densities for the LC-HPC bridge decks. To be consistent, the crack maps obtained by Schmitt and Darwin (1995, 1999), and Lindquist et al. (2005, 2006) are re-scanned and converted to AutoCAD files. The AutoCAD is used to recalculate the crack densities for use in the current analysis.

The individual crack survey results for the CONV, LC-HPC, and Extra Control bridge decks are listed in Tables D.1 and D.2 in Appendix D. Figure 3.2 shows crack density as a function of bridge age for the 26 CONV bridge deck placements used in the analysis, with each point representing the result of an individual crack survey. As shown in the figure, crack density increases with age for 24 out of 26 placements. Figure 3.3 compares crack density with age for the 13 LC-HPC and Extra Control bridge decks. In this case, cracking increases with age for all placements, except for LC-HPC-1 P1, LC-HPC-1 P2, and LC-HPC-7, the three represented by red lines at the bottom of the figure, for which the increases in cracking with increasing age are negligible.

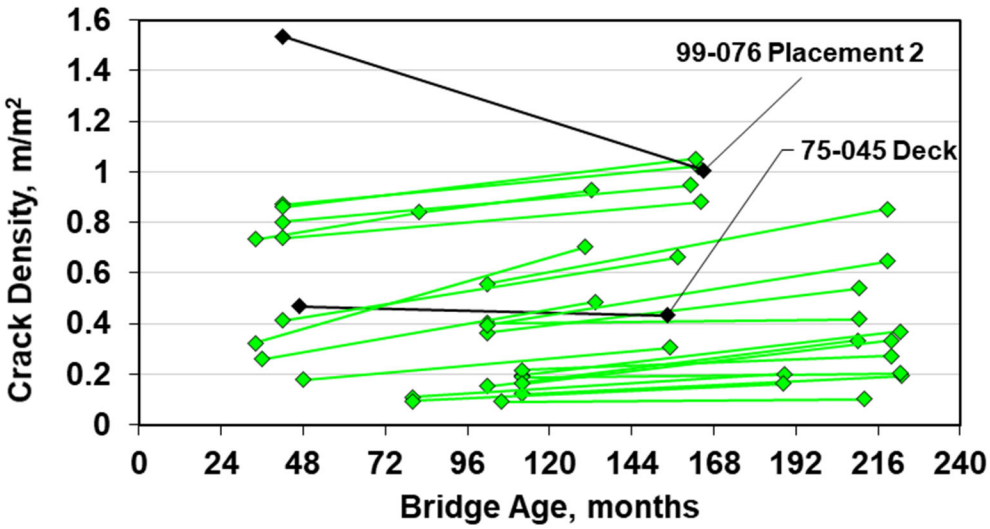


Figure 3.2–Crack density versus age for the CONV deck placements

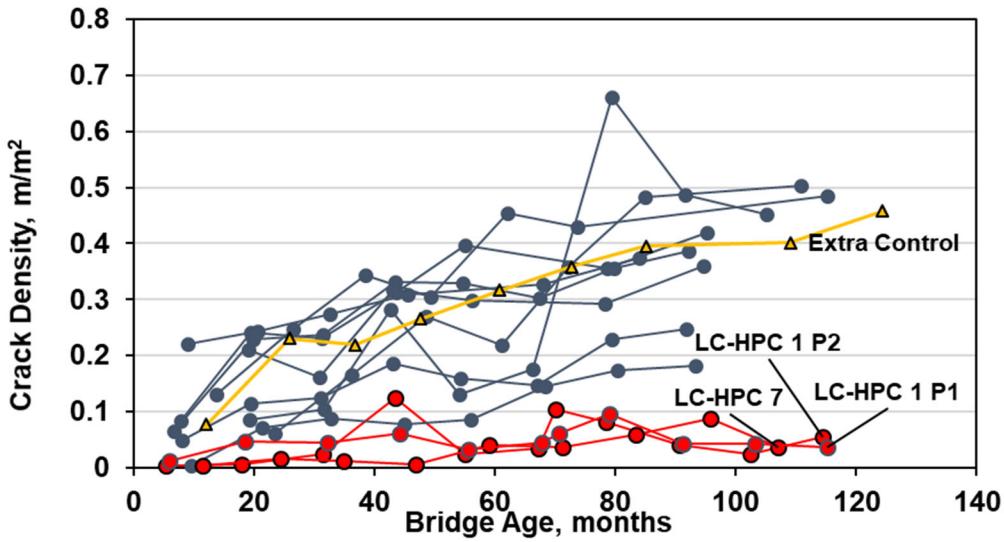


Figure 3.3–Crack density versus age for the LC-HPC and Extra Control bridge deck placements

Because cracking appears to be a function of age, a fair comparison can be made among the performance of the bridge decks based on their crack densities only if the crack densities are compared at the same age. Due to limited long-term survey data for the LC-HPC decks, Yuan et al. (2011) and Pendergrass et al. (2014) evaluated the cracking performance of the CONV and LC-

HPC decks at 36 and 42 months, respectively. In this study, 96 months is chosen because all of the CONV bridge decks had survey data at or beyond 96 months, and the latest surveys of the LC-HPC decks, reported by Alhmood et al. (2015) and Darwin et al. (2016), and surveys conducted in 2017 and 2018 provide data adequate to evaluate the cracking performance up to 96 months. Data taken from a survey at an age of 96 ± 6 months is used to establish the crack density at 96 months. If a crack density was not available at 96 ± 6 months, linear interpolation between the crack densities before and after 96 months is used to estimate the 96-month crack density. Although it would have been possible to use interpolation to find the 96-month crack density for the second placement of 99-076 bridge, for which crack density decreases significantly with age as shown in Figure 3.2, the 96-month crack density is assumed to be equal to the latest available crack density at 165 months (1.0 m/m^2). This is done since the only available prior crack density for 99-076 Placement 2 (1.54 m/m^2 at 48 months) is much greater than similar bridges in this study, causing the interpolated 96-month crack density value to be too high.

The crack densities at 96 months for six CONV placements, the five placements of bridge 3-045 and bridge 70-104, and one LC-HPC placement, LC-HPC-17, are extrapolated using the data obtained in the final two surveys, at ages of 112 and 223 months for five placements of 3-045, 106 and 212 months for 70-104, 68 and 84 months for LC-HPC-17. Data for these seven placements are shown in Figure 3.4. The 96-month crack densities used in the analysis for the 40 CONV, LC-HPC, and Extra Control bridge deck placements are listed in Table 3.3.

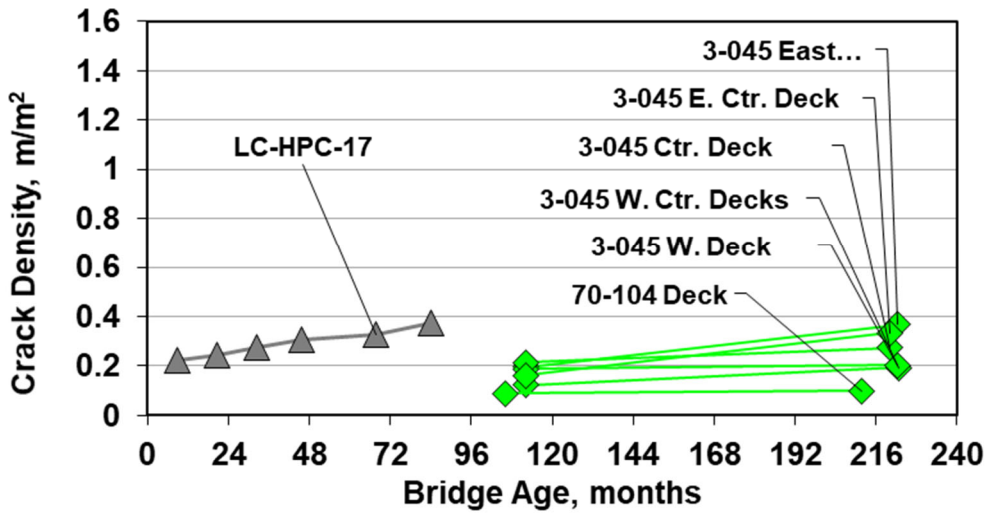


Figure 3.4–Crack density versus age for the CONV and LC-HPC bridge deck placements where the 96-months crack densities were obtained by extrapolating two nearest data points

Table 3.3–96-month crack densities of CONV, LC-HPC, and Extra Control bridge placements

Bridge Placements	96-month Crack Density (m/m^2)	Bridge Placements	96-month Crack Density (m/m^2)
3-046 Ctr. Deck	0.153	**3-045 West Deck**	0.112
3-045 E. Ctr. Deck	0.138	**3-045 W. Ctr. Deck**	0.186
70-095 Deck	0.063	**3-046 West Deck**	0.362
70-104 Deck	0.081	**70-107 Deck**	0.567
70-103 Left	0.557	**56-142 N. Pier**	0.106
70-103 Right	0.395	**56-142 + Moment**	0.122
99-076 p2	1	**LC-HPC 1-p1**	0.043
89-208 Deck	0.169	**LC-HPC 1-p2**	0.040
3-045 East Deck	0.171	**LC-HPC 2**	0.116
3-045 Ctr. Deck	0.206	**LC-HPC 3**	0.487
3-046 East Deck	0.402	**LC-HPC 4-p2**	0.181
99-076 p3	0.802	**LC-HPC 5**	0.247
99-076 p5	0.946	**LC-HPC 6**	0.386
99-076 North (East Ln.)	0.530	**LC-HPC 7**	0.087
99-076 p4	0.939	**LC-HPC 9**	0.460
75-045 Deck	0.452	**LC-HPC 11 (North Lane)**	0.492
99-076 North (West Ln.)	0.867	**LC-HPC 15**	0.360
75-044 Deck	0.235	**LC-HPC 16**	0.420
56-148 Deck	0.399	**LC-HPC 17**	0.409
89-204 Deck	0.854	**Extra Control**	0.398

3.6 ANALYSIS OF FACTORS AFFECTING CRACKING

To determine the factors that have the greatest influence on bridge deck cracking, the 96-month crack densities for the CONV, LC-HPC, and Extra Control bridge deck placements (total of 40 placements) are evaluated as a function of concrete mixture proportions and material properties, specifically, paste content (volume of the cement and water), slump, concrete compressive strength, and air content; plus environmental influences, including, range of the air temperature and high air temperature on the day of deck placement, and for LC-HPC bridge decks, time of placement (information not available for the CONV, and Extra Control decks).

3.6.1 Regression Analysis

In the following sections, the principal factors contributing to bridge deck cracking are discussed, and the relative impact of each is reported based the results of a linear regression analysis, which accounts for the effects of the multiple variables on cracking. A regression model quantifies the relationship between the independent variables (in this case, material properties and environmental conditions) and the dependent variable (96-month crack density). Accurate selection of the independent variables is of utmost importance, and in doing so, the correlations between variables need to be considered. A *stepwise regression by forward addition* (Efroymson 1960, Hines et al. 2003, Hocking 2013) can be used to select the independent variables to avoid inclusion of highly correlated variables, as well as those that do not significantly affect the dependent variable. This method involves adding one variable to the model at each step and retaining that variable only if its inclusion results in a meaningful contribution to the model. The criterion by which the contribution of the added variable to the model is evaluated is the probability value or *p*-value of the *t*-statistic test of the slope coefficient (b) of that variable. The *p-value* quantifies the test of the hypothesis H_0: b=0 (that the slope coefficient is zero and, therefore, the

variable does not contribute to the model) versus H_A: $b \neq 0$ (that the coefficient is not zero and, therefore, the variable contributes to the model). If the *p*-value is smaller than a threshold value (0.05 used as a universally accepted limit), that variable is retained (Hines et al. 2003, Hocking 2013) in the model. The *Adjusted* R^2 value is also evaluated at each step; unlike the standard R^2, which only increases with the addition of any independent variable to an existing model, the *Adjusted* R^2 only increases when the added variable does not contribute to the model due to chance. The *Adjusted* R^2 is a modified version of R^2, which is always less than or equal to R^2 and accounts for the number of variables in a regression model. Therefore, a variable is retained in the model if it has a *p-value* not greater than 0.05 and its addition to the model increases the *Adjusted* R^2. Forward-addition continues until no variable can be justifiably added to the model based on these two criteria, resulting in the "*final model*."

Although high air temperature on the day of casting, as an environmental influence, is available for all 40 bridges, it is not included in the regression analysis. The reason is that specific construction procedures were controlled and enforced for the LC-HPC bridge decks, that is controlled concrete temperature, limited surface finishing, and early application of 14-day wet-curing, which can affect the temperature-related stresses developed in the concrete decks, but were not controlled for CONV decks. Thus the high air temperature on the day of construction likely affected LC-HPC decks differently than CONV decks. The effect of high air temperature on cracking, however, is discussed separately and in detail for CONV and LC-HPC bridges. Also, because time of placement was only available for the LC-HPC decks, the linear regression analysis does not include this parameter.

3.6.1.1 Initial analysis

The parameters considered for the *initial regression* model are paste content (22.8 to 28.8 %), slump (1½ to 4 in.), compressive strength (3800 to 7400 psi), air content (4.5 to 9.5%), and air temperature range on the day of casting (4° to 40°F). After the regression analysis was performed, three out of five variables remained in the final model, each with a *p-value* of less than 0.05 and resulting in an increase in the value of the *Adjusted* R^2.

In the first step to develop the initial regression model, regression analyses were performed with the 96-month crack density as the dependent variable and paste content, slump, compressive strength, air content, and air temperature range individually as candidate independent variables. The variable producing the smallest value of p, 0.0027, was cement paste, which resulted in R^2 and *Adjusted* R^2 values of 0.21 and 0.19 for the model, respectively. Next, the regression analysis was performed between the 96-month crack density as dependent variable, paste content as an independent variable and one of the other variables (slump, air content, and air temperature range) as the candidate second independent variable with the variable having the lowest *p-value* added to the model. This variable was slump, with a p = 0.0084 and R^2 and *Adjusted* R^2 values of 0.35 and 0.31, respectively. In the third step, using the 96-month crack density as the dependent variable and paste content and slump as independent variables, air temperature range produced the lowest value of p = 0.0205 and values of R^2 and *Adjusted* R^2 of 0.44 and 0.39, respectively. In the following step, none of the remaining variables had p less than the defined threshold of 0.05. The lowest value of p, 0.127, was attained by compressive strength, which resulted in R^2 and *Adjusted* R^2 values of 0.48 and 0.42, respectively. Thus, the model included paste content, slump, and air temperature range as the *principle variables*. The principle variables included in the final model along with the slope coefficients, R^2, and *Adjusted* R^2 for the model are listed in Table 3.4.

Table 3.4- Principle variables found in the first regression analysis

Principle Variables	Slope Coefficients, β	*p*-values of the *t*-statistics of the Slope Coefficients	R^2/*Adjusted* R^2
Paste, %	0.128 ($m/m^2/\%$)	2.39816E-05	0.21/0.19*
Slump, in.	0.196 ($m/m^2/in.$)	0.007744	0.35/0.31**
Air Temperature Range, °F	0.011 ($m/m^2/°F$)	0.0205	0.44/0.39***

*for the model having paste as only independent variable**for the model having paste and slump as independent variables***for the model having paste, slump, and air temperature range as independent variables

The regression analyses are based on the assumption that there is a linear relationship between the dependent variable, in this case cracking and the independent variables. If the independent variables do not affect the dependent variable in a linear fashion, they will not be evaluated effectively. As discussed next, this is the case for the effect of paste content on cracking. As will be discussed, a modified regression analysis is, thus, required.

3.6.1.1.1 Paste content and cracking in bridge decks

Drying shrinkage of concrete is caused by the volume reduction of its paste constituent (cementitious materials and water). Deshpande, Darwin, and Browning (2007) evaluated the effects of paste volume (20, 30, and 40%) and *w/c* ratio (0.40, 0.45, and 0.50) on the free shrinkage of concrete and concluded that shrinkage was primarily a function of the paste volume and nearly independent of the *w/c* ratio. In fact, for a given paste volume, shrinkage decreased slightly as the *w/c* ratio increased – likely due to the larger pores in the concrete with the higher *w/c* ratio. The significant effect of paste on concrete shrinkage, regardless of the effects of the other factors, has also been observed by Ödman (1968) and Bissonnette et al. (1999). Symons and Fleming (1980) evaluated the relationship between 56-day free shrinkage and paste content on three series of concrete mixtures with paste contents between 25 and 40% and different binder compositions: one series with 100% portland cement, one with 20% fly ash replacement of cement, and one with 30% fly ash replacement of cement. As shown in Figure 3.5, they observed that 56-day free shrinkage increased almost linearly as the paste content increased for each of the binder compositions.

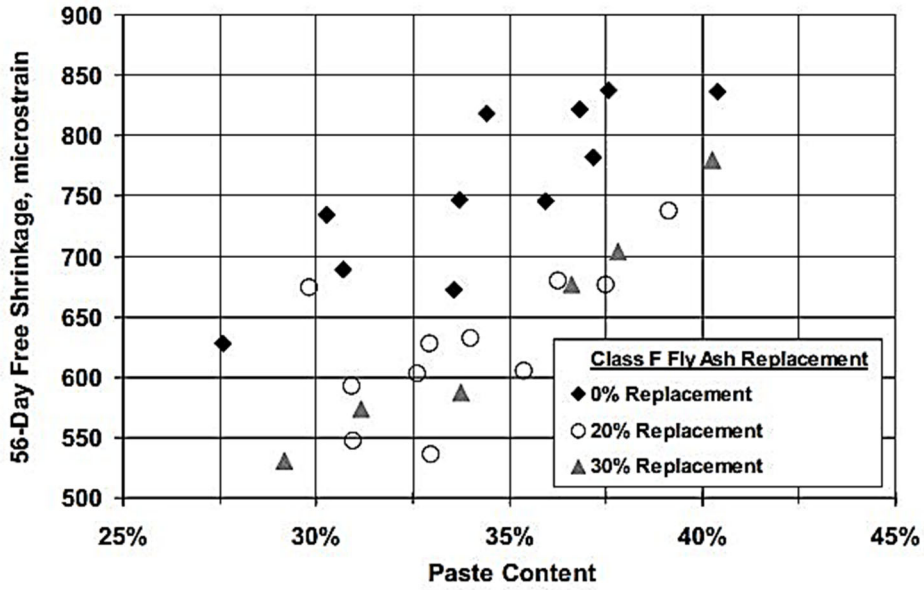


Figure 3.5–56-day free shrinkage versus paste content for a series of control mixtures (100% cement) and two series with various replacements of cement with Class F Fly Ash (Symons and Fleming 1980)

Radlińska and Weiss (2011) introduced a stochastic approach to account for the variations in concrete material properties and testing methods in predicting the probability of cracking. They showed that cracking potential in restrained concrete structures can be related to shrinkage, an effect that follows a nonlinear trend, unlike the relationship between free shrinkage and paste content, which appears to be linear, as shown in Figure 3.5. Radlińska and Weiss (2011) indicate that there is a threshold of shrinkage for a given degree of restraint, beyond which the probability of cracking significantly increases. Because of the complexity of the restraint in structures and the effects of other parameters on concrete deformation and cracking such as thermal effects, however, their simulations are not capable of establishing the threshold for a specific structure. One of the objectives of the current study is to establish a threshold for the paste content of concrete in bridge

decks supported by steel girders based on an analysis of the available crack survey results of the 40 bridge deck placements under evaluation.

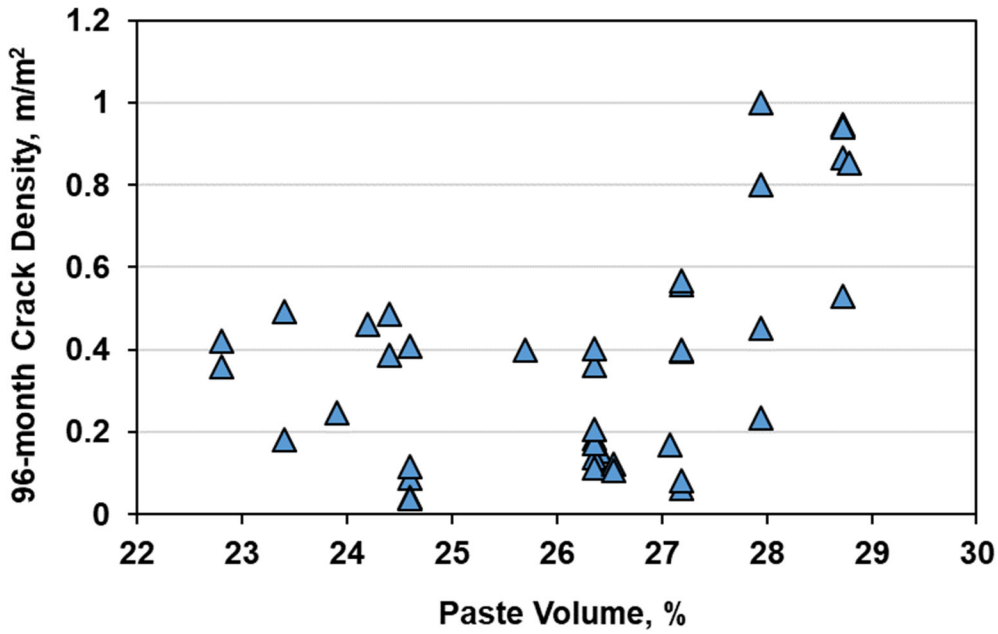


Figure 3.6–Paste content versus 96-month crack density for the CONV, LC-HPC, and Extra Control bridge deck placements

Figure 3.6 compares paste content (ranging from 22.8 to 28.8%) with 96-month crack density for the 40 bridge deck placements. The decks with paste contents between 22.8 and 26.4 appear not to be significantly affected by variations in paste content. There is a small increase in cracking when the paste content increases from 26.4 to 27.2%, but the tendency of the bridge decks to exhibit cracking greater than 0.6 m/m^2 increases substantially when the paste content exceeds 27.2%. This nonlinear relationship between paste content and 96-month crack densities supports the observations by Radlińska and Weiss et al. (2011) and shows that there is a range of paste content where cracking remains unaffected, beyond which the cracking abruptly increases.

The effect of paste content on cracking can be illustrated further by comparing the cracking performance of bridge decks other than the decks included in Figure 3.6. Figure 3.7 compares crack density with age for eight partial-depth precast deck panels with cast-in-place concrete toppings supported by precast prestressed concrete girders constructed in Kansas and Utah, four decks in each state. The Kansas decks, labeled as "KS DP" had concrete toppings with water-to-cementitious materials (*w/cm*) ratios of 0.42 or 0.45. The two decks with 0.42 *w/cm* ratio were constructed with a binary mixture with a 35% slag replacement of portland cement and a paste content of 24.77%. The two decks with 0.45 *w/cm* ratio were constructed with a ternary mixture with 35% slag and 5% silica fume replacements of portland cement and paste content of 23.99%. All four decks in Utah, labeled as "UT IC or UT Control" have a *w/cm* ratio of 0.44, a binary mixture with a 20% fly ash replacement of portland cement, and a paste content of 28%. Two of the Utah deck toppings (labeled as UT IC) are internally-cured by replacing a portion (16.7% by volume) of the aggregate with pre-wetted lightweight aggregate to minimize shrinkage and cracking and the other two Utah deck toppings (labeled as UT Control) have identical properties to UT IC decks but are not internally-cured. Detailed descriptions of the Kansas and Utah decks and results of field cracking surveys conducted are provided by Shrestha et al. (2013) and Bitnoff (2014), respectively.

Figure 3.7 shows that the Kansas deck panels, with paste contents of 23.99 or 24.77%, despite being 66 months older than the Utah decks at the time of their last surveys, exhibit significantly lower cracking than the Utah deck panels, with 28% paste. The results also show that the negative effect of significantly greater paste content on cracking cannot be offset by using the crack reduction technology of internal curing. These additional comparisons support the observations that paste content can significantly affect cracking.

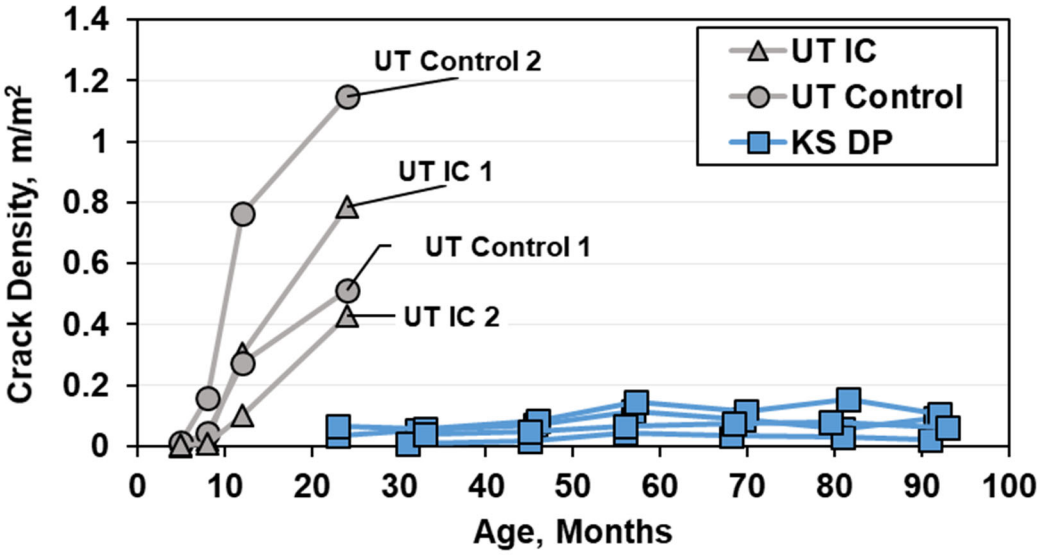


Figure 3.7– Crack density versus age for deck panels in Kansas and Utah

3.6.1.2 Second analysis

As discussed in the previous section, there appears to be a threshold of paste content above which cracking is significantly affected. As illustrated in Figure 3.6, the 96-month crack densities of the bridge decks with paste contents between 22.8 and 26.4% appear not to be greatly affected by the variations in paste content. Based on this observation, a value of 26.4% was assigned to bridges with paste contents less than 26.4% and a second regression analysis was performed. The parameters considered for the second regression model are the same as those used in the initial analysis but with the paste contents modified to the range of 26.4 to 28.8%. Following the second regression analysis, two out of the five variables remained in the final model. The final model includes paste content and air temperature range as the principle variables. The key difference is that once the insensitivity of cracking for paste contents below 26.4% was addressed within the model, slump no longer appeared to have a significant effect. The values of R^2 and *Adjusted* R^2 of this model are 0.60 and 0.58, significantly greater than the values of 0.44 and 0.39 obtained in the initial regression analysis. The principle variables included in the final model along with the slope

coefficients, R^2, and *Adjusted* R^2 for the new model are listed in Table 3.5. As shown in the table, the slope coefficient for paste content increased from 0.128 ($m/m^2/\%$) in the first regression analysis to 0.237 ($m/m^2/\%$) in the second.

Table 3.5–Principle variables found in the second regression analysis

Principle Variables	Slope Coefficients, β	*p*-values of the *t*-statistics of the Slope Coefficients	R^2/*Adjusted* R^2
Paste, %	0.237 ($m/m^2/\%$)	2.73E-08	0.53/0.52*
Air Temp Range, °F	0.0092 ($m/m^2/°F$)	0.014942251	0.60/0.58**

*for the model having paste as only independent variable**for the model having paste and air temperature range as independent variables

To evaluate how variables other than the dominant variables of paste content and air temperature range affect cracking, the effects of the two dominant variables must be removed from the analysis. This can be done by modifying the measured crack densities CD using the slope coefficients to equivalent values for a single paste content and a single air temperature range. For this analysis, the adjustments are made based on a paste content of 26.4% and an air temperature range of 24°F, values close to the average in each case. The calculation is shown in Eq. (1).

$$CD_{Adj} = CD + \Delta CD_{paste} + \Delta CD_{TempRange} \quad (1)$$

where CD_{Adj} = adjusted 96-month crack density,

CD = measured 96-month crack density,

$\Delta CD_{paste} = (\beta_{Paste})(26.4\%\text{-Paste})$,

$\Delta CD_{TempRange} = (\beta_{TempRange})(24°F\text{-AirTemp})$,

Paste = paste content, %,

Air Temp = air temperature range, °F,

β_{Paste} = slope coefficient for paste, 0.237 $m/m^2/\%$, and

$\beta_{Temp\ Range}$ = slope coefficient for air temperature range, 0.0092 $m/m^2/°F$.

When the effect on cracking of a principle variable (paste content or air temperature range) is evaluated, only the term for the other principal variable is included in Eq. (1). This way, the calculated adjusted crack density accounts for the effect of any other variable except for the one variable under evaluation. When the effect of a non-principal variable (strength, air content, or slump) is analyzed, Eq. (1) is used as shown.

Student's t-test is used when average crack densities are compared between two groups of bridge decks, to determine whether difference in the average crack density is due to random variations or represents a statistically significant difference in behavior. When sample sizes are small and the population standard deviation is unknown, the t-test can verify whether the difference in the means of two samples, X1 and X2, represents a difference in the population means, $\mu 1$ and $\mu 2$. There are several ways to describe the outcome of a t-test. In this chapter, the results are described based on a probability value or *p-value*, one that is different from the value of *p* used for determining the significance of the slope coefficients in the regression analysis. This *p-value* quantifies the test of the hypothesis H_0: $b=0$ (that the true mean difference between the two samples is zero) versus H_A: $b \neq 0$ (that the true mean difference between the two samples is not zero). Traditionally, *p-values* less than 0.02 or 0.05, and sometimes 0.10, are treated as indicative that the difference between two means is statistically significant rejecting the H_0: $b=0$ hypothesis (that is, unlikely to have arisen by chance). In this study, *p-values* less than 0.05 are considered to show a difference between two means that is statistically significant.

3.7 FACTORS AFFECTING CRACKING

3.7.1 Material Properties

The following section evaluates the effects of material properties on cracking.

3.7.1.1 Paste content

Figure 3.8 shows the average measured and adjusted 96-month crack densities in two paste volume categories. Decks in the 28.5% paste content category exhibit nearly three times the average crack density as the decks in 26.5% paste content category. Figure 3.8 also indicates that when the measured and adjusted crack densities are nearly the same. The fact that there is not a noticeable difference between the measured and adjusted average crack densities suggests that the paste content alone, regardless of the other parameters, has a dominant effect on cracking.

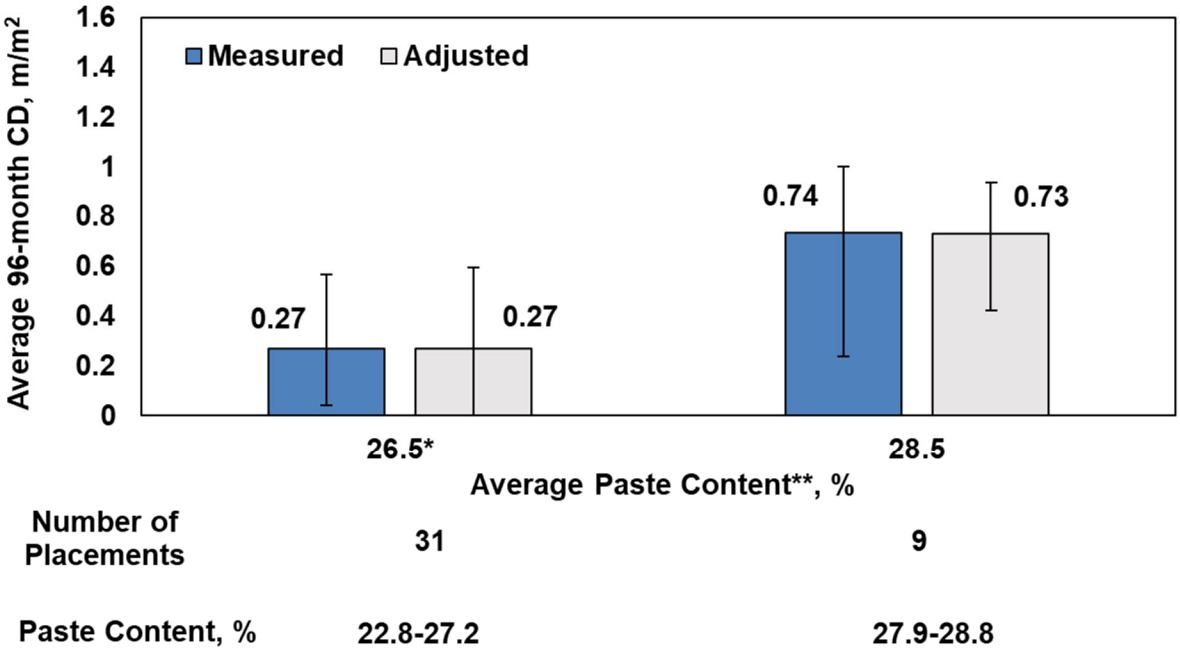


Figure 3.8–Average measured and adjusted 96-month crack densities for bridge decks with different paste contents. *: Average paste contents are based on modified paste contents; the actual average paste content is 25.50%. **: Average paste contents rounded to the nearest 0.5%.

3.7.1.2 Slump

Concrete with a greater slump can experience more cracking directly above the top reinforcing steel due to the settlement of fresh concrete. The average slumps range from 1½ to 4 in. for bridge decks evaluated in this study.

Figure 3.9 compares the average measured and adjusted crack densities with slump for the 40 bridge decks. The results indicate not a clear relationship between average slump and average measured crack densities due to the fact that these values are affected by variations in other parameters, in particular paste content that has a significant effect. When the average crack densities are adjusted using the Eq. (1), cracking increases from 0.21 to 0.25 m/m^2 when slump increases from 2 to 2 ¾ in. When slump increases from 2 ¾ to 3 ½ in. cracking increases from 0.25 to 0.26 m/m^2; these differences are small and not statistically significant. These results indicate that within the range of slump evaluated (1½ - 4 in.), representing *low slump* concrete in current practice, cracking is slightly affected by changes in slump. Further observations are needed to evaluate the effects of greater slump values on cracking. Lindquist et al. (2005 and 2006) who evaluated the cracking performance of CONV bridge decks up to 78 months concluded that slump has a small but measurable effect on cracking.

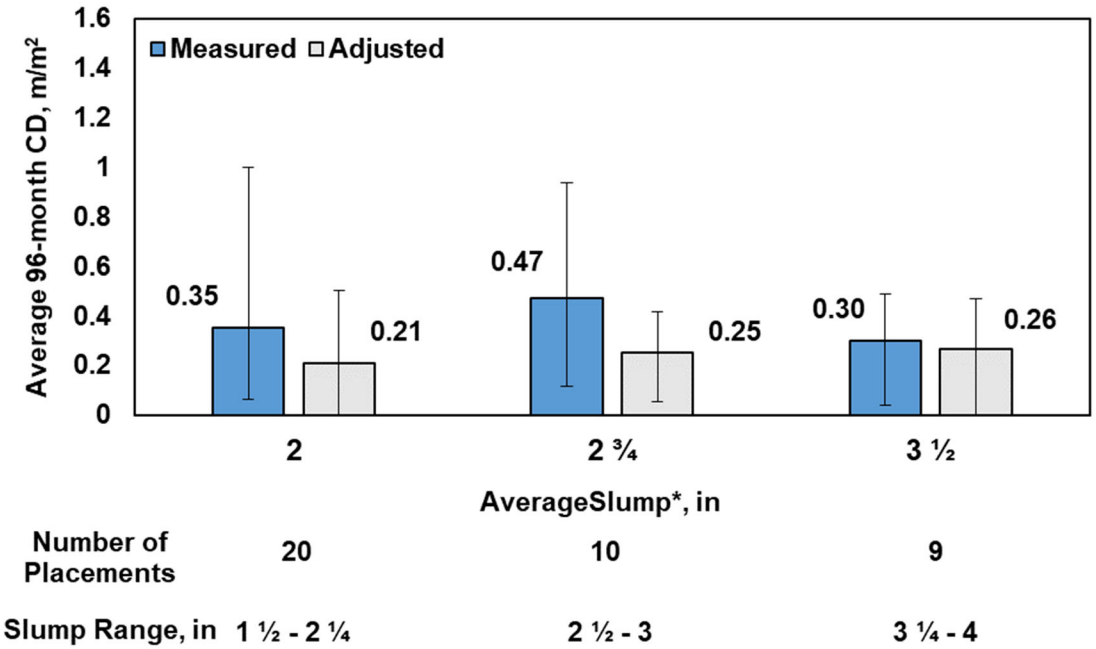


Figure 3.9– Average measured and adjusted 96-month crack densities for the bridge decks with different slump. *Average slump values are rounded to nearest ¼ in.

3.7.1.3 Strength

Concrete compressive strength for the 40 bridge decks evaluated in this study ranges from 3790 to 7430 psi. Figure 3.10 compares the average measured and adjusted crack densities for the bridge decks with average strength. As shown in Figure 3.10, average measured crack density increases with increasing strength, which is also true for the adjusted crack densities that are calculated using the Eq. (1). The bridge decks in strength category of 5500 psi exhibit 0.08 m/m^2 greater cracking than the bridge decks in strength category of 4500 psi, a difference that is not statistically significant. When strength increases from 5500 to 6500 psi, crack density only increases 0.01 m/m^2, a small difference that is also not statistically significant. The results of this section indicate that compressive strength has a small but measurable effect on cracking. This effect can be attributed to decreased creep in concretes with greater strengths; creep helps to relieve the tensile stresses and reduces the cracking.

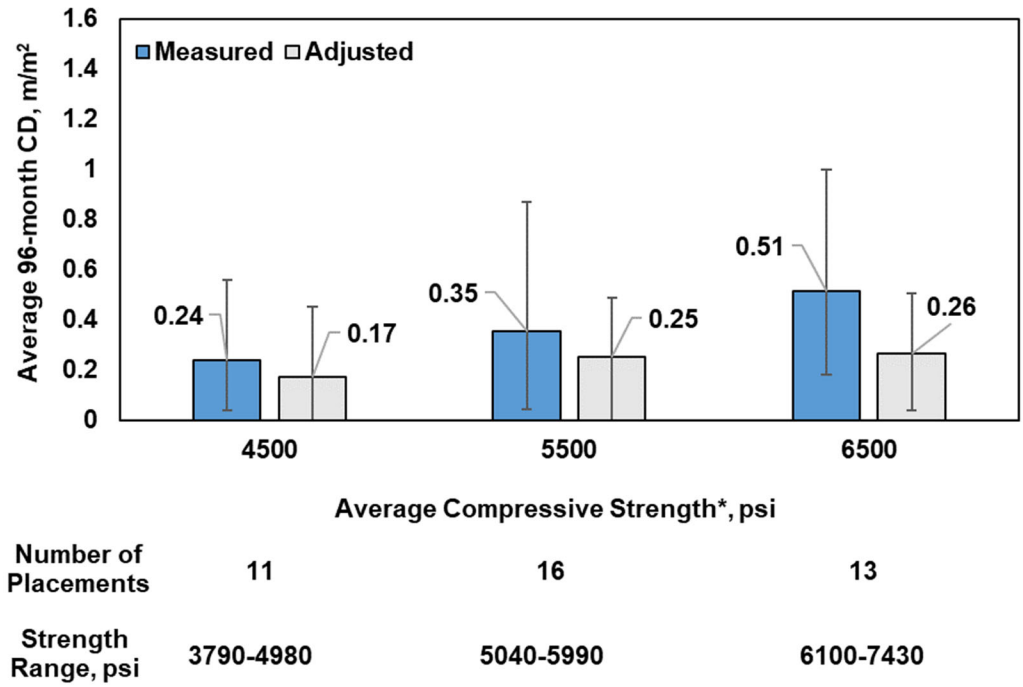


Figure 3.10–The average measured and adjusted 96-month crack densities for the bridge decks with different strength. *The average strength values are rounded to nearest 500 psi.

3.7.1.4 Air content

Air content ranges from 4.5 to 9.5% for the bridge decks evaluated in this study. Figure 3.11 shows the measured and adjusted crack densities for the bridge decks in different air content categories. As shown in Figure 3.11, the average measured crack density decreases from 0.49 to 0.25 m/m^2 when the air content increases from 5.25 to 7.25%. When the crack densities are adjusted using the Eq. (1) an increase from 5.25to 7.25% in air content results in just 0.01 m/m^2 less cracking; this difference, however, is small and not statistically significant. The higher air contents, however, will improve the freeze-thaw and salt-scaling performance, resulting in a more durable bridge deck, and reduce compressive strength. The LC-HPC specifications require a minimum of 6.5% fresh air content for bridge decks.

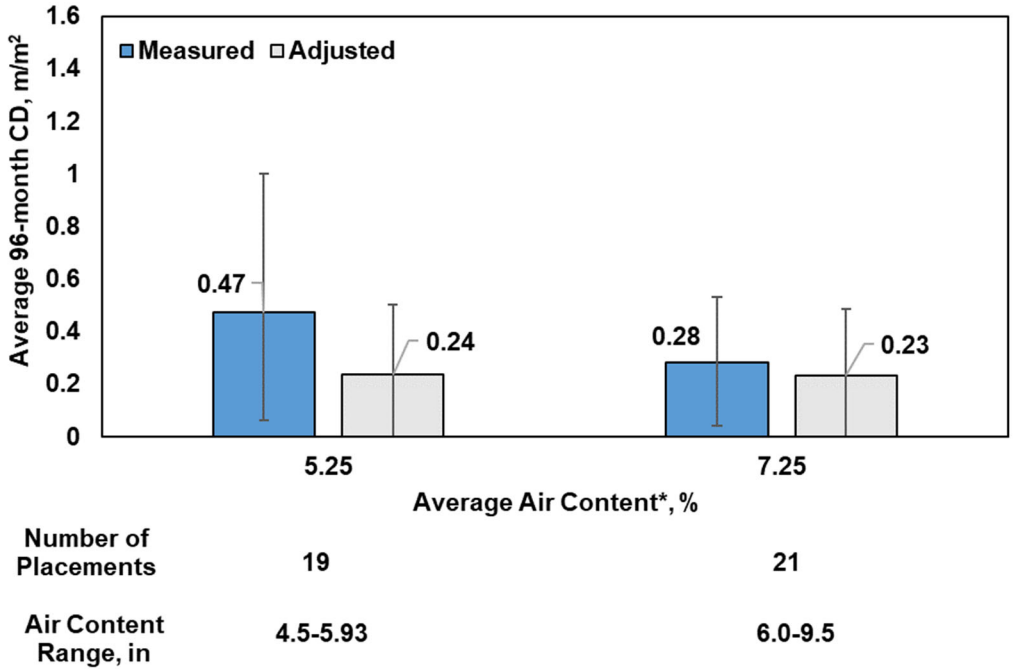


Figure 3.11–Average measured and adjusted 96-month crack density for bridge decks with different air contents. *Average air content values are rounded to nearest 0.25%.

3.7.2 Environmental Influences

Previous studies have shown that environmental conditions can greatly affect bridge deck cracking (Babaei and Purvis 1996, Krauss and Rogalla 1996, French et al. 1999). Lindquist et al. (2005 and 2006) evaluated the factors affecting cracking of CONV decks concluded that cracking increases with increasing high air temperature but did not account for the effects of other parameters on cracking, in particular that of paste content.

In his section, the effects of high air temperature and air temperature range on cracking of the 40 decks in this study are evaluated. The effect of time of the deck placement on cracking is evaluated only for the LC-HPC decks since the time of the placement was not available for CONV and Extra Control bridge decks.

3.7.2.1 High air temperature

The high air temperature on the day of placement ranges from 43° to 97°F for the 40 bridge deck placements evaluated in this study. Figure 3.12 shows the average measured 96-month crack densities for bridge decks in two high air temperature categories (58° and 83°F). As shown in Figure 3.12, the average measured crack density drops somewhat for the bridge decks as the high air temperature on the day of placement increases, with densities for the 58° and 83°F high air temperature categories decreasing from 0.39 to 0.35 m/m^2, respectively. When the crack densities are adjusted using the Eq. (1), the drop is even greater, with the bridge decks in 83°F high air temperature category exhibiting 0.10 m/m^2 less cracking than the bridge decks in 58°F. This difference, however, is not statistically significant and this observation itself is somewhat misleading because it does not account for the differences in the construction specifications between LC-HPC and CONV bridge decks. Separate analyses LC-HPC and CONV decks that follow demonstrate that placing concrete on a hot day was actually beneficial for LC-HPC decks, but not for CONV decks.

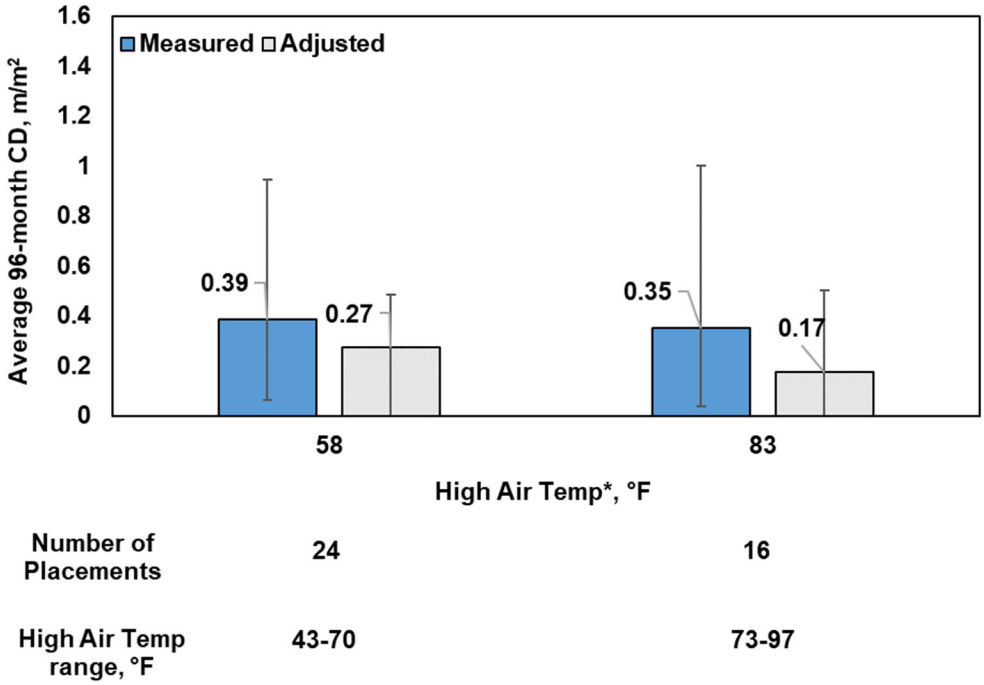


Figure 3.12–Average measured and adjusted 96-month crack densities for bridge decks placed on days with different high air temperatures. *High air temperature values are rounded to nearest 1.0°F.

Figure 3.13 shows the average measured 96-month crack densities for bridge decks in two high air temperature categories (63° and 83°F). The LC-HPC decks belonging to the 63° and 83°F high air temperature categories have crack densities of 0.39 and 0.20 m/m^2, respectively. When the crack densities are adjusted using the Eq. (1), the values change to 0.40 and 0.14 m/m^2, respectively. Concrete temperature was controlled for the LC-HPC decks, regardless of the air temperature, to values between 58° and 72°F. The lower cracking exhibited by the decks cast on warmer days result from the differences in temperature between the concrete and the bridge girders. On a hot day, the temperature of an LC-HPC deck will be less than the air temperature, and thus the temperature of the girders. As the girders cool, the concrete is placed into compression, counteracting the effects of concrete settlement over reinforcing steel and tensile stresses caused by drying shrinkage. On a cold day, the concrete had higher temperatures than the

girders, resulting in tensile stresses in deck once the girders warmed and expanded, inducing tensile stresses in the deck. This effect is known to cause cracking in bridge decks (Babaei and Purvis 1996). In addition, the construction procedures enforced for the LC-HPC bridge decks required that curing (double layers of wet burlap) be applied within 10 to 20 minutes after the concrete deck was finished and be maintained for 14 days. This precluded the formation of plastic shrinkage cracking when the concrete was placed on hot days by limiting the surface evaporation and early-age shrinkage.

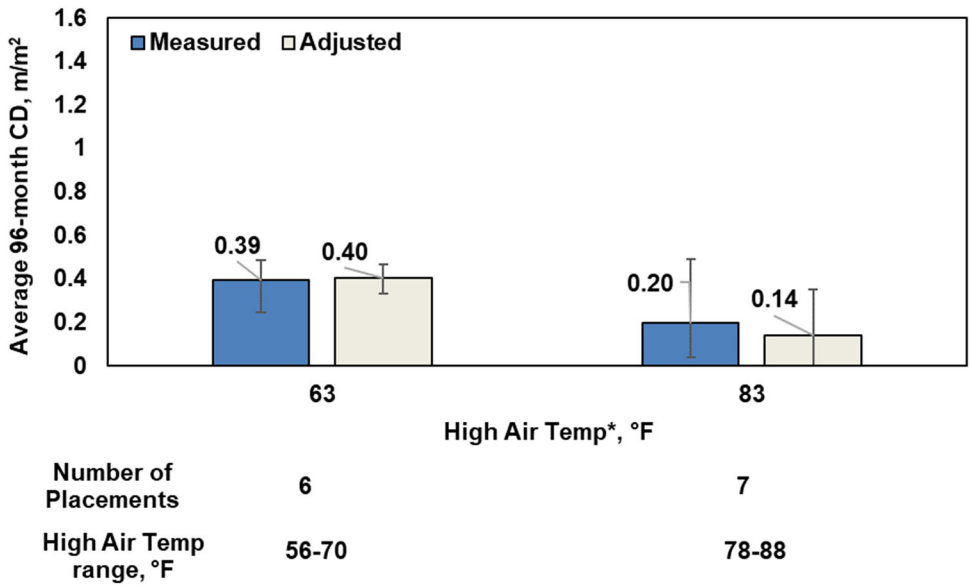


Figure 3.13–Average measured and adjusted 96-month crack densities for the LC-HPC decks placed on days with different high air temperatures. *High air temperature values are rounded to nearest 1.0 F°.

This trend is not observed for the CONV decks. As shown in Figure 3.14, which shows the average measured 96-month crack densities for the CONV bridge decks in two high air temperature categories (57° and 83°F), exhibited measured crack densities of 0.39 and 0.48 m/m^2,

respectively. When adjusted using Eq. (1), the respective values are nearly identical at 0.23 and 0.20 m/m^2. The difference is not statistically significant.

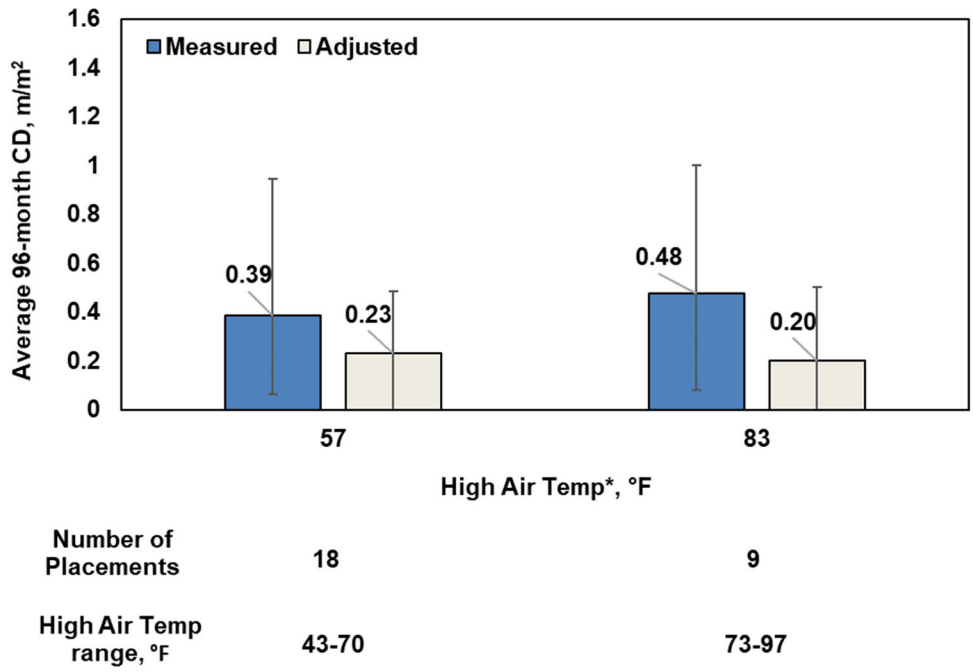


Figure 3.14- Average measured and adjusted 96-month crack densities for the CONV decks placed on days with different high air temperatures. *High air temperature values are rounded to nearest 1.0°F.

3.7.2.2 Air temperature range

Wide variations of temperature on the day of deck placement can increase the chance of cracking because the stresses induced between the steel girder and concrete deck will increase due to the differences between the coefficients of thermal expansion of the two materials. Temperature range is independent of high air temperature. The temperature ranges on the day of deck placement varied from 4° to 40°F for the 40 bridge deck placements evaluated in this study. Figure 3.15 compares average measured and adjusted 96-month crack density for the bridges in 19° and 31°F temperature range categories. The results indicate that cracking increases with increasing the range of temperature during the day of deck placement. Average measured crack densities of 0.33 and 0.43 m/m^2 were obtained for the low and high temperature range categories, respectively. The

corresponding modified crack densities are 0.18 and 0.31 m/m^2. The results demonstrate that the air temperature range, regardless of other factors, greatly affects bridge deck cracking. This finding suggests there is a great benefit to placing concrete during seasons when the variation in daily temperature is low.

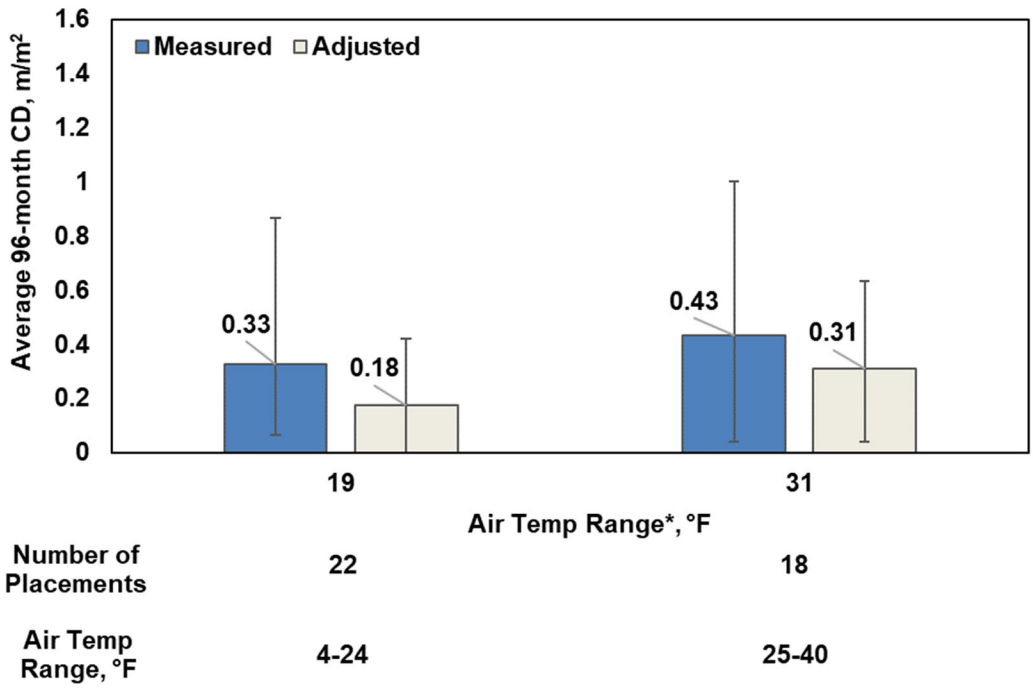


Figure 3.15–Average measured and adjusted 96-month crack densities for bridge decks cast on days with different range of temperatures. *Air temperature range values are rounded to nearest 1.0°F.

3.7.2.3 Time of placement during the day

Since KU research team members were present during the placement of the LC-HPC bridge decks and recorded the construction procedures, including the time of the deck placement, it is possible to evaluate the effect of time of the placement on deck cracking for these decks. A similar analysis is not possible for the CONV and Extra Control decks because this information is not available.

Figure 3.16 shows that the average cracking for the LC-HPC decks placed and finished between 1:30 AM and 11:10 AM ("Night Time") is lower than those placed and finished between 5:30 AM and 9:30 PM ("Day Time"), with respective values of 0.21 and 0.41 m/m^2.for measured crack densities and 0.18 and 0.39 m/m^2 for adjusted crack densities. The differences are statistically significant. These results strongly suggest that the time of placement alone will significantly affect cracking and there is a great benefit to placing and finishing concrete bridge decks between 1:00 AM and 12:00 PM.

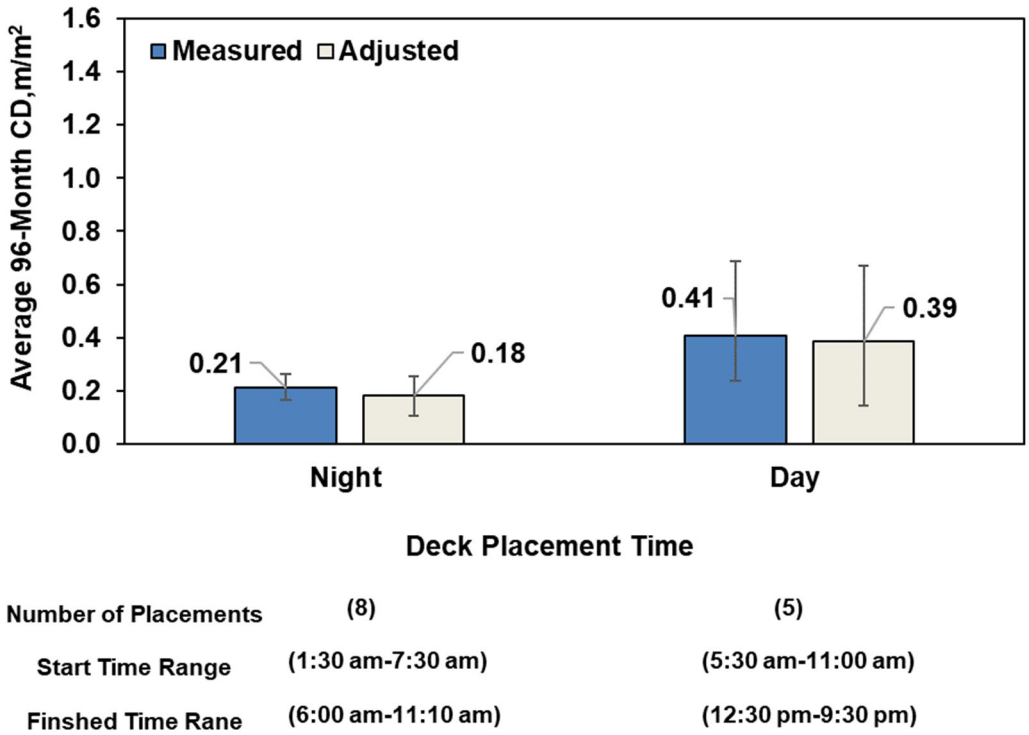


Figure 3.16–Average measured and adjusted 96-month crack densities for the LC-HPC decks placed at different time periods during the day

3.8 SUMMARY AND CONCLUSIONS

In this study, the results of the crack surveys conducted on 40 monolithic bridge deck placements supported by steel girders in Kansas are used to evaluate the effects on cracking of

material properties (paste content, slump, compressive strength, and air content) and environmental conditions (high air temperature, air temperature range, and time on the day of deck placement). Results are compared with bridge decks outside this study.

The following conclusions are based on the results of that evaluation.

1. Bridge deck cracking increases with age.
2. Paste content has a dominant effect on cracking. Paste contents above 27.2% by volume are associated with high crack densities in bridge decks, and increases in paste above 27.2% are associated with progressively greater crack densities.
3. Paste contents below 26.4% are associated with low crack densities in bridge decks, but reductions below 26.4% do not appear to result in substantial reductions in crack density.
4. Bridge decks placed on days with greater variations of temperature exhibit significantly greater cracking than those placed on days with less variation, independent of the other factors.
5. Slump, within the narrow range evaluated in this study (1½-4 in.), representing low slump concrete in current practice, has a slight effect on cracking, with greater slump values corresponding with small increases in cracking.
6. Strength (ranging from 3790 to 7430 psi) has a measurable but small effect on cracking with greater strength values corresponding with small increases in cracking.
7. Bridge deck cracking does not appear to be affected by changes in air content (ranging from 4.5 to 9.5%).
8. If concrete temperatures are maintained within a range of between approximately 55 and 70°F and early age curing is provided, bridge deck cracking decreases with increasing high air temperature on the day of casting.

9. The bridge decks placed and finished between midnight and noon exhibit lower cracking than those placed and finished between early morning and late night.

CHAPTER 4 - CONSTRUCTION PRACTICES AND BRIDGE DECK CRACKING: EXAMPLES OF BRIDGE DECKS WITH LOW-SHRINKAGE CONCRETE EXHIBITING HIGH CRACKING

4.1 GENERAL

As discussed in Chapters 1 and 3, concrete cracking is one of the major causes compromising the durability and sustainability of highway bridges. Researchers have agreed for several decades that cracking in concrete decks can be attributed primarily to restrained shrinkage or thermally-induced volume change of concrete. Although, there has been a great deal of research addressing the cracking problem, research that has resulted in the development of optimized concrete mixtures and the introduction of additional crack-reducing technologies, early-age cracking still exists in newly-constructed bridge decks. In spite of this fact, only a small number of researchers have considered or evaluated construction practices as a major factor in bridge deck cracking.

In this chapter, findings from multiple crack surveys of 28 bridge deck placements are used along with data collected during the construction to evaluate the effects of construction practices on cracking. Twenty-seven of the decks were cast with concretes with paste contents below 27.2% by volume and, thus, qualify as *low-shrinkage* concretes, as discussed in Chapter 3, while some were cast with concrete that also contained synthetic fibers as an additional crack-reducing technology. All the bridges are in Kansas, and all but four are supported by steel girders and have monolithic concrete decks in which portland cement is the only cementitious material.

The data presented in this chapter show that construction practices, in particular consolidation, have a significant effect on bridge deck cracking, and that the use of low-shrinkage concrete and synthetic fibers as crack-reducing technologies does not neutralize the negative effects of poor construction practices on cracking. The results show that contractors exhibiting poor performance on one project tended to consistently exhibit poor performance.

4.2 INTRODUCTION

Many studies have been conducted to understand the causes of bridge deck cracking (Schmitt and Darwin 1995, Babaei and Purvis 1996, Krauss and Rogalla 1996, Saadeghvaziri and Hadidi 2002, Darwin et al. 2004, Pendergrass and Darwin 2014, 2016, Hopper et al. 2015, to name just some). Most of the studies have yielded useful information: Researchers agree that most of the cracks on bridge decks are the result of volume change (due to shrinkage and/or thermal effects) of concrete resisted by restraint (abutments, composite action between girder and deck, and internal steel reinforcement of the deck) in the structure. It is an undeniable fact, however, that cracking of bridge decks, severe and early-age in many cases, still exists (Peyton et al. 2012, Polley et al. 2015). Although some studies mention construction practices as a contributing factor to cracking (Krauss and Rogalla 1996, Issa 1999, McLeod et al. 2009, Pendergrass and Darwin 2014, Hopper et al. 2015), it is hard to find studies that directly evaluate the effects of construction practices on cracking of actual bridge decks. The analysis presented in Chapter 3 showed the significant effect of paste content, regardless of other factors, on cracking. It was concluded that the bridge decks cast with concretes with paste contents greater than 27.2 % will exhibit significantly greater cracking than those cast with concretes with lower paste contents, independent of other factors. This observation, including the "threshold" paste content of 27%, is also supported by a study by Lafikes et al. (2018) of 10 bridge decks constructed in Indiana (Di Bella et al. 2012, Barrett et al. 2015) and Utah (Bitnoff 2014). In this chapter, the cracking results for the 28 bridge deck placements are presented. Twenty-four of these bridges have steel girders and monolithic concrete decks. Twenty-seven of the bridges have decks cast with concretes with paste contents of 27.2 % or less, thus qualifying as low-shrinkage concretes. Some of the decks on these bridges, however, despite having a low-shrinkage concrete and synthetic fibers, exhibited unusually high

cracking. This unusual behavior is compared with data collected during the construction and/or performance of the contractors who built those bridges.

4.3 BRIDGES

The 28 bridge deck placements include 19 low-cracking high-performance (LC-HPC) deck placements, five deck placements (two decks with two placements and one deck with one) constructed in Topeka, Kansas, and four constructed on highway US-59 south of Lawrence, Kansas. Some of the decks in this study were cast in a single placement and some in more than one placement on different days. The placements will be treated as individual decks since the environmental conditions on the day of construction and the details of construction varied between placements on the same deck. As explained in Chapters 1 and 3, the LC-HPC decks were constructed as part of a 13-year Pooled-Fund study in the state of Kansas. The LC-HPC specifications limit the cement content to between 500 and 540 lb/yd^3 and the water-to-cement ratio (*w/c*) to be 0.44 or 0.45. Concrete slump is limited to 1½ to 3½ in., compressive strength to 3500 to 5500 psi, and air content to 6.5 to 9.5%. The 19 LC-HPC placements and five Topeka placements are monolithic and supported by steel girders. Two of US-59 decks have overlays and two are monolithic; all four are supported by prestressed girders. For the 19 LC-HPC bridge decks, the cracking performance of 13 (bridge decks that were used for the analysis of the principle factors affecting cracking in Chapter 3) is used as a basis for evaluating the effects of construction practices and/or contractor performance on cracking of the bridge decks supported by steel girders since these 13 placements did not experience problems during construction. The six other LC-HPC placements: two LC-HPC-12 (referred to as the "Hartford Bridge"), one LC-HPC-13 (referred to as the "Railroad Bridge"), and three LC-HPC-14 (referred to as "OP-14"), and the five Topeka

decks did experience problems during the construction and exhibited cracking greater than the average cracking exhibited by the 13 LC-HPC decks.

The contractors (B and D) who built the Railroad and Hartford Bridges also constructed other bridge decks. This made it possible to evaluate the performance of these contractors on different projects. Contractor B constructed the Railroad bridge and two decks on US-59, 9, a deck with a silica fume overlay, and 10, a deck cast with fiber-reinforced concrete (FRC). Since the decks on US-59 9 and 10 are supported by prestressed girders and are not comparable with the LC-HPC bridge decks, the cracking behavior of these two decks is compared with that of US-59 11 and 12, that have similar properties (same girder and deck designs and same concrete type) to US-59 9 and 10, which were constructed by another contractor. Contractor D constructed the Hartford Bridge and the five bridge deck placements in Topeka, Kansas, including the three cast with FRC supported by steel girders (Topeka Fiber) and two similar decks with no fibers (Topeka Control).

The LC-HPC, US-59, and Topeka decks were surveyed by a KU research team three to 11 times (a function of deck age) after their construction and a team monitored the construction of all of the LC-HPC and two of the Topeka Fiber decks. Table 4.1 includes the date of placement, bridge type (girder and deck), paste content, measured compressive strength, air content, and slump for the 28 bridge decks. All the LC-HPC decks, including the Hartford, Railroad, and OP-14 bridges, and the Topeka decks (Fiber and Control) have monolithic decks incorporating no supplementary cementitious material and supported by steel girders with paste contents between 22.2 and 24.6%, compressive strengths between 3700 and 6400 psi, air contents between 5.5 and 9.9%, and slumps between 2¾ to 5¼ in. The concrete in the three Topeka Fiber placements have 1.5 lb/yd^3 of synthetic microfibers (0.75 in. long) in their concrete. Based on the mixture

proportions, all of these bridge decks can be categorized as being cast with low-shrinkage concrete, since the paste contents are below 27.2%. The US-59 bridge decks are supported by prestressed concrete girders. US-59 9 and 11 have silica fume overlay decks. US-59 10 and 12 have monolithic plain decks incorporating 3 lb/yd^3 of synthetic 0.75-in. long microfibers and 5 lb/yd^3 of synthetic 1.55-in. long macrofibers, respectively. The differences in properties of US-59 bridge decks and how they may have affected their cracking will be discussed in detail in Section 5.5.2.1.

Table 4.1- Date of placement, bridge type, and concrete properties

Bridge Placements	Placement Date	Bridge Type (Deck/Girder)	Paste (%)	Compressive Strength (psi)	Air Content (%)	Slump (in.)
LC-HPC 1-p*1	10/14/2005	Monolithic/Steel	24.6	5200	7.9	3¾
LC-HPC 1-p2	11/2/2005		24.6	5000	7.8	3¼
LC-HPC 2	9/13/2006		24.6	4600	7.7	3
LC-HPC 3	11/13/2007		24.4	6000	8.7	3¼
LC-HPC 4-p2	10/2/2007		23.4	4800	8.8	3
LC-HPC 5	11/14/2007		23.9	6400	8.7	3
LC-HPC 6	11/3/2007		24.4	5800	9.5	4
LC-HPC 7	6/24/2006		24.6	3800	8.0	3¾
LC-HPC 9	4/15/2009		24.2	4200	6.7	3½
LC-HPC 11	6/9/2007		23.4	4700	7.7	3
LC-HPC 15	11/10/2010		22.8	4400	9.0	3¼
LC-HPC 16	10/28/2010		22.8	5000	6.4	3¾
LC-HPC 17	9/28/2011		24.6	5200	7.0	3¼
OP-14 P1 (LC-HPC 14 p1)	12/19/2007		24.4	4400	8.7	3¾
OP-14 P2 (LC-HPC 14 p2)	5/2/2008		24.4	3700	9.8	4¼
OP-14 P3 (LC-HPC 14 p3)	5/21/2008		24.4	3800	9.9	5¼
Hartford P1 (LC-HPC 12-p1)	4/4/2008		24.3	4600	7.4	2¾
Hartford P2 (LC-HPC 12-p2)	3/18/2009		24.2	4500	7.8	4¼
Topeka Fiber 1 p1	4/11/2014	Monolithic/Steel**	22.2	5200	6.5	3¼
Topeka Fiber 2 p1	8/19/2014	Monolithic/Steel**	22.2	5300	6.5	3¼
Topeka Fiber 2 p2	8/26/2014	Monolithic/Steel**	22.2	5500	6.7	3¼
Topeka Control p1	6/13/2014	Monolithic/Steel	22.2	-#	5.5	3¼
Topeka Control p2	6/20/2014		22.2	5700	5.7	3¼
Railroad Bridge (LC-HPC 13)	4/29/2008		24.1	4300	8.1	3
US-59 9 (subdeck)	10/21/2008	Overlay/prestressed concrete	26.7	5100	6.3	3¾
US-59 9 (overlay)	-#		23.5	9100	7.0	4
US-59 10	12/6/2008	Monolithic/Prestressed Concrete**	24.6	5100	7.0	3
US-59 11 (subdeck)	10/3/2008	Overlay/prestressed concrete	27.9	4500	7.8	4¾
US-59 11 (overlay)	-#		23.5	5500	7.3	3¼
US-59 12	1/9/2009	Monolithic/prestressed Concrete**	24.6	5700	7.0	4

*p=placement, **fiber-reinforced concrete deck, #data missing or not recorded

4.4 DATA COLLECTION METHOD

4.4.1 Crack Surveys

A standard survey procedure was used to quantify the cracking performance of the bridge decks, as described in Chapter 3 and Appendix B. Individual crack survey results for the bridges used in this chapter are listed in Tables D.2 and D.3, Appendix D.

4.4.2 Construction Observations

To evaluate the effects of construction practices on bridge deck cracking, the construction practices were correlated with the crack survey results. Since construction practices can change from state to state and even from company to company, such a correlation would seem to be a reliable method to evaluate the effects of construction practices on cracking of bridge decks. Such correlations are, however, do not appear in the literature, and thus, the current study may be unique. Researchers at KU started monitoring construction of bridge decks in 2005 with the construction of the LC-HPC bridge decks. The process of observing the construction starts by coordinating with parties involved in the project, the DOT engineers, inspectors, material suppliers, contractors, and subcontractors. The plans, concrete mixture proportions and material properties, special provisions and specifications, contractor information, and location and time of the bridge placement are gathered before the construction. A team of at least three researchers, familiar with and trained in the safety requirements, who can change from one project to another, are present during the construction to document the construction procedures. The information recorded during construction include, but is not limited to, fresh concrete properties, weather information, finishing and curing procedures, and the equipment used. The information is archived for each bridge and is correlated with the results of the crack surveys to help determine the factors that may have contributed to cracking. The construction procedures for the decks in this study were monitored

by a research team from KU. Pendergrass and Darwin (2014) provide detailed accounts of the construction of the LC-HPC bridge decks. Construction of the Topeka Fiber bridge decks was monitored for this study.

Table 4.2 lists the time-to-burlap placement, indicating the time between finishing and initiation of curing, the evaporation rate calculated using the concrete and air temperatures, relative humidity, and wind speed that were measured during the construction, and the concrete temperature for the bridge decks evaluated in this chapter. Average values ranged from 6.3 to 103 minutes with extremes of 1 and 145 minutes. Values of air temperature, relative humidity, and wind speed are listed in Table D.4, Appendix D. A low value for time-to-burlap placement can be indicative of minimum finishing and an organized plan to place the burlap. By specification, the maximum allowable time to placement for two layers of burlap on LC-HPC decks is 15 minutes; 15 of the 19 LC-HPC deck placements had a time to burlap placement not exceeding 20 minutes. Both placements of Topeka Fiber 2, however, had time to burlap placements greater than 60 minutes. The evaporation rate can be related to the likelihood of plastic shrinkage cracking; the higher the rate of evaporation, the higher the potential for plastic shrinkage cracking. The evaporation rate increases with increasing concrete and air temperatures, increasing wind speed, and decreasing relative humidity. The LC-HPC specifications include a maximum evaporation rate of 0.2 lb/ft^2/hr and require additional actions to be taken to limit the evaporation if greater values are experienced during the construction, all but one LC-HPC deck placements met this requirement. All the deck placements had an evaporation rate less than 0.22 lb/ft^2/hr, except for Hartford p1 (LC-HPC 12 p1) with the value of 0.22 lb/ft^2/hr.

Table 4.2-Data collected during construction of the bridge decks

Bridge Name	Time to Burlap Placement (min.)			Evaporation Rate ($lb/ft^2/hr$)		Avg. Concrete Temp (° F)
	Avg.	Min	Max	Min	Max	
LC-HPC-1 p1	16	11	29	0.02	0.06	67
LC-HPC-1 p2	11	7	17	0.04	0.09	68
LC-HPC-2	16	10	28	0.01	0.02	67
LC-HPC-3	15	9	25	0.01	0.03	58
LC-HPC-4-p2	16	7	43	0.01	0.02	64
LC-HPC-5	12	5	22	0.02	0.05	61
LC-HPC-6	7	2	20	0.02	0.06	60
LC-HPC-7	38	11	90	0.02	0.05	71
LC-HPC-9	10	3	18	0.07	0.13	64
LC-HPC-11	14	4	19	0.02	0.07	60
LC-HPC 15	16	2	35	0.01	0.05	63
LC-HPC 16	18	10	65	0.01	0.05	59
LC-HPC 17	65	25	120	0.02	0.03	72
Hartford P1 (LC-HPC-12 p1)	7	4	12	0.01	0.05	58
Hartford P2 (LC-HPC-12 p2)	6.3	1	24	0.10	0.22	67
Railroad (LC-HPC-13)	12	2	24	0.03	0.09	69
OP-14 p1 (LC-HPC-14-p1)	28	20	40	0.06	0.08	65
OP-14 p2 (LC-HPC-14-p2)	21	12	74	0.05	0.10	64
OP-14 p3 (LC-HPC-14-p3)	15	9	21	0.03	0.12	65
Topeka Fiber 1	-#	-#	-#	-#	-#	66
Topeka Fiber 2 p1	103	63	145	0.02	0.06	83
Topeka Fiber 2 p2	65	33	115	0.03	0.09	84
Topeka Control p1	Data Not Collected					72
Topeka Control p2						79
US-59 9						71*/58**
US-59 10						64
US-59 11						76/70
US-59 12						62

*subdeck, **overlay, #data missing or not recorded

4.5 RESULTS AND DISCUSSION

4.5.1 Hartford Bridge

The deck on the Hartford Bridge (LC-HPC-12) was constructed in two placements (east and west lanes) by contractor D. Both placements consisted of a monolithic plain concrete deck supported by steel girders. The concrete was successfully placed by cranes delivering concrete from the trucks using two buckets for the first placement (east lane) on April 4, 2008 and concrete met the LC-HPC specifications (as summarized in Chapter 3) for slump, air content, concrete temperature, and compressive strength. The concrete surface received minimal finishing, followed by rapid placement of the wet burlap. The average time to burlap placement was 7 minutes, and the evaporation rate remained below 0.10 lb/ft^2/hr throughout construction (Table 4.2). Consolidation, however, was not adequate because the consolidation equipment, consisting of three hanging gang vibrators, was located about 5 ft ahead of the roller screed, which allowed workers to walk on the initially consolidated concrete, disturbing the concrete and leaving large voids in the surface (Figure 4.1), prior to finishing. This now unconsolidated concrete would have an increased likelihood of early-age settlement and with cracks forming above the transverse reinforcement.

Figure 4.1- Consolidation and finishing for the first deck placement on the Hartford Bridge showing workmen walking between the vibrator and the roller screed

The second placement (west lane) was completed on March 18, 2009, also using two crane buckets. The concrete producer had difficulty providing concrete meeting the LC-HPC slump requirements, as most of the concrete placed on the deck had slump higher than 3½ in. Other concrete properties (air content, compressive strength, and concrete temperature) were within the required specifications. As for the first placement, the concrete was finished minimally with an average time to burlap placement of 6 minutes. Although the evaporation rates (reaching to a maximum value of 0.22 lb/ft^2/hr) slightly exceeded the recommended maximum value of 0.20 lb/ft^2/hr for some sections, the 6-minute time to burlap placement likely minimized any effects of rapid evaporation (Table 4.2). The same equipment was used for the consolidation and finishing as used for the first placement, with the workers again walking on the initially consolidated concrete.

In addition, as shown in Figure 4.2, the concrete trucks and cranes were located on the recently-constructed placement 1, which applied excessive loads and resulted in noticeable torsion in the first placement.

Figure 4.2-Equipment placed on the first placement of the Hartford Bridge during the construction of the second placement

This bridge deck has been surveyed six times since 2009. The 2014 crack map in Figure 4.3 shows that both placements have high crack density dominated by long, closely-spaced transverse cracks. Likely contributors to the high crack density were the insufficient consolidation and the effects of the high loading during construction.

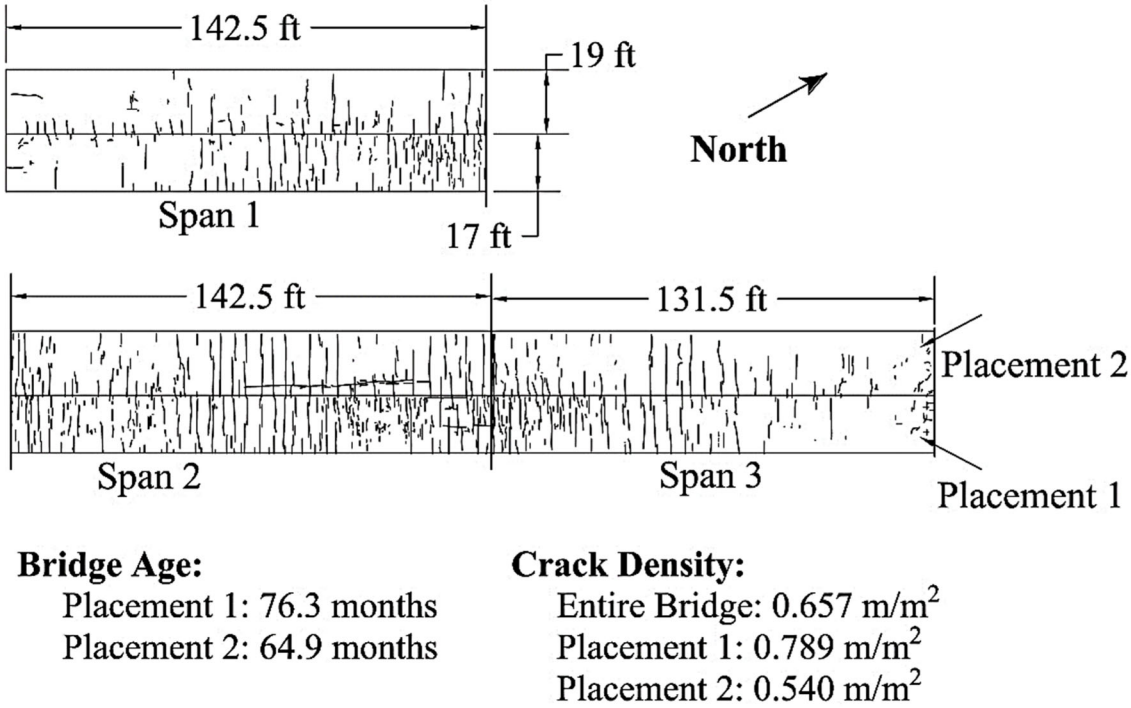


Figure 4.3-2014 Hartford Bridge crack map (Darwin et al. 2016)

4.5.1.1 Performance of Contractor D on other Bridge Decks

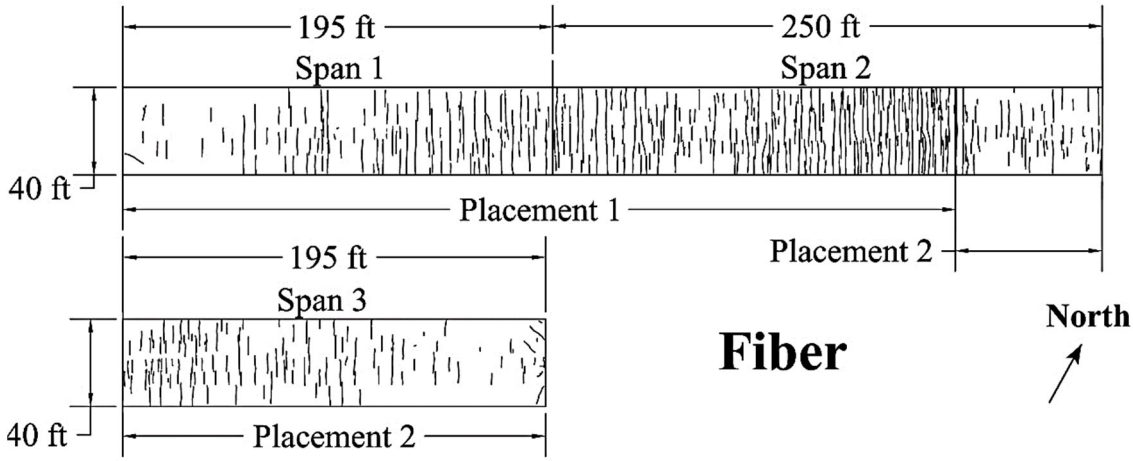
Contractor D constructed three monolithic plain bridge decks (five placements) supported by steel girders (Topeka Fiber 1, Topeka Fiber 2, and Topeka Control) in 2014. These bridge decks were not LC-HPC bridge decks. However, they are of interest because they were cast with low-shrinkage concretes. The deck on Topeka Fiber 1 was cast on April 4, 2014, and the two placements of Topeka Fiber 2 were cast on August 19 and 26, 2014. A research team from KU was present for both placements of Topeka Fiber 2, but not the others. The three bridge decks were surveyed in 2015, 2016, and 2017.

All three Topeka bridge decks had a paste content of 22.2%, and the measured properties met the requirements of the LC-HPC specifications, with the exception of concrete temperature, which was 82° and 84°F for the first and second placements of Topeka Fiber 2 bridge, respectively (Table 4.1 and Table 4.2). As listed in Table 4.2, there were significant delays in placement of the

burlap, with the average times to burlap placement of 103 and 65 minutes for placements 1 and 2, respectively. The evaporation rates, however, remained below 0.10 lb/ft^2/hr (Table 4.2) for both Topeka Fiber 2 placements. It rained for about 8 minutes right at the beginning of the first placement of Topeka Fiber 2, but the contractor decided to continue the construction. Contractor D used the same procedure for finishing and consolidation of this deck as it did for the Hartford Bridge, with wide spacing between the vibrators and the finishing equipment and workers walking in initially consolidated concrete (Figure 4.4).

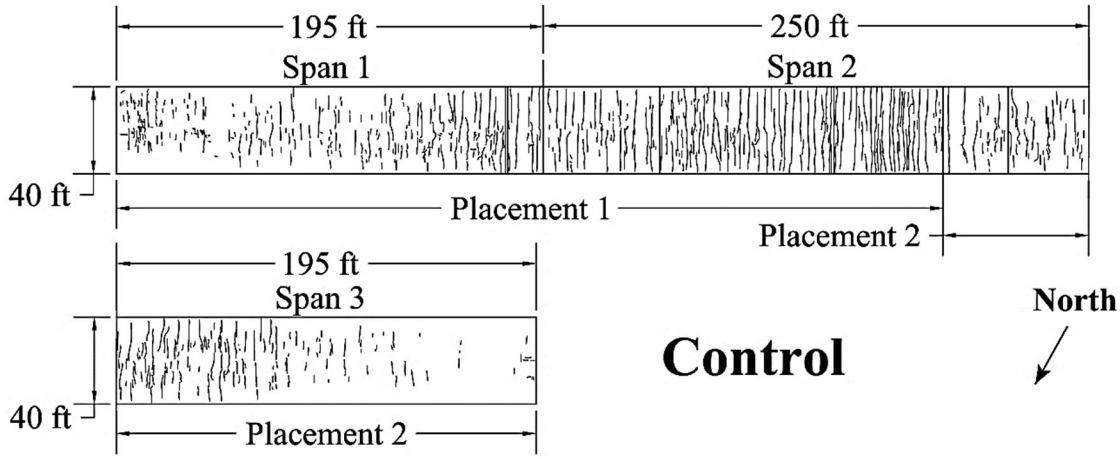
Figure 4.4-Finishing and Consolidation for Topeka Fiber 2 Placement 2

Figure 4.5 shows the 2017 crack maps for Topeka Fiber 2 and Topeka Control bridge decks. Both decks exhibit long closely-spaced transverse. This cracking pattern is very similar to that seen on both placements of the Hartford Bridge, also constructed by Contractor D. Given the low cracking of other decks with low paste contents, the conclusion that the high crack density observed on these decks resulted from loss of consolidation seems inescapable.



Bridge Age:
Placement 1: 34 months
Placement 2: 33.7 months

Crack Density:
Entire Bridge: 0.597 m/m^2
Placement 1: 0.713 m/m^2
Placement 2: 0.429 m/m^2



Bridge Age:
Placement 1: 35.8 months
Placement 2: 35.6 months

Crack Density:
Entire Bridge: 0.615 m/m^2
Placement 1: 0.766 m/m^2
Placement 2: 0.393 m/m^2

Figure 4.5-2017 crack maps for Topeka Fiber (top) and Topeka Control (bottom) bridge decks

In Figure 4.6, crack density is compared with age for the LC-HPC bridge decks (including the deck for the Hartford Bridge, but not the Railroad Bridge) and the bridge decks constructed by Contractor D in Topeka. The figure shows that the seven placements by Contractor D exhibit

cracking at or above the maximum of the other LC-HPC bridge decks. The incorporation of fibers, as a technology used to reduce cracking, does not appear to overcome the negative effects of problems occurring during the construction. Contractor D's poor performance on the Hartford Bridge clearly carried over to the Topeka bridge decks.

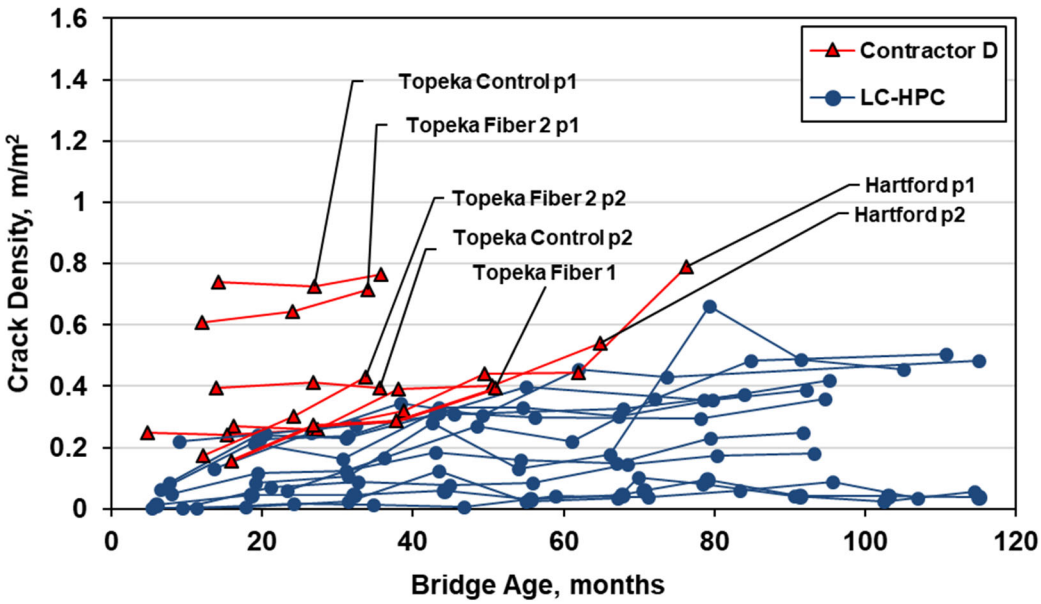


Figure 4.6-Crack density versus age for the LC-HPC bridge decks and bridge decks constructed by Contractor D

4.5.2 Railroad Bridge

The deck on the Railroad Bridge (LC-HPC-13) was constructed by Contractor B in one placement on April 29, 2008. The monolithic plain concrete deck is supported by steel girders. Placement went smoothly, and the concrete met the LC-HPC specifications. The concrete surface received minimal finishing. The average time to burlap placement was 12 minutes, and the maximum evaporation rate was below 0.10 lb/ft^2/hr (Table 4.2). The equipment used for the consolidation and finishing of the deck and the presence of workers between the vibrators and the roller screed, however, were similar to that observed on the Hartford and Topeka Bridges, as were the resulting voids in the concrete surface (Figure 4.7).

Figure 4.7–Finishing and consolidation of Railroad Bridge showing workers walking on the concrete that already has been consolidated

This bridge has been surveyed seven times, beginning in 2009. Figure 4.8 compares crack density with age for the LC-HPC bridge decks, including the Railroad bridge deck. The Hartford decks are excluded. For most of its life, the deck on the Railroad Bridge has exhibited greater cracking than the other LC-HPC decks, especially at later ages. As for the decks on the Hartford and Topeka Bridges, the loss of consolidation on Railroad Bridge deck increased the likelihood of settlement cracking. Based on photos and videos taken during construction, the only other LC-HPC bridge deck with workers walking in previously consolidated concrete was the deck on the Hartford Bridge. The usual case, from the first placement of LC-HPC-1, is illustrated in Figure 4.9.

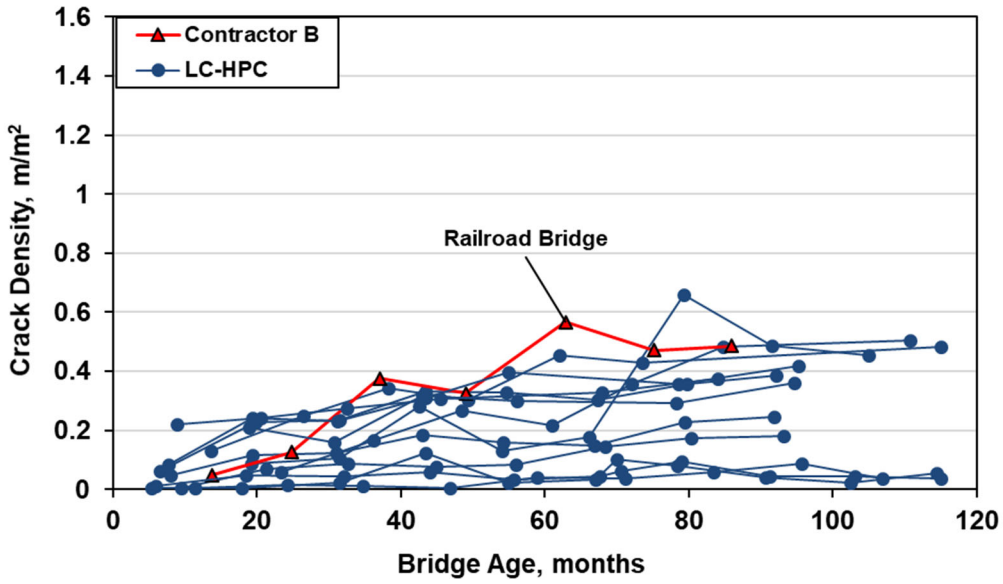


Figure 4.8–Cracking performance of Railroad bridge constructed by contractor B compared to the LC-HPC bridge decks

To avoid problems arising from loss of consolidation, specifications should prohibit workers from walking in concrete after it is consolidated. This will be aided further if the specifications require vibrators and finishing equipment be placed as possible to prevent workers from walking between these pieces of equipment.

Figure 4.9– Consolidation and finishing of the first placement of LC-HPC-1 where limited off-set between vibrators and roller screed did not allow workers to walk over the concrete that was consolidated

4.5.2.1 Performance of Contractor B on the US-59 Bridge Decks

As discussed in Chapter 5, a number of bridge decks on US-59 south of Lawrence provided an opportunity to evaluate the effects of different parameters (deck and superstructure type and concrete mixture constituents) on bridge deck cracking. Twelve bridge decks were constructed using the LC-HPC specifications as the basis for concrete mixture proportioning. Many of the decks had paste contents of 27% or less by volume, maximum compressive strengths limited to 5500 psi, and moderate water-to-cementitious materials ratios (Harley et al. 2011, Shrestha et al. 2013). KU researchers monitored the cracking performance of these decks for seven years after the construction. Results of the surveys up to 3 years are summarized by Harley et al. (2011) and Shrestha et al. (2013) and the results obtained in 2014, 2015, and 2017 by the authors of the current study are evaluated in Chapter 5. KU researchers were not present during the construction of these bridge decks.

Contractor B constructed two of the 12 US-59 bridge decks. Another contractor, Contractor C, built two other bridge decks on US-59 that were identical to the two built by Contractor B, providing an opportunity to evaluate the impact of the contractor on cracking. The decks on US-59 bridges 9 and 11 consisted of a subdeck topped with a silica fume overlay. They were constructed by Contractors B, and C, respectively. The decks on US-59 bridges 10 and 12 were monolithic FRC decks and were constructed by Contractors B and C, respectively.

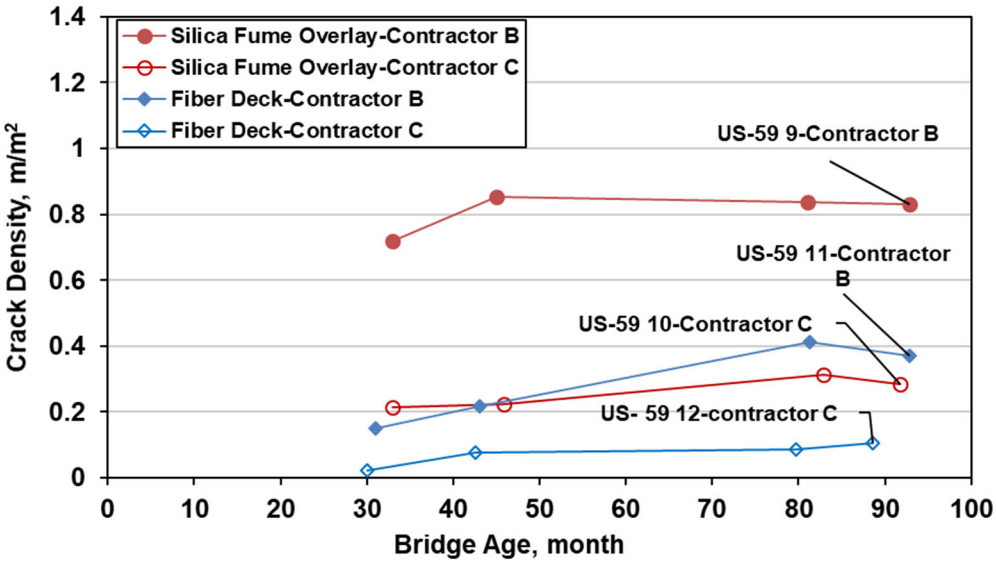


Figure 4.10-Crack density versus age for two pairs of the US-59 bridge decks constructed by contractors C and D

Figure 4.10 compares crack density with age for the bridge decks on the pairs of US-59 bridges. The figure shows that the decks with the overlays have considerably more cracking than the monolithic decks. But it also shows that there is a significant difference in cracking based on the contractor. Although, the concrete in the subdeck of US-59 bridge 9 (Contractor B) had a lower slump (3¾ versus 4¾ in.) and a lower paste content (26.7 versus 27.9 %) than the concrete in the subdeck of US-59 bridge 11, differences that should have resulted less cracking for US-59 9 than

for US-59 11, US-59 9 has almost three times the cracking at an age of 90 months. Likewise, US-59 10 and 12 had identical mixture properties, except for fiber types and dosage: US-59 10 had 5 lb/yd^3 of macrofibers, while US-59 12, 3 lb/yd^3 of microfibers. In terms of the concrete properties, US-59 10 had a lower slump (3 vs 4 in), and a lower compressive strength (5100 psi vs 5740 psi) than US-59 12. Although greater fiber dosage, a lower slump, and a lower compressive strength should have resulted in less cracking in US-59 10 deck, this deck had almost four times as much cracking compared to US-59 12 at an age of 80 months. These results strongly suggest that Contractor B used the same procedures as it did on the Railroad Bridge, with the same detrimental effect on cracking. As observed on the Topeka Fiber decks, the use of fibers did not seem to overcome the negative effects of loss of consolidation.

4.5.3 Overland Park Bridge

The Overland Park Bridge, designated OP-14, was contracted by the City of Overland Park, Kansas, unlike the LC-HPC bridge decks that were contracted by KDOT. According to detailed accounts of the construction provided by McLeod et al. (2009) and Pendergrass and Darwin (2014), the deck on OP-14 was cast in three placements. The first attempt for construction of the first placement was a failure because the contractor experienced significant problems with pumping and placing the concrete. As a result, conveyor belts were used for the second attempt, which was completed on December 19, 2007. The majority of concrete properties (air content, compressive strength, and concrete temperature) were within the limits of the LC-HPC specifications, except slump exceeded the limit of 3½ in. in many of the tests. The specified consolidation, finishing, and curing procedures, however, were not followed. The concrete was consolidated using spring-loaded gang vibrators. These vibrators popped out of the concrete after they were inserted rather than being removed slowly leaving holes in the concrete a sign of

insufficient consolidation (Figure 4.11).

Figure 4.11–Holes left on the concrete surface as a result of inadequate consolidation (McLeod et al. 2009, Pendergrass and Darwin 2014)

Finishing the concrete surface was done by a subcontractor who specialized in slabs-on-grade, for which finished surfaces of slabs are usually more highly worked and smoother than those of bridge decks. Extra effort was put to bullfloating and hand-finishing of the OP-14 concrete to achieve a smooth surface, which often was accomplished by adding water. This excessive finishing likely resulted in a thickened layer of paste at the concrete surface, increasing the potential for plastic shrinkage cracking. On bridge decks, bullfloating finishing is usually done perpendicular to the direction of traffic; however, the subcontractor for this bridge finished the concrete parallel to the traffic, requiring more time and delaying initiation of curing, which also may have contributed to cracking, with an average time to burlap placement of 28 minutes and

some sections as high as 40 minutes. The evaporation rates throughout the construction remained below 0.10 lb/ft^2/hr (Table 4.2).

Similar problems were encountered in second and third placements, which took place on May 2, and May 21, respectively. Four surveys were conducted on the three placements of OP-14 starting in 2009. Figure 4.12 compares the crack density with age for the LC-HPC bridge decks and the three placements of OP-14. The three placements of OP-14 have crack densities that are well above the top of the range for the LC-HPC bridge decks at similar ages. This once again illustrates that bridge decks having similar mixture properties can exhibit significantly different cracking behavior as a function of the construction procedures used. Insufficient consolidation coupled with unusually excessive finishing of the three placements of the OP-14 bridge are the most likely causes of the significantly greater cracking observed in the OP bridge decks compared to that observed in the LC-HPC bridge decks.

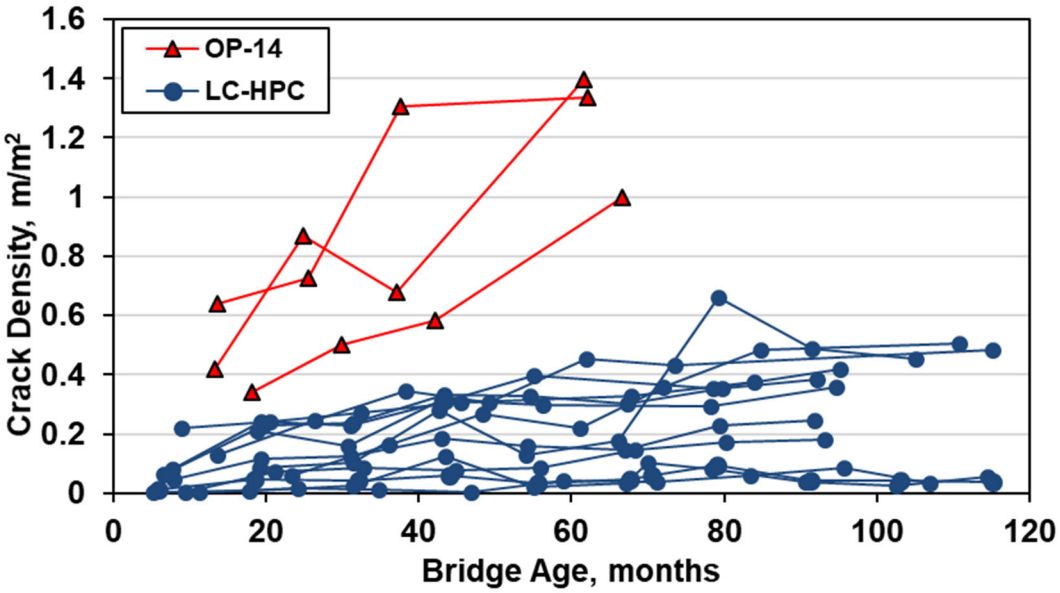


Figure 4.12–Crack density versus age for the LC-HPC and OP-14 bridge deck placements

4.6 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

In this chapter, findings from multiple crack surveys of 28 bridge deck placements are used along with data collected during the construction to evaluate the effects of construction practices on cracking. All but one of the decks was cast with low-shrinkage concrete with a paste content of 27% or less. Some of the decks, however, exhibited high cracking that can be correlated with the construction procedures.

The following conclusions are based on the comparisons between the measured crack densities and construction procedures used on the decks.

1. Bridge decks with similar material properties, mixture proportions, and structural properties can exhibit different cracking behavior that can be tied to construction practices.
2. The negative effects of construction practices can neutralize the positive effects of a low-shrinkage concrete and other crack-reducing technologies, such as fibers.
3. Insufficient consolidation, loss of consolidation caused by workers walking in consolidated concrete, and over finishing can significantly increase cracking in bridge decks.

Based on the findings described in this chapter, it is recommended (1) that concrete be finished minimally to limit the paste content on the deck surface, resulting in lower drying shrinkage at the surface, and allowing curing to be initiated as soon as possible, and (2) that specifications prohibit workers from walking in concrete after it is consolidated. The latter will be aided if the specifications also require vibrators and finishing equipment be placed as close to each other as possible to prevent workers from walking between these pieces of equipment.

CHAPTER 5 - EVALUATION OF CRACKING IN BRIDGES WITH DIFFERENT SUPERSTRUCTURE AND DECK TYPES

5.1 GENERAL

In this chapter, data obtained through six crack surveys at ages of 22 through 100 months are used to evaluate the effects of a number of parameters on the cracking performance of 12 bridge decks. The bridges are located on highway US-59, south of Lawrence, Kansas, and were constructed in 2008 and 2009. The parameters include the contractor, superstructure type (prestressed concrete or steel), deck type (monolithic, precast partial-depth concrete panels with cast-in-place topping, or silica fume overlays), and concrete type (binary, ternary, or fiber-reinforced).

Although limited by the sample size, the results indicate that the cracking performance of two of the bridge decks was highly influenced by the contractor. In addition, the bridge decks incorporating precast concrete deck panels supported by precast prestressed girders exhibited significantly less cracking than other decks in this study, as well as similar decks constructed in elsewhere. Among bridges with overlays supported by steel girders, incorporation of fibers did not improve the cracking performance. The average of the measured crack widths ranged from 0.005 to 0.012 in. and greater average crack widths corresponded with greater crack densities.

5.2 INTRODUCTION

As discussed in Chapters 1 and 3, cracking in concrete bridge decks can negatively affect their durability and sustainability. As a response to the bridge deck cracking problem, in 2005, the Kansas Department of Transportation (KDOT) in cooperation with the University of Kansas (KU) and other state DOTs started a 13-year Pooled-Fund study on the "*Construction of Crack-Free Bridge Decks*," that resulted in the construction of over 20 bridge decks in Kansas and other states following the low-cracking high-performance concrete (LC-HPC) specifications, which were

based on prior studies in Kansas by Schmitt and Darwin (1995, 1999), Miller and Darwin (2000), Darwin et al. (2004), and Lindquist et al. (2005, 2006). The LC-HPC specifications, which successfully mitigate bridge deck cracking and improve durability (Lindquist et al. 2008, McLeod et al. 2009, Darwin et al. 2010, Yuan et al. 2011, Pendergrass and Darwin 2014, Alhmood et al. 2015, Darwin et al. 2016), require the cement contents between 500 and 540 lb/yd^3, a water-to-cement ratio (*w/c*) of 0.44 or 0.45, and a cement paste content (cement and water) of 27% or less by volume. Concrete slump is limited to 1½ to 3½ in., compressive strength to 3500 to 5500 psi, and air content to 6.5 to 9.5%. Construction requirements cover concrete temperature, evaporation rates, consolidation, finishing, and curing. Parallel to construction of the LC-HPC bridge decks, KDOT started a project in 2008 to evaluate the effects of deck and superstructure type, and concretes with various binder compositions and synthetic fibers on cracking. The project involved 12 bridge decks constructed by three contractors on highway US-59 south of Lawrence, Kansas. Many of the US-59 decks have concrete mixture proportions and material properties similar to those required by the LC-HPC specifications providing low paste contents (less than 27%), a maximum concrete compressive strength of 5500 psi, and moderate *w/cm* ratios, thus qualifying the materials as *low-shrinkage* concrete. The cracking performance of these bridge decks has been described by Harley et al. (2011) and Shrestha et al. (2013). This chapter provides an update, including the results of crack surveys through 2016.

Although the small number of bridges in this study makes it difficult to reach solid conclusions based solely on this chapter, the observations from previous chapters and other studies are used here to see if the expected trends can be observed here. The analyses in Chapters 3 and 4, using the data from multiple surveys on more than 50 bridges with similar decks (monolithic), concrete (100% portland cement), and superstructure type (steel girders), showed that bridge decks

cast with concrete with paste contents greater than 27.2% experienced significantly greater cracking than those with lower paste contents and that other material properties, such as slump, compressive strength, and air content, have much less effect on cracking. Because most US-59 bridge decks were all cast with, low-shrinkage concrete the results of the crack surveys of those decks are used to discuss the effects of deck type (monolithic, precast panels, or silica fume overlays), concrete type (binary, ternary, or fiber-reinforced), and superstructure type (prestressed or steel girders) on cracking. Based on the findings of Lindquist et al. (2005 and 2006) and Hopper et al. (2015), it is expected that bridges with monolithic decks perform better than those with overlay decks and bridges supported by prestressed girders perform better than those supported by steel girders. This type of analysis was not possible on Chapters 3 and 4 bridges because most of the decks had identical deck, concrete, and superstructure types.

The analysis in Chapter 4 showed that cracking can be significantly affected by construction practices. Acknowledging this finding, the performance of two pairs of US-59 bridges that had identical properties but were constructed by different contractors (B and C) are discussed to assess the direct influence of these two contractors on cracking. This comparison is of interest because, as discussed in Chapter 4, Contractor B also constructed one of the poorest-performing LC-HPC bridges. Based on the results of Chapter 4, it would be expected that the bridges constructed by Contractor B would exhibit noticeably greater cracking. The results from Chapter 4 also indicate that incorporation of fibers, alone, cannot guarantee a low-cracking performance. This observation is used in this chapter to evaluate the effectiveness of fibers in two US-59 decks.

Another point of interest in this chapter is the evaluation of the performance of bridges with deck panels. The states of Utah (Bitnoff 2014) and Missouri (Wenzlick 2005) have had unsuccessful experiences with the use of partial-depth precast deck panels with cast-in-place (CIP)

concrete toppings. In contrast, the four US-59 deck panel bridges on US-59 have performed very well. As will be discussed, a key reason may be the low paste content used in the CIP topping concrete on the US-59 bridges.

Experimental studies by multiple researchers have shown that cracks having a width as small as 0.003 in. can provide a path for deicing agents to penetrate concrete (Rodriguez and Hooton 2003, Pease 2010). There is not much data on the width of cracks on bridge decks. To correct this lack of data, more than 1000 measured crack widths from recent surveys of the bridge decks on US-59 bridges help quantify the average crack width and compare the relationship between crack width and crack density.

5.3 PROPERTIES OF BRIDGES

Table 5.1 lists the location and date of placement and Table 5.2 lists the properties of the US-59 bridge decks. The 12 decks include four supported by steel girders and eight supported by prestressed concrete girders. Among the four steel-girder bridges, two have monolithic decks (Steel-M), one a silica fume overlay deck (Steel-O), and one a silica fume overlay deck incorporating synthetic fibers (Steel-OF). Among the prestressed concrete-girder bridges, two have monolithic decks incorporating synthetic fibers (Steel-MF), two have silica fume overlay decks (PS-O), and four have precast partial-depth concrete deck panels with a CIP concrete topping (PS-DP). All decks have a total thickness of 8 ½ in. and clear cover to the top reinforcement of 3 in. The bridges with overlay decks have a 7 in.-thick subdeck and 1½ in.-thick concrete overlay containing cement and slag cement (binary mixture) or cement, slag cement, and silica fume ternary (ternary mixture). The partial-depth precast concrete deck panels were 3 in. thick deck panels and had a 5½ in. CIP binary or ternary concrete topping. Three contractors (A, B, and C)

constructed the 12 bridge decks. Additional details such as bridge number, skew, span length, transverse steel size and spacing are listed in Table 5.2.

Table 5.1—The US-59 bridge locations

Bridge Name	Bridge Location	Placement Date
US-59 1	SB US-59 over Sand Creek Rd.	11/3/2008
US-59 2	NB US-59 over Sand Creek Rd.	11/25/2008
US-59 3	SB US-59 over BNSF R.R.	9/30/2008
US-59 4	NB US-59 over BNSF R.R.	9/19/2008
US-59 5	SB US-59 over Midland R.R.	5/14/2008
US-59 6	NB US-59 over Midland R.R.	4/30/2008
US-59 7	SB US-59 over I-35	11/1/2008
US-59 8	NB US-59 over I-35	10/29/2008
US-59 9	SB US-59 over Stafford Rd.	10/21/2008
US-59 10	NB US-59 over Stafford Rd.	12/6/2008
US-59 11	SB US-59 over Stafford Rd.	10/3/2008
US-59 12	NB US-59 over Stafford Rd.	1/9/2009

Table 5.2—Bridge properties

Bridge ID	Contractor	Girder and Deck Type*	Bridge Skew	Bridge Length		Total Deck Thickness		Transverse Steel				Angle of Reinf.
								Size		Spacing		
			(deg.)	(ft)	(m)	(in.)	(mm)	No.	(mm)	(in.)	(mm)	(deg.)
US-59 1	A	Steel-M	45.63	387.9	118.2	8.5	216	5	16	6	152	0
US-59 2	A	Steel-M	45.63	387.9	118.2	8.5	216	5	16	6	152	0
US-59 3	A	PS-DP	8.43	242.9	74.0	8.5	216	5	16	6	152	0
US-59 4	A	PS-DP	8.43	242.9	74.0	8.5	216	5	16	6	152	0
US-59 5	A	Steel-OF	39.17	264.8	80.7	8.5	216	5	16	7	178	0
US-59 6	A	Steel-O	39.17	266.2	81.1	8.5	216	5	16	7	178	0
US-59 7	A	PS-DP	2.3	333.5	101.7	8.5	216	5	16	7	178	0
US-59 8	A	PS-DP	2.3	333.5	101.7	8.5	216	5	16	7	178	0
US-59 9	B	PS-O	0	225.5	68.7	8.5	216	5	16	6	152	0
US-59 10	B	PS-MF	0	225.5	68.7	8.5	216	5	16	6	152	0
US-59 11	C	PS-O	0	196.5	52.6	8.5	216	5	16	7	178	0
US-59 12	C	PS-MF	0	196.5	52.6	8.5	216	5	16	7	178	0

*PS = Prestressed concrete girder, DP = Deck panels, O = Deck with silica fume overlay, M = Monolithic deck
F = Fibers in the deck or overlay

Table 5.3 and Table 5.4 list, respectively, the mixture proportions for the decks, subdecks, and CIP concrete toppings of the deck panels and the mixture proportions for the overlays. The concretes for the decks, subdecks, and toppings have *w/cm* ratios ranging from 0.42 to 0.45, paste contents ranging from 23.99 to 27.95%, and were binary or ternary mixtures, with the exception of the subdecks of the decks with overlays (US-59 5, 6, 9, and 11) and US-59 10 and 12 in which only portland cement was used. Two types of coarse aggregate, crushed limestone and crushed

granite, and one type of fine aggregate, river sand, were used. Macrofibers (F-3), a blend of polypropylene and polyethylene, at the dosage rate of 5 lb/yd^3 were used in US-59 10 and microfibers (F-6), virgin polypropylene in monofilament forms, at a dosage rate of 3 lb/yd^3 were used in US-59 12.

Table 5.3—Mixture proportions for decks or subdecks of decks with silica fume overlays

Bridge ID	Cementitious Material**	Fibers in Deck	Aggregates by Weight$	Water Content (lb/yd^3)	Water Content (kg/m^3)	Cementitious Material (lb/yd^3)	Cementitious Material (kg/m^3)	*w/cm*	Paste Vol. %
US-59 1	60% C, 35% S., 5% SF	NA	45% CA-2, 15.2% CA-3, 39.8% FA	225	134	540	317	0.42	23.99
US-59 2	60% C, 35% S, 5% SF	NA	45% CA-2, 15.2% CA-3, 39.8% FA	225	133	540	318	0.42	23.99
US-59 3	65% C, 35% S	NA	45% CA-2, 15.2% CA-3, 39.8% FA	241	143	540	317	0.45	24.77
US-59 4	65% C, 35% S	NA	45% CA-2, 15.2% CA-3, 39.8% FA	241	143	540	317	0.45	24.77
US-59 5*	100% C	NA	50% CA-1, 50% FA	274	163	620	369	0.44	27.95
US-59 6*	100% C	NA	50% CA-1, 50% FA	274	163	620	369	0.44	27.95
US-59 7	60% C, 35% S, 5% SF	NA	45% CA-2, 15.2% CA-3, 39.8% FA	225	134	540	317	0.42	23.99
US-59 8	60% C, 35% S, 5% SF	NA	45% CA-2, 15.2% CA-3, 39.8% FA	225	134	540	317	0.42	23.99
US-59 9*	100% C	NA	50% CA-1, 50% FA	259	154	600	358	0.44	26.68
US-59 10	100% C	5 lb/yd^3 F-3#	50% CA-1, 50% FA	237	141	560	334	0.42	24.62
US-59 11*	100% C	NA	50% CA-1, 50% FA	274	163	620	369	0.44	27.95
US-59 12	100% C	3 lb/yd^3 F-6#	50% CA-1, 50% FA	237	141	560	334	0.42	24.62

*Bridge decks have overlays and proportions are for the subdecks.
**C = Cement, S = Slag, SF = Silica fume
$CA-1= ½ in. Crushed limestone, CA-2 = ¾ in. Crushed granite, CA-3= ½ in. Crushed granite , FA= River sand
#F-3= 1.55 in. long polyolefin macro fibers, F-6 = ¾ in. long fibrillated polypropylene microfibers

The concrete used for the overlays on US-59 5, 6, 9, and 11 (Table 5.4) has a *w/cm* ratio of 0.37 and a paste content of 23.54 %. The overlays for US-59 5 and 6 have a ternary binder

composition and US-59 9 and 10, a binary binder composition. All overlays used crushed limestone as the coarse aggregate and river sand as the fine aggregate. US-59 5 contains F-3 fiber at the dosage rate of 5 lb/yd^3 in its overlay.

Table 5.4—Mixture proportions for silica fume overlay

Bridge ID	Cementitious Material*	Fibers in Overlay	Aggregates by Weight**	Water Content		Cementitious Material		*w/cm*	Paste Vol.
				(lb/yd^3)	(kg/m^3)	(lb/yd^3)	(kg/m^3)		%
US-59 5	66% C, 30.1 S, 3.9% SF	5 lb/yd^3 F-3	50% CA-1 50% FA	239	142	645	382	0.37	23.54
US-59 6	66% C, 30.1 S, 3.9% SF	NA	50% CA-1 50% FA	239	142	645	382	0.37	23.54
US-59 9	92.2% C, 7.8% SF	NA	50% CA-1 50% FA	239	142	645	382	0.37	23.54
US-59 11	92.2% C, 7.8% SF	NA	50% CA-1 50% FA	239	142	645	382	0.37	23.54

*C = Cement, S = Slag, SF = Silica fume

**CA-1= ½ in. Crushed limestone, FA= River sand

Table 5.5—Average plastic concrete properties and concrete compressive strength

Bridge ID	Slump		Air Content	Average Concrete Temp		28-Day Strength	
	(in.)	(mm.)	(%)	(°F)	(°C)	(psi)	(MPa)
US-59 1	4	100	6.5	65.5	18.6	5090	35.1
US-59 2	3½	90	6.75	65.3	18.5	6390	44.1
US-59 3	4	100	7.25	76.9	24.9	4260	29.4
US-59 4	4	100	6.75	78.7	26	5000	34.5
US-59 5*	5	130	6.75	65	18.3	5010	34.5
US-59 6*	4½	115	6.25	66	18.9	4850	33.4
US-59 7	3¼	80	6.25	68.3	20.2	4960	32.5
US-59 8	2½	65	6.25	66.2	19	4580	31.6
US-59 9*	3¾	95	6.25	71.3	21.8	5110	35.2
US-59 10	3	75	7	63.7	17.6	5100	35.2
US-59 11*	4¾	120	7.75	76.3	24.6	4480	30.9
US-59 12	4	100	7	61.5	16.4	5740	39.6

*Bridge decks have overlays and properties listed are for the subdecks.

Table 5.5 lists the concrete properties (slump, air content, concrete temperature, and 28-day compressive strength) for the monolithic decks, overlay subdecks, and CIP concrete toppings

of the bridges with deck panels. Slump ranges from 2½ to 4¾ in., air content from 6.25 % to 7.75 %, and compressive strength from 4260 to 6390 psi. The concrete properties for the overlays are listed in Table 5.6. Slump ranges from ¾ to 4½ in., air content from 6.75 to 7.75 %, and compressive strength from 5470 to 9100 psi.

Table 5.6—Average Overlay Plastic Concrete Properties and Compressive Strengths

Bridge ID	Slump		Air Content	Average Concrete Temp		28-Day Strength	
	(in.)	(mm.)	(%)	(°F)	(°C)	(psi)	(MPa)
US-59 5	4½	115	6.75	81	27.2	6450	44.5
US-59 6	¾	20	7.75	74	23.3	7480	51.6
US-59 9	4	100	7	58	14.4	9100	62.7
US-59 11	3¼	85	7.25	70	21.1	5470	37.7

5.4 DATA COLLECTION METHOD

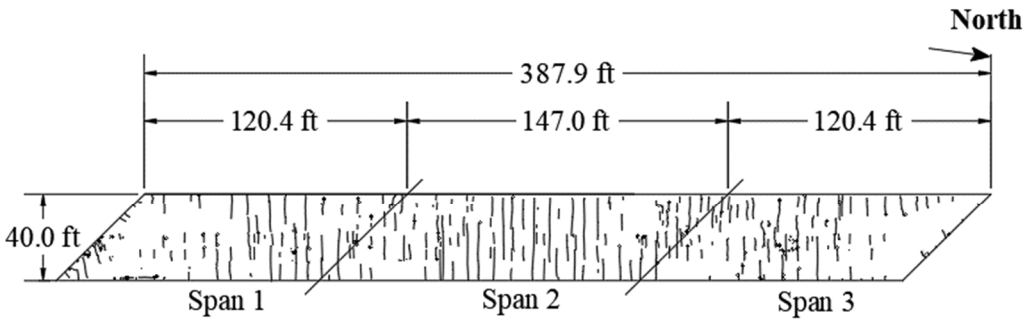
The results of crack surveys are used to evaluate the cracking performance of the bridge decks in this study. Appendix B includes the complete crack survey specifications.

5.5 RESULTS AND DISCUSSION

A total of 67 crack surveys were conducted between 2010 and 2016. The bridge decks had ages between 89 and 100 months at the time of their last survey. Table 5.7 summarizes the crack densities measured during the surveys taken in 2014, 2015, and 2016. The results for surveys before 2014 are reported by Harley et al. (2011) and Shrestha et al. (2013). Table E.1 in Appendix E includes all the individual crack survey results for the decks. Figure 5.1 shows the crack map obtained in 2016 for US-59 1 (a steel-M bridge). As with the other bridge decks on US-59, the majority of cracks are transverse. For the decks with the partial-depth precast concrete deck panels, the transverse cracks mainly appear near the precast deck joints, as illustrated in Figure 5.2 for US-59 3 (a PS-DP bridge).

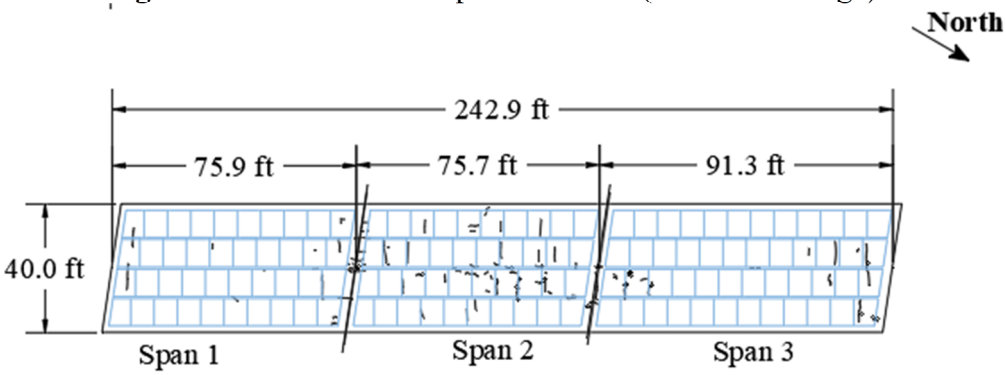
Table 5.7—2014, 2015, 2016, and 96-month crack densities for decks on US-59 Bridges

Bridge Name	2014 Crack Density (m/m^2)	2015 Crack Density (m/m^2)	2016 Crack Density (m/m^2)	96-Month Crack Density (m/m^2)
US-59 1	0.555	Did not survey	0.543	0.543
US-59 2	0.373	Did not survey	0.383	0.385
US-59 3	0.088	0.056	0.091	0.091
US-59 4	0.115	0.153	0.108	0.108
US-59 5	0.633	Did not survey	0.559	0.559
US-59 6	0.395	Did not survey	0.412	0.412
US-59 7	0.036	0.031	0.023	0.023
US-59 8	0.073	0.081	0.064	0.064
US-59 9	1.51	0.838	0.832	0.832
US-59 10	did not survey	0.411	0.396	0.372
US-59 11	did not survey	0.314	0.285	0.285
US-59 12	did not survey	0.086	0.104	0.119



Bridge Age: 92.3 months **Crack Density:** 0.306 m/m^2

Figure 5.1—2016 crack map for US-59 1 (a Steel-M bridge)



Bridge Age: 91.6 months **Crack Density:** 0.091 m/m^2

Figure 5.2—2016 crack map for US-59 3 (a PS-DP bridge)

Figure 5.3 compares crack density with age for the decks on the US-59 bridges. Cracking behavior differs, even for similar bridges and decks, significantly in some cases, although the four PS-DP bridge decks exhibit consistently low cracking. In the following sections, the factors that

may cause this differing behavior are discussed. The factors include the contractor, superstructure type, deck type, and binder type.

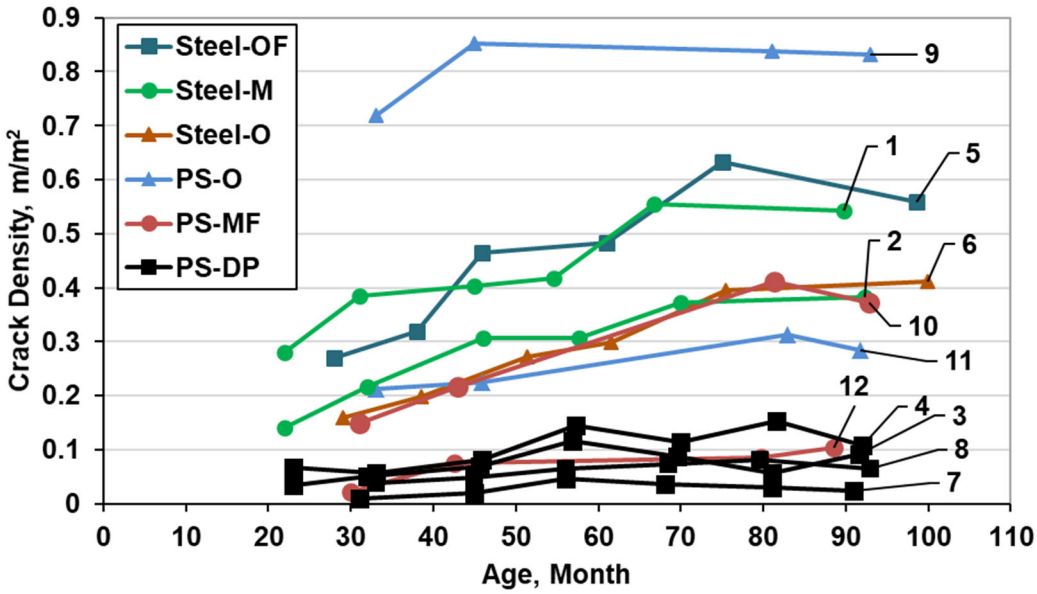


Figure 5.3—Age versus crack density for US-59 bridges
Note: Numbers next to each curve refer to the US-59 bridges number as listed in Table 5.1

As concluded in Chapter 3 and shown in Figure 5.3, for many of the US-59 bridges, deck cracking varies, and in most cases increases, with age. When comparisons are made between the US-59 bridge decks, the values of crack density at 96 months are used. Data taken from a survey at an age of 96 ±6 months is used to establish the crack density at 96 months. For US-59 12, which was younger than 90 months at the time of its last survey, linear extrapolation between the latest two crack densities at ages of 80 and 89 months is used to estimate the 96-month crack density.

5.6 EVALUATION OF FACTORS AFFECTING CRACKING

5.6.1 Contractor

As discussed in Chapter 4, construction practices can significantly affect cracking of bridge decks, even decks with similar properties. As shown in Table 5.2, the 12 US-59 bridge decks were constructed by Contractors A, B, and C. Contractor A constructed eight bridges, while Contractors

B and C constructed two each. Contractor B also constructed one of the worst-performing LC-HPC bridges, as discussed in Chapter 4. Figure 5.4 shows the average 96-month crack densities for the US-59 bridges based on contractor. Figure 5.4 cannot be used to directly compare the three contractors because of differences among the bridges other than the contractor, it can be seen that decks cast by Contractor B perform markedly worse, with an average crack density of 0.602 m/m^2, compared to values of 0.273 m/m^2 and 0.202 m/m^2 for Contractors A and C, respectively. In Figure 5.3, the bridges constructed by Contractor B are US-59 9 and 10. Both decks are supported by prestressed girders and exhibit more cracking than the other six decks supported by prestressed girders. Further, US-59 9 has the greatest cracking of the 12 decks evaluated in this study. The difference between the average crack densities for the decks constructed by Contractors A and C, 0.071 m/m^2, is not as significant as the differences for both with respect with Contractor B and may be related to differences in superstructure or concrete used in those decks, as discussed in the following sections.

While Figure 5.4 cannot be used to directly compare the three contractors, it is possible to directly illustrate the effect of the contractor by comparing the cracking performance of two pairs of bridges, where one in each pair was constructed by Contractors B and C. In this case, the bridge decks constructed by Contractor B exhibit significantly greater cracking than those constructed by Contractor C.

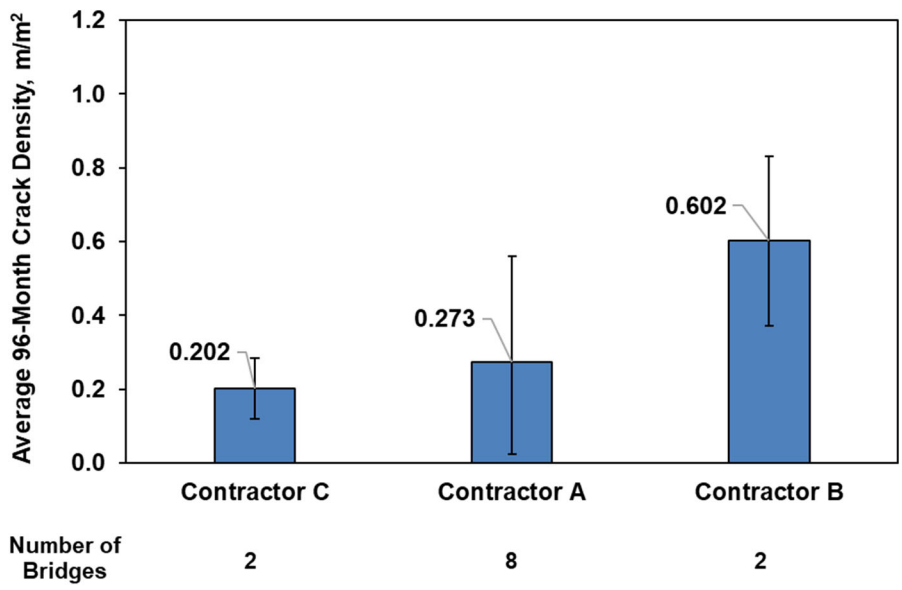


Figure 5.4—Average 96-month crack densities for the US-59 bridges based on their contractors

Since the two pairs of the bridges constructed by Contractor B (US-59 9 and 10) and C (US-59 11 and 12) have identical deck and superstructure types, as shown in Table 5.2 through Table 5.6, they provide an opportunity to evaluate the direct effect of the contractor on cracking.

Figure 5.5 compares crack density with age for the two overlay decks supported by prestressed girders, US-59 9 by Contractor B and US-59 11 by Contractor C. The figure shows a significant difference in cracking between these two bridge decks, where the deck by Contractor B exhibits cracking consistently greater than 0.70 m/m^2, versus the cracking exhibited by the deck constructed by Contractor C that does not exceed 0.32 m/m^2. The subdeck on US-59 9 has a paste content of 26.68%, slump of 3¾ in., compressive strength of 5110 psi, and air content of 6.25%. The subdeck on US-59 11 has a paste content of 27.95%, slump of 4¾ in., compressive strength of 4480 psi, and air content of 7.75%. The overlays on both US-59 9 and 11 have paste contents of 23.54%. The overlay on US-59 9 has a slump of 4 in., compressive strength of 9100 psi, and air content of 7 %. The overlay on US-59 11 overlay has a slump of 3¼ in., compressive strength of

5470 psi, and air content of 7.25%. As discussed in Chapter 3, variations in slump, compressive strength, and air content have much less effect on cracking than paste content, particularly when the paste content exceeds 27.2%. US-59 11, with a subdeck paste content of 27.95% (exceeding 27.2%), would have been expected to exhibit greater cracking than US-59 9, with a subdeck paste content of 26.68%. At the age of about 80 months, however, US-59 11 by Contractor C has a crack density of 0.314 m/m^2 versus that of 0.838 m/m^2 exhibited by US-59 9 by Contractor B.

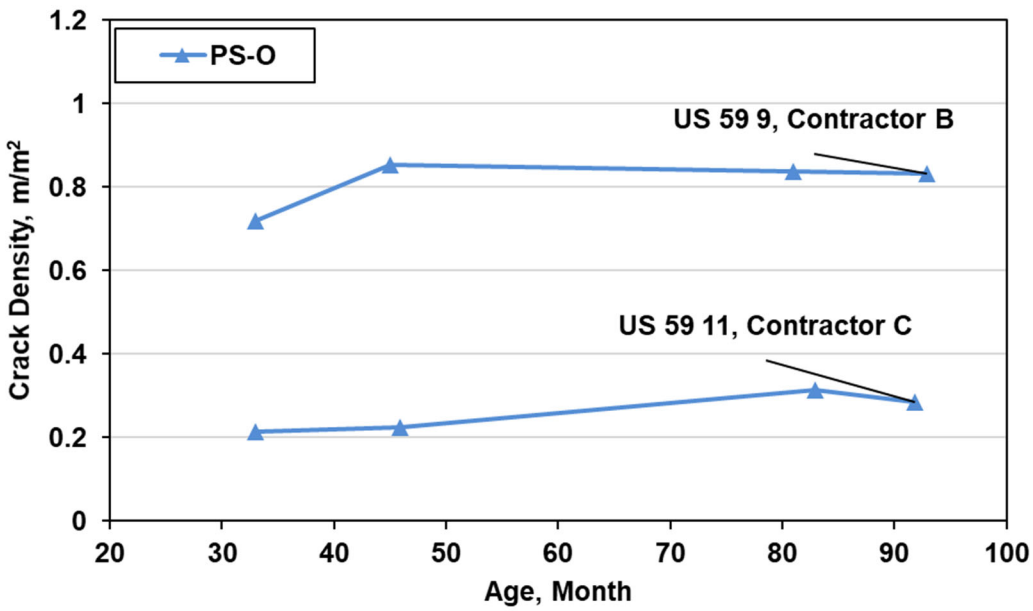


Figure 5.5—Crack density versus age for two overlay decks constructed by different contractors

Figure 5.6 compares crack density with age for the two monolithic fiber-reinforced decks constructed by Contractors B and C, US-59 10 and 12, respectively. These two decks have identical mixture proportions except for fiber type and dosage. The deck on US-59 10 contains 5 lb/yd^3 of macrofibers while the deck US-59 12 contains 3 lb/yd^3 of microfibers. The concrete for the deck US-59 10 had a lower slump (3 vs 4 in.) and a lower compressive strength (5100 psi vs 5740 psi) than the deck on US-59 12. The concrete for both have a paste content of 24.62% and air content of 7%. In spite of the similarities between these two decks, at an age of 88 months, the deck US-59 10 exhibits almost four times as much cracking as the deck on US-59 12 [0.388 (interpolated)

vs 0.104 m/m^2]. These findings, again, point to the direct influence of Contractor B on cracking of these two pairs of bridges. Since Contractors B and C did not construct any of the bridge types constructed by Contractor A, the performance of Contractor A cannot be compared with that of Contractors B and C. In light of the poor performance of Contractor B on the two decks in this comparison and on the single LC-HPC deck it constructed, the bridges constructed by Contractor B will not be considered in the balance of this chapter.

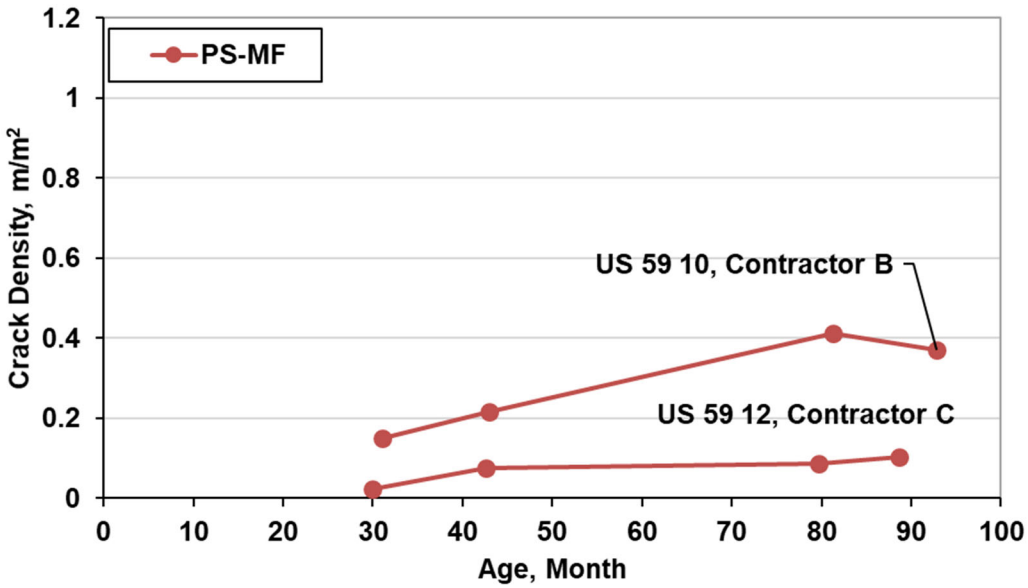


Figure 5.6— Crack density versus age for two monolithic FRC decks constructed by different contractors

5.6.2 Superstructure

This section evaluates the effect on deck cracking based superstructure type (prestressed or steel girders) on bridges with either overlay or monolithic decks constructed by contractors A and C. *Durability* (1970) and Hoppe et al. (2015) conclude that bridges supported by prestressed girders, in general, perform better than those supported by steel girders.

5.6.2.1 Bridge Decks with Overlays

Figure 5.7 shows the 96-month crack densities for bridges with overlay decks: US-59 11, constructed by Contractor C, supported by prestressed girders (PS-O), and US-59 5 and 6, constructed by Contractor A, supported by steel girders (Steel-OF and Steel-O). The bridge decks in the figure have identical mixture proportions in both subdeck and overlay, except for the binder composition in the overlay: binary for PS-O, and ternary for Steel-O and Steel-OF. As for the subdeck concrete properties, slump ranged from 4½ to 5 in., compressive strength from 4480 to 5010 psi, and air content from 6.25 to 7.75% and for the overlay, slump ranged from ¾ to 4½ in., compressive strength from 5470 to 7480 psi, and air content from 6.75 to 7.75%. Given the nearly identical paste contents for these decks, differences in cracking should not be the result of mixture or material properties (see Chapter 3) but may be affected by the contractor. On average, the two overlay bridge decks supported by steel girders (Steel-O and Steel-OF) and constructed by Contractor A have crack density of 0.486 m/m^2 compared to 0.285 m/m^2 for the deck supported by prestressed girders (PS-O) and constructed by Contractor C. These results support the expected, that is, bridge decks supported by prestressed girders exhibit less cracking than bridge decks supported by steel girders.

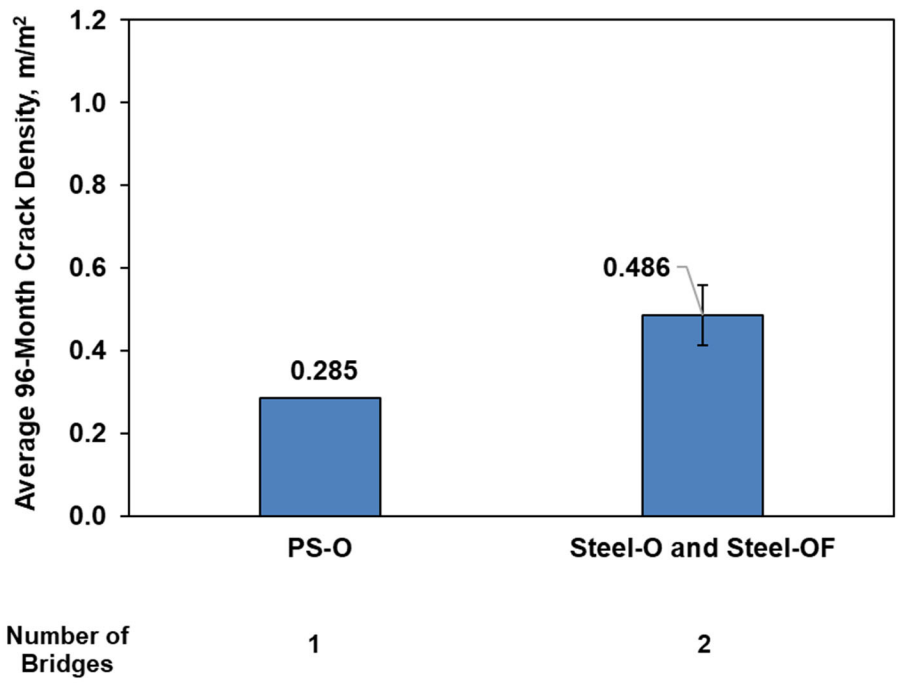


Figure 5.7—Average 96-month crack densities for the bridges with overlay decks supported by prestressed or steel girders. *Note: The Steel-OF has fibers in the overlay*

Both US-59 5 (Steel-OF) and US-59 6 (Steel-O) were constructed by Contractor A and have identical paste contents. Although US-59 5 (Steel-OF) contains fibers, it has greater cracking than US-59 6 (Steel-O) with no fibers (Steel-O), suggesting that the incorporation of fibers in an overlay is not enough, by itself, to improve the cracking performance.

5.6.2.2 Monolithic Bridge Decks

Figure 5.8 shows the 96-month crack densities for bridge decks with monolithic decks: US-59 12, constructed by Contractor C, supported by prestressed girders (PS-MF) and US-59 1 and 2, constructed by Contractor A, supported by steel girders (Steel-M). The concretes in the three bridge decks in the figure have a *w/cm* ratio of 0.42 and paste contents ranging from 23.99 to 24.60%. US-59 12 (PS-MF) has a 100% portland cement deck with 3 lb/yd^3 of F-6 fibers while US-59 1 and 2 (Steel-M) that have ternary (cement, slag, and silica fume) concrete decks. On

average, the two monolithic bridge decks supported by steel girders (Steel-M) and constructed by Contractor A have a crack density of 0.462 m/m^2 compared to 0.119 m/m^2 for the monolithic bridge supported by prestressed girders (PS-MF) constructed by Contractor C. There is not enough data to conclude if the lower cracking is caused because of the prestressed girders, incorporation of fibers, different binder compositions, or the contractor. The average 96-month crack density of the Steel-M decks, however, is higher by about 80% than observed on other comparable non-LC-HPC monolithic decks that, like the Steel-M decks, have paste contents less than or equal to 27.2% (see Chapter 3).

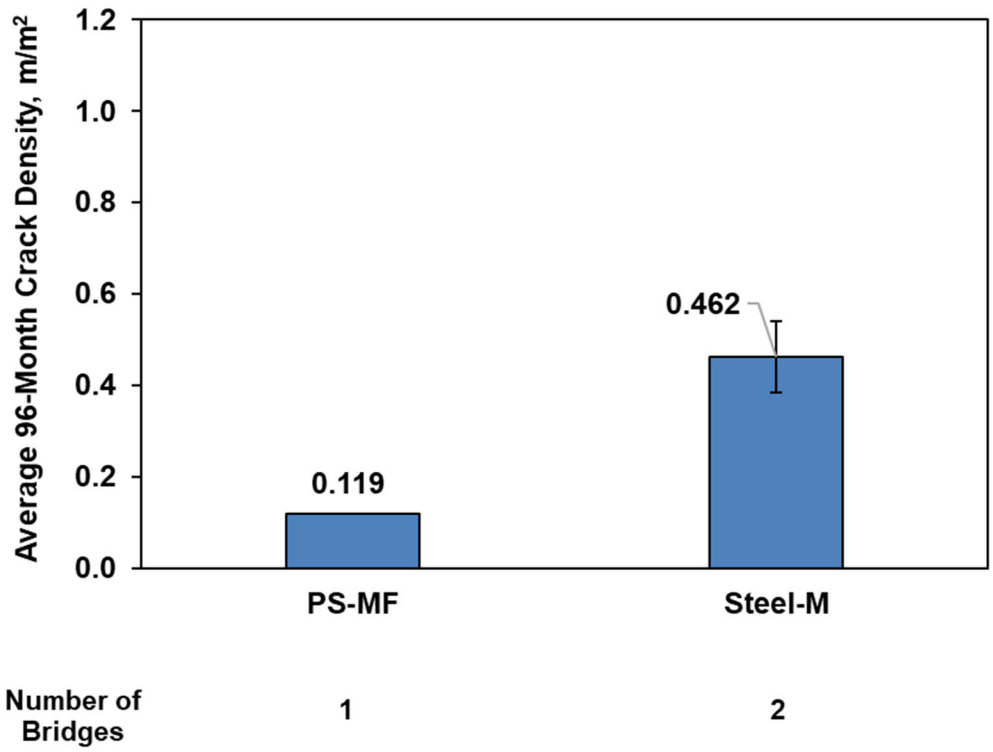


Figure 5.8—Average 96-month crack densities for the bridges with monolithic decks supported by prestressed or steel girders. *Note: The PS-MF bridge has fibers in its deck*

5.6.3 Deck Type

This section evaluates the effect of deck type on cracking for bridges constructed by Contractors A and C supported by either prestressed or steel girders. Lindquist et al. (2005 and 2006) showed that steel-girder bridges with monolithic decks perform better than those with overlay decks (7% silica fume overlays, 5% silica fume overlays, and high-density conventional overlays with 100% portland cement). Wan et al. (2010) found a significant amount of early-age cracking in newly constructed overlay decks in Wisconsin.

5.6.3.1 Decks supported by prestressed girders

Figure 5.9 shows the 96-month crack densities for the prestressed-girder bridges: US-59 3, 4, 7, and 8 with deck panels (PS-DP) constructed by Contractor A, US-59 12 with a monolithic deck containing fibers (PS-MF) constructed by Contractor C, and US-59 11 with a silica fume overlay deck (PS-O) constructed by Contractor C. The PS-DP decks have binary or ternary mixtures while PS-MF and the subdeck of PS-O bridges were cast 100% portland cement mixtures. All of the bridge decks in the figure have paste contents below 27.2% (PS-DP with 23.99 or 24.77% and PS-MF with 24.62%), except for the subdeck of PS-O that has a paste content of 27.95% (overlay paste content is 23.54%) and exhibits the greatest cracking. Although the figure suggests that of the bridge decks supported by prestressed girders, those with deck panels (PS-DP) and the monolithic bridge incorporating fibers (PS-MF) perform better than the overlay deck (PS-O). Differences in binder compositions, contractors, and paste contents, however, make it difficult to make a solid conclusion.

As discussed in Section 5.6.2.2, the two steel-girder bridges with monolithic decks constructed by Contractor A, Steel-M, exhibit significantly greater cracking than typical non-LC-HPC bridges with monolithic decks supported by steel girders in Kansas, suggesting that Contractor A may have performed poorly in construction of this type of bridge (steel-girder bridges

with monolithic decks). The PS-DP bridges, also constructed by Contractor A, however, exhibit, on average, not only the best performance among the bridges in this study but much better performance than similar bridges in other states (as will be discussed in Section 5.7). This may suggest that contractor performance and its effects on cracking may also be a function of the bridge type since different bridge types often require different construction practices.

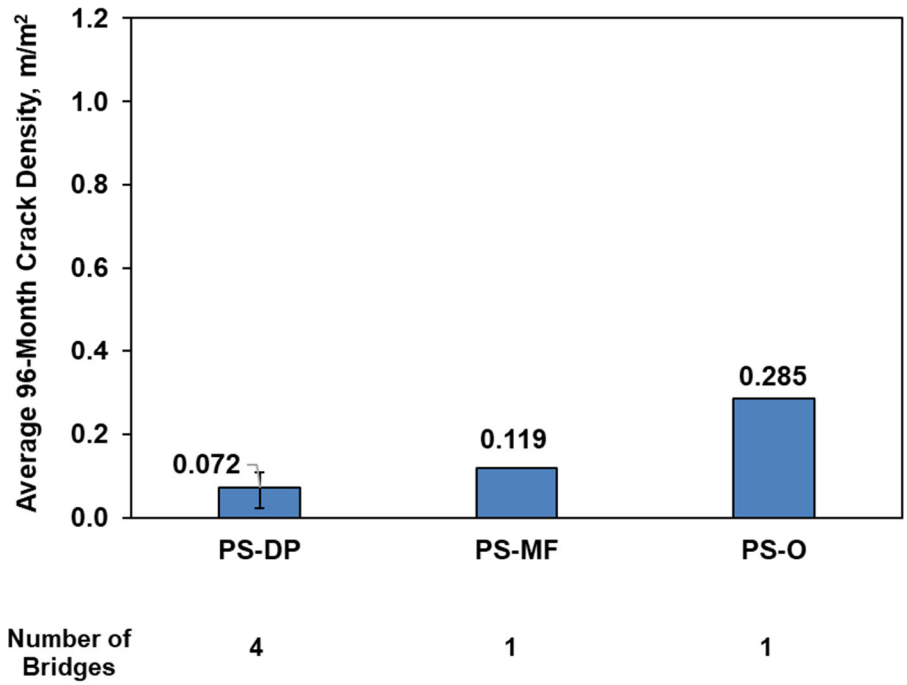


Figure 5.9—Average 96-month crack densities for different deck types supported by prestressed girders

5.6.3.2 Decks supported by steel girders

Figure 5.10 compares the crack densities of the steel-girder bridges: US-59 1 and 2 with monolithic decks (Steel-M), US-59 6 with a silica fume overlay deck (Steel-O), and US-59 5 with a silica fume overlay deck incorporating fibers (Steel-OF), all of which were constructed by Contractor A. The results show that the monolithic decks (Steel-M) have, on average, just slightly less cracking than the overlay decks (Steel-O, Steel-OF), 0.463 versus 0.486 m/m^2, respectively.

These two groups of bridge decks, however, have different concrete properties and binder compositions. The most obvious difference, that can affect cracking significantly, is the paste content in subdeck of US-59 5 and 6 (Steel-O and Steel-OF), 27.95%, a value exceeding the threshold of 27.2%, compared to 23.99% for US-59 1 and 2 (Steel-M), which may have been the reason overlay decks show greater cracking. The paste content of the overlay of US-59 5 (Steel-OF) and 6 (Steel-O) is 23.54%. As mentioned in Section 5.5.2.2, the crack density of the two monolithic decks averages about 80% greater than observed in similar low-paste content, non-LC-HPC monolithic decks in Kansas. If these results show anything, it is only that Contractor A had higher cracking than would be expected on the monolithic decks.

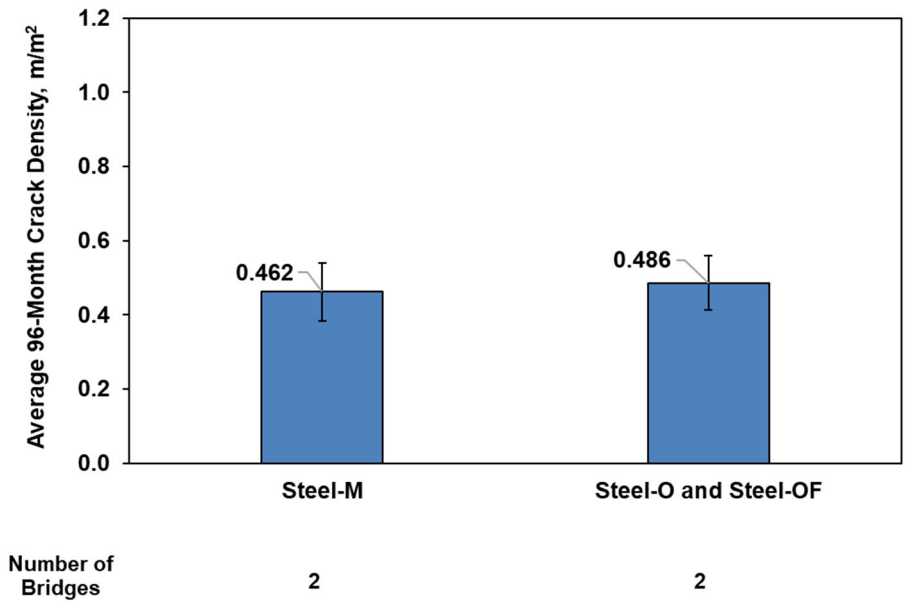


Figure 5.10—Average 96-month crack density for bridge decks supported by steel girders

5.6.4 Cementitious Material Type

Pendergrass et al. (2018) concluded that ternary concrete mixtures experience less drying shrinkage than binary mixtures. They showed that the addition of 3 or 6% silica fume to concrete with cement and slag reduced both early-age and long-term shrinkage. Among the 10 bridge decks

constructed by Contractors A and C, concrete with three cementitious material combinations were used: plain with 100% portland cement, binary with cement and slag, ternary with cement, slag, and silica fume. US-59 5, 6, 11, and 12 used limestone as coarse aggregate while the others that used granite.

A comparison based on the cementitious material type for some matching bridge decks is not possible: US-59 1 and 2 both have a ternary concrete (60% cement, 35% slag, 5% silica fume); US-59 5 and 6 have the same cementitious materials in both the subdecks (100% portland cement) and the overlays (66% cement, 30% slag, 3.9% silica fume). US-59 11 and 12 are bridges with different deck types; therefore, a comparison based on the cementitious material is not possible for these bridge decks either. The CIP concrete toppings on US-59 3 and 4, however, contain binary cementitious material (65% cement, 35% slag) while those on US-59 7 and 8 contain a ternary cementitious material (60% cement, 30% slag, 5% silica fume). A comparison is possible in this case since these four bridges have a similar deck and superstructure type (PS-DP) and all were constructed by Contractor A.

Figure 5.11 compares the average crack densities at 96 months for PS-DP bridges with a paste content of 24.77% and binary cementitious materials and those with a paste content of 23.99% and ternary cementitious materials. The decks with the ternary mixtures have an average 96-month crack density of 0.044 m/m^2 compared to 0.100 m/m^2 for the two decks with binary bridges, both of which are considered very good. The difference in crack density may also have been due to the small difference in paste contents.

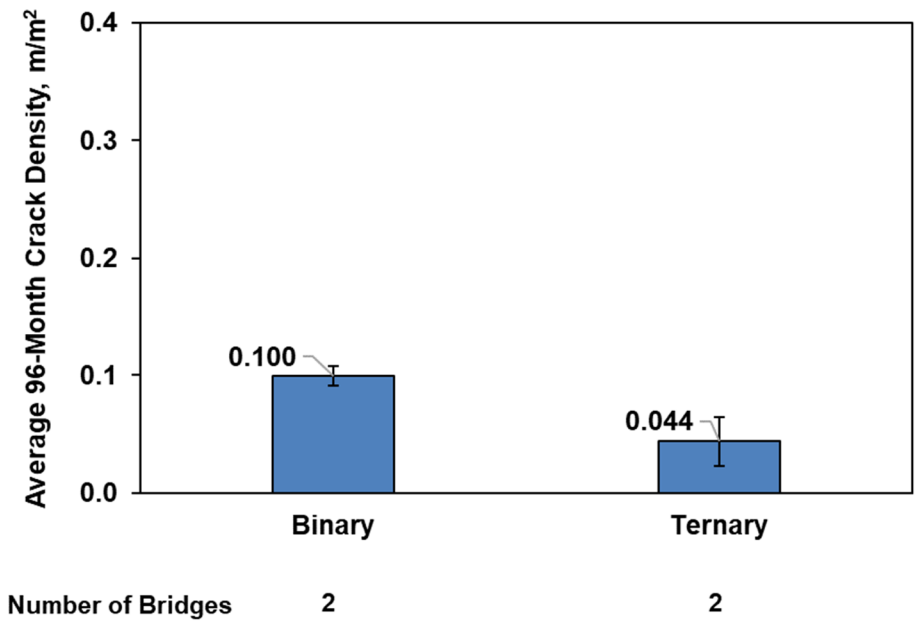


Figure 5.11—Average 96-month crack densities for the bridges supported by prestressed girders and having deck panels with binary or ternary CIP concrete toppings

5.7 PERFORMANCE OF PARTIAL-DEPTH DECK PANELS (PS-DP) COMPARED TO UTAH DECKS

One of the main findings of the analysis in Chapter 3 was the significant effect of paste content on cracking of monolithic decks supported by steel girders. The results in Chapter 3 showed that when the paste content exceeds 27.2%, cracking increases significantly, regardless of the influence of other factors. The cracking performance of US-59 deck panel bridges (PS-DP) can be used to illustrate this effect.

The excellent performance of PS-DP bridges in this study is in contrast with the performance of the similar decks in Missouri and Utah (Wenzlick 2005, Bitnoff 2014). Wenzlick (2005) found that prestressed-girder bridges with partial-depth precast deck panels in Missouri exhibited twice as much cracking as cast-in-place bridge decks in the state. Wenzlick (2005) did not report concrete properties. Bitnoff (2014) evaluated the cracking performance of four

prestressed-girder bridges with partial-depth precast concrete deck panels in Utah by conducting crack surveys for two years. Bitnoff reported crack densities ranging from 0.43 to 1.148 m/m^2 at 24 months, values that cannot be considered low at that age. Figure 5.12 compares the most recent cracking performance at 24 months for the four Utah decks and 96 months for the four PS-DP decks. Two PS-DP decks have a paste content of 23.99%, a *w/cm* ratio of 0.42, and a ternary (cement, slag, and silica fume) mixture and the other two have a paste content 24.77%, a *w/cm* ratio of 0.45, and a binary (cement and slag) mixture. The four Utah decks (UT IC or UT Control) have a paste content of 27.96%, a *w/cm* ratio of 0.44, and a binary (cement and fly ash) mixture. In addition, two of the Utah bridges are internally-cured (labeled as UT IC), a technology used for mitigating bridge deck cracking, by replacing a portion (16.7% by volume) of aggregates with pre-wetted lightweight aggregates. The results show that the PS-DP decks with an average crack density of 0.072 m/m^2 at the age of 96 months have significantly less cracking than the UT IC decks with an average crack density of 0.607 m/m^2 and UT Control decks with an average crack density of 0.831 m/m^2 at 24 months. This comparison strongly demonstrates the significant effect that paste content greater than the 27.2% can have on cracking of deck-panel bridges. It also shows that incorporation of internal curing, alone, does not guarantee a low-cracking performance if the concrete has paste content exceeding 27.2%.

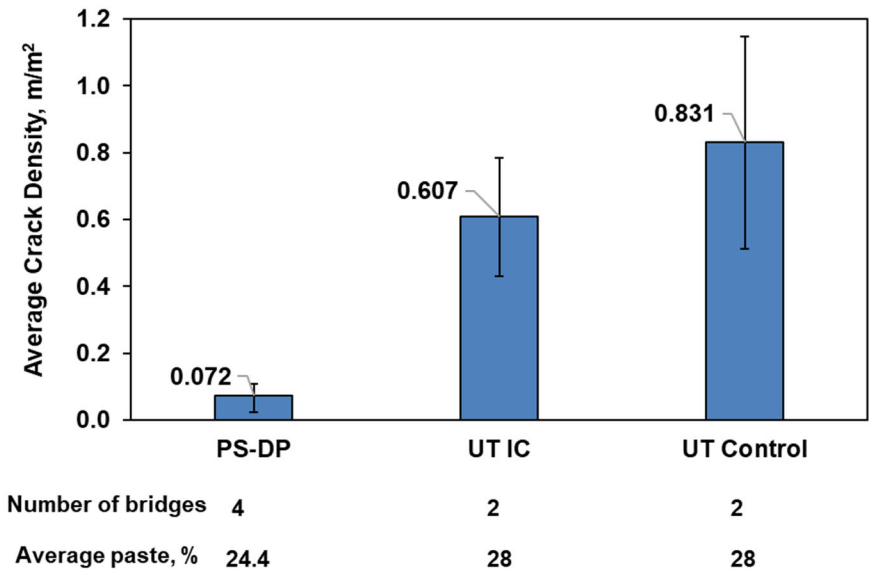


Figure 5.12—Latest crack densities for pre-stressed girder bridges with partial-depth deck panels in Kansas and Utah

Note: crack densities for Utah bridges (UT IC and UT Control) are at the age of 24 months versus that of 96 months for PS-DP bridges

5.8 CRACK WIDTH

With the exception of US-59 11, cracks widths were measured on the US-59 decks during the final crack surveys (summer 2016), as listed in Table 5.8. Table E.2 in Appendix E includes all measured crack widths.

A crack comparator was used for measuring crack width. Figure 5.13 compares crack density with average crack width for the 11 US-59 bridge decks. The average crack widths in individual decks range from 0.005 to 0.012 in. The results indicate that, in general, crack width increases with increasing crack density. Given the fact that the average crack widths are above 0.003 in., it can be assumed that the measured cracks would allow deicing agents to penetrate concrete (Rodriguez and Hooton 2003, Pease 2010).

Table 5.8—Measured crack width information

Bridge Number	Girder-Deck	Average Crack Width (10^{-3} in.)	# Width Measured
US-59 1	**Steel-M**	12.46	124
US-59 2	**Steel-M**	8	134
US-59 3	**PS-DP**	7.1	53
US-59 4	**PS - DP**	5.7	51
US-59 5	**Steel-OF**	11.1	93
US-59 6	**Steel-O**	7	151
US-59 7	**PS-DP**	5.2	36
US-59 8	**PS-DP**	5.4	52
US-59 9	**PS-O**	8.7	147
US-59 10	**PS-MF**	5.7	152
US-59 11	**PS-O**	Not Measured	Not Measured
US-59 12	**PS-MF**	7.4	36

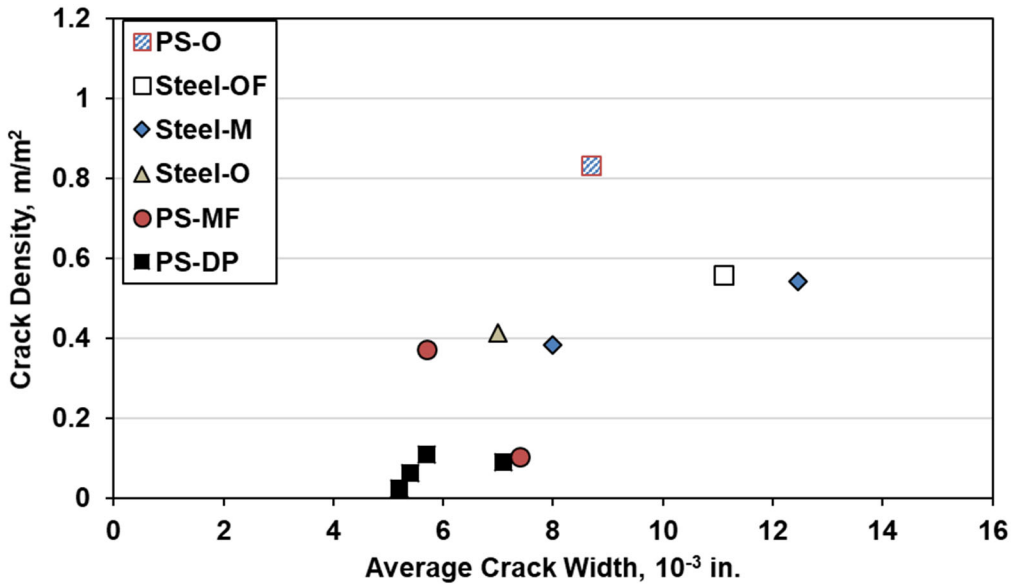


Figure 5.13— Crack density versus average crack width for eleven out of twelve US-59 bridges

5.9 SUMMARY AND CONCLUSIONS

This chapter evaluates the effects of superstructure type (prestressed or steel girders), deck type (monolithic, precast panels, or silica fume overlays), and concrete type (ternary, binary, or fiber-reinforced), on cracking of 12 bridge decks on highway US-59 south of Lawrence, Kansas.

The 12 bridge decks include four supported by steel girders and eight supported by prestressed concrete girders. Among the four steel girder bridges, two had monolithic decks (Steel-M), one had a silica fume overlay deck (Steel-O), and one had a silica fume overlay deck incorporating synthetic fibers (Steel-OF). Among the prestressed concrete bridges, two had monolithic decks incorporating synthetic fibers (Steel-MF), two had silica fume overlay decks (PS-O), and four had precast partial-depth concrete deck panels (PS-DP). Three contractors (A, B, and C) constructed these 12 bridge decks.

The following conclusions are based on the analysis presented in this chapter.

1. Two bridge decks constructed by Contractor B experienced significantly greater cracking than two similar bridge decks constructed by Contractor C, pointing to the direct influence of the contractor on cracking of these two pairs of bridges.
2. Among the bridge decks with overlays those supported by steel girders showed greater cracking than the bridge deck supported by prestressed girders.
3. In the only comparable case, incorporation of fibers did not reduce cracking in an overlay.
4. The bridge decks with precast partial-depth concrete deck panels with cast-in-place concrete toppings showed significantly lower cracking than the other bridge deck types evaluated in this study. These bridge decks also had significantly lower cracking than similar bridge decks constructed in the state of Utah, likely due to the use of lower paste contents compared to that used in the Utah bridge decks.
5. Incorporation of internal curing in concrete with paste content exceeding 27.2% does not improve cracking performance.
6. The average of the measured crack widths ranged from 0.005 to 0.012 in. with greater average crack widths associated with greater crack densities.

CHAPTER 6 - SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 SUMMARY

Laboratory studies incorporating crack-reducing technologies with those included in specifications for low-cracking high-performance concrete (LC-HPC) bridge decks and analyses of data from crack surveys and construction observations on more than 50 concrete decks are described.

The laboratory investigations include the combination of the supplementary cementitious materials (SCMs) slag cement and silica fume as partial replacements for portland cement, internal curing through use of pre-wetted lightweight aggregates (IC), and calcium oxide-based (CaO) or magnesium oxide-based (MgO) shrinkage-compensating admixtures (SCAs) or expansive additives with LC-HPC. A modified version of ASTM C157 in which length-change measurements begin 5½ ± ½ hr after casting concrete is developed and used to evaluate swelling and shrinkage.

The results of crack surveys on LC-HPC bridge decks, old monolithic decks, and a number of other bridge decks are combined with construction observations to improve the level of understanding of the principal factors affecting cracking, evaluate the effects of construction practices, including the effect of individual contractors, on cracking, and assess the effectiveness of crack-reducing technologies, such as synthetic fibers and IC.

6.2 CONCLUSIONS

The following conclusions are based on the results and analyses presented in this report.

6.2.1 Laboratory Evaluations

6.2.1.1 Combined effects of internal curing (IC), supplementary cementitious materials (SCMs), and shrinkage-compensating admixtures (SCAs) on concrete shrinkage

1. The modified ASTM C157 method developed in this study helps to capture the early-age behavior of concrete mixtures.
2. IC provided by partial replacement of total aggregate with intermediate-size pre-wetted LWA is effective in reducing 0 to 20-day drying shrinkage in concrete made with moderate *w/cm*. In every pair of mixtures with and without IC, the mixture with IC exhibited greater early-age expansion and less shrinkage during the first 20 days of drying. The opposite was observed in drying shrinkage between 20 and 180 days, during which mixtures with IC experienced greater shrinkage than their comparable pairs with no IC. Overall, IC reduced shrinkage.
3. Partial replacements of portland cement with slag cement and silica fume increase first-day expansion and reduce shrinkage. A further increase in first-day expansion and a reduction in shrinkage is obtained when internal curing is used in conjunction with slag cement and silica fume.
4. The incorporation of the CaO- or the MgO-based SCAs results in expansion during the curing period and a reduction in developing shrinkage strain. The CaO-based SCA induces a more rapid expansion of greater magnitude, with most of the expansion taking place within the first day after casting while the MgO-based SCA causes an expansion that steadily increases throughout the curing period and is lower in magnitude. The MgO-based SCA also contributes to lower drying shrinkage because

it incorporates a shrinkage-reducing admixture.

5. When the CaO-based SCA is incorporated in a mixture containing SCMs or SCMs and IC, a larger expansion and, consequently, a lower overall shrinkage strain is observed. The increase in expansion is greater than observed for CaO-based SCA alone or the CaO-based SCA used in conjunction with pre-wetted LWA. The same observations cannot be made for mixtures incorporating SCMs with the MgO-based SCA.

6.2.2 Field Evaluations

6.2.2.1 Principal factors affecting cracking in concrete bridge decks

1. Bridge deck cracking increases with age.
2. Paste content has a dominant effect on cracking. Paste contents above 27.2% by volume are associated with high crack densities in bridge decks, and increases in paste above 27.2% are associated with progressively greater crack densities.
3. Paste contents below 26.4% are associated with low crack densities in bridge decks, but reductions below 26.4% do not appear to result in substantial reductions in crack density.
4. Bridge decks placed on days with greater variations of temperature exhibit significantly greater cracking than those placed on days with less variation, independent of the other factors.
5. Variations in slump (ranging from 1 ½ to 4 in.), compressive strength (ranging from 3790 psi to 7430 psi), and air content (ranging from 4.5 to 9.5%) have much less effect in cracking.

6. If concrete temperatures are maintained within a range of between approximately 55° and 70°F and early age curing is provided, bridge deck cracking decreases with increasing high air temperature on the day of casting.
7. Bridge decks placed and finished between midnight and noon exhibit lower cracking than those placed and finished between early morning and night.

6.2.2.2 Construction practices and bridge deck cracking: examples of bridge decks with low-shrinkage concrete exhibiting high cracking

1. Bridge decks with similar material properties, mixture proportions, and structural properties can exhibit different cracking behavior that can be tied to construction practices.
2. The negative effects of construction practices can neutralize the positive effects of a *low-shrinkage* concrete and other crack-reducing technologies, such as fibers.
3. Insufficient consolidation, loss of consolidation caused by workers walking in consolidated concrete, and over-finishing can significantly increase cracking in bridge decks.

6.2.2.3 Evaluation of cracking in bridges with different superstructure and deck types

1. For bridge decks with overlays, those supported by steel girders exhibit greater cracking than those supported by prestressed girders.
2. The bridge decks with precast partial-depth concrete deck panels with cast-in-place concrete toppings showed significantly lower cracking than the other bridge deck types evaluated in this study. These bridge decks also had significantly lower cracking than similar bridge decks constructed in the state of Utah, likely due to the use of lower paste contents compared to that used in the Utah bridge decks.

3. Incorporation of a crack-reducing technology such as IC in the decks cast with concrete having paste contents exceeding 27.2% cannot overcome the negative effects of the greater paste content on cracking.
4. Greater average crack widths appear to correspond with greater crack densities.

6.3 RECOMMENDATIONS

1. Concrete that is pumpable, workable, and easy to consolidate with the lowest possible paste content, not exceeding 27.2%, should be used to minimize shrinkage-induced cracking.
2. Paste contents below 27.2% should be used regardless of incorporation of additional crack-reducing technologies such as synthetic fibers and internal curing or different binder composition in the concrete deck.
3. Concrete should be finished minimally to limit the paste content on the deck surface. This will result in lower drying shrinkage at the surface and allow curing to be initiated as soon as possible. Bridge decks do not need shiny surfaces.
4. Concrete should be thoroughly consolidated. Increasing slump beyond the specified limit should be allowed at the discretion of the material engineer if the higher slump (through use of appropriate water reducing admixtures/superplasticizers) can improve the consolidation. Slump should not be increased by adding additional water.
5. Specifications should prohibit workers from walking in concrete after it is consolidated. The latter will be aided if the specifications also require vibrators and finishing equipment be placed as close as possible to prevent workers from walking between these pieces of equipment. If the initially consolidated concrete is disturbed for any reason, it must be fully reconsolidated.

6. To limit the effects of high temperature and thermally-induced cracking, placement and finishing of the concrete deck is best on days and during seasons when less temperature variations are anticipated. Best results are obtained when the placement and finishing take place between midnight and noon rather than between early morning and night.
7. Incorporation of SCMs, IC, and SCAs in the concrete mixture, alone or in combination, should be considered to reduce cracking. To successfully reduce cracking, however, as a minimum, the first five recommendations should be followed.

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APPENDIX A: LOW-CRACKING HIGH-PERFORMANCE CONCRETE (LC-HPC) SPECIFICATIONS – AGGREGATES, CONCRETE, AND CONSTRUCTION

KANSAS DEPARTMENT OF TRANSPORTATION
SPECIAL PROVISION TO THE
STANDARD SPECIFICATIONS, 2007 EDITION

Add a new SECTION to DIVISION 1100:

LOW-CRACKING HIGH-PERFORMANCE CONCRETE – AGGREGATES

1.0 DESCRIPTION

This specification is for coarse aggregates, fine aggregates, and mixed aggregates (both coarse and fine material) for use in bridge deck construction.

2.0 REQUIREMENTS

a. Coarse Aggregates for Concrete.

(1) Composition. Provide coarse aggregate that is crushed or uncrushed gravel, chat, or crushed stone. (Consider calcite cemented sandstone, rhyolite, basalt and granite as crushed stone

(2) Quality. The quality requirements for coarse aggregate for bridge decks are in **TABLE 1-1**:

TABLE 1-1: QUALITY REQUIREMENTS FOR COARSE AGGREGATES FOR BRIDGE DECK

Concrete Classification	Soundness (min.)	Wear (max.)	Absorption (max.)	Acid Insol. (min.)
Grade 3.5 (AE) (LC-HPC) [1]	0.90	40	0.7	55

[1] Grade 3.5 (AE) (LC-HPC) – Bridge Deck concrete with select coarse aggregate for wear and acid insolubility.

(3) Product Control.

(a) Deleterious Substances. Maximum allowed deleterious substances by weight are:

- Material passing the No. 200 sieve (KT-2)..... 2.5%
- Shale or Shale-like material (KT-8)..... 0.5%
- Clay lumps and friable particles (KT-7) 1.0%
- Sticks (wet) (KT-35)..... 0.1%
- Coal (AASHTO T 113)..... 0.5%

(b) Uniformity of Supply. Designate or determine the fineness modulus (grading factor) according to the procedure listed in the Construction Manual Part V, Section 17 before delivery, or from the first 10 samples tested and accepted. Provide aggregate that is within ±0.20 of the average fineness modulus.

(4) Do not combine siliceous fine aggregate with siliceous coarse aggregate if neither meet the requirements of **subsection 2.0c.(2)(a)**. Consider such fine material, regardless of proportioning, as a Basic Aggregate that must conform to **subsection 2.0c**.

(5) Handling Coarse Aggregates.

(a) Segregation. Before acceptance testing, remix all aggregate segregated by transportation or stockpiling operations.

(b) Stockpiling.

- Stockpile accepted aggregates in layers 3 to 5 feet thick. Berm each layer so that aggregates do not "cone" down into lower layers.
- Keep aggregates from different sources, with different gradings, or with a significantly different specific gravity separated.
- Transport aggregate in a manner that insures uniform gradation.
- Do not use aggregates that have become mixed with earth or foreign material.

- Stockpile or bin all washed aggregate produced or handled by hydraulic methods for 12 hours (minimum) before batching. Rail shipment exceeding 12 hours is acceptable for binning provided the car bodies permit free drainage.
- Provide additional stockpiling or binning in cases of high or non-uniform moisture.

b. Fine Aggregates for Basic Aggregate in MA for Concrete.

(1) Composition.

(a) Type FA-A. Provide either singly or in combination natural occurring sand resulting from the disintegration of siliceous or calcareous rock, or manufactured sand produced by crushing predominately siliceous materials.

(b) Type FA-B. Provide fine granular particles resulting from the crushing of zinc and lead ores (Chat).

(2) Quality.

(a) Mortar strength and Organic Impurities. If the District Materials Engineer determines it is necessary, because of unknown characteristics of new sources or changes in existing sources, provide fine aggregates that comply with these requirements:

- Mortar Strength (Mortar Strength Test, KTMR-26). Compressive strength when combined with Type III (high early strength) cement:
  - At age 24 hours, minimum……..100%*
  - At age 72 hours, minimum……..100%*

  *Compared to strengths of specimens of the same proportions, consistency, cement and standard 20-30 Ottawa sand.
- Organic Impurities (Organic Impurities in Fine Aggregate for Concrete Test, AASHTO T 21). The color of the supernatant liquid is equal to or lighter than the reference standard solution.

(b) Hardening characteristics. Specimens made of a mixture of 3 parts FA-B and 1 part cement with sufficient water for molding will harden within 24 hours. There is no hardening requirement for FA-A.

(3) Product Control.

(a) Deleterious Substances.

- Type FA-A: Maximum allowed deleterious substances by weight are:
  - Material passing the No. 200 sieve (KT-2)……..……. 2.0%
  - Shale or Shale-like material (KT-8) ……. 0.5%
  - Clay lumps and friable particles (KT-7)……..……. 1.0%
  - Sticks (wet) (KT-35)……...……..…… 0.1%
- Type FA-B: Provide materials that are free of organic impurities, sulfates, carbonates, or alkali. Maximum allowed deleterious substances by weight are:
  - Material passing the No. 200 sieve (KT-2)…….……..... 2.0%
  - Clay lumps & friable particles (KT-7)……. 0.25%

(c) Uniformity of Supply. Designate or determine the fineness modulus (grading factor) according to the procedure listed in the Construction Manual Part V, Section 17 before delivery, or from the first 10 samples tested and accepted. Provide aggregate that is within ±0.20 of the average fineness modulus.

(4) Proportioning of Coarse and Fine Aggregate. Use a proven optimization method such as the Shilstone Method or the KU Mix Method.

Do not combine siliceous fine aggregate with siliceous coarse aggregate if neither meet the requirements of **subsection 2.0c.(2)(a)**. Consider such fine material, regardless of proportioning, as a Basic Aggregate and must conform to the requirements in **subsection 2.0c**.

(5) Handling and Stockpiling Fine Aggregates.

- Keep aggregates from different sources, with different gradings or with a significantly different specific gravity separated.
- Transport aggregate in a manner that insures uniform grading.
- Do not use aggregates that have become mixed with earth or foreign material.

- Stockpile or bin all washed aggregate produced or handled by hydraulic methods for 12 hours (minimum) before batching. Rail shipment exceeding 12 hours is acceptable for binning provided the car bodies permit free drainage.
- Provide additional stockpiling or binning in cases of high or non-uniform moisture.

c. Mixed Aggregates for Concrete.

(1) Composition.

(a) Total Mixed Aggregate (TMA). A natural occurring, predominately siliceous aggregate from a single source that meets the Wetting & Drying Test (KTMR-23) and grading requirements.

(b) Mixed Aggregate. A combination of basic and coarse aggregates that meet **TABLE 1-2**.

- Basic Aggregate (BA). Singly or in combination, a natural occurring, predominately siliceous aggregate that does not meet the grading requirements of Total Mixed Aggregate.

(c) Coarse Aggregate. Granite, crushed sandstone, chat, and gravel. Gravel that is not approved under **subsection 2.0c.(2)** may be used, but only with basic aggregate that meets the wetting and drying requirements of TMA.

(2) Quality.

(a) Total Mixed Aggregate.

- Soundness, minimum (KTMR-21) …….……0.90
- Wear, maximum (KTMR-25) …….……50%
- Wetting and Drying Test (KTMR-23) for Total Mixed Aggregate
  Concrete Modulus of Rupture:
  - At 60 days, minimum…….550 psi
  - At 365 days, minimum…..…….……550 psi

  Expansion:
  - At 180 days, maximum…….…….0.050%
  - At 365 days, maximum…….…….0.070%
  - Aggregates produced from the following general areas are exempt from the Wetting and Drying Test:
  - Blue River Drainage Area.
  - The Arkansas River from Sterling, west to the Colorado state line.
  - The Neosho River from Emporia to the Oklahoma state line.

(b) Basic Aggregate.

- Retain 10% or more of the BA on the No. 8 sieve before adding the Coarse Aggregate. Aggregate with less than 10% retained on the No. 8 sieve is to be considered a Fine Aggregate described in **subsection 2.0b**. Provide material with less than 5% calcareous material retained on the ⅜" sieve.
- Soundness, minimum (KTMR-21)…….0.90
- Wear, maximum (KTMR-25)…….……50%
- Mortar strength and Organic Impurities. If the District Materials Engineer determines it is necessary, because of unknown characteristics of new sources or changes in existing sources, provide mixed aggregates that comply with these requirements:
  - Mortar Strength (Mortar Strength Test, KTMR-26). Compressive strength when combined with Type III (high early strength) cement:
    - At age 24 hours, minimum……..100%*
    - At age 72 hours, minimum……..100%*
      *Compared to strengths of specimens of the same proportions, consistency, cement and standard 20-30 Ottawa sand.
  - Organic Impurities (Organic Impurities in Fine Aggregate for Concrete Test, AASHTO T 21). The color of the supernatant liquid is equal to or lighter than the reference standard solution.

(3) Product Control.

(a) Size Requirement. Provide mixed aggregates that comply with the grading requirements in **TABLE 1-2**.

TABLE 1-2: GRADING REQUIREMENTS FOR MIXED AGGREGATES FOR CONCRETE BRIDGE DECKS

Type	Usage	Percent Retained on Individual Sieves - Square Mesh Sieves										
		1½"	1"	3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100
MA-4	Optimized for LC-HPC Bridge Decks*	0	2-6	5-18	8-18	8-18	8-18	8-18	8-18	8-15	5-15	0-5

*Use a proven optimization method, such as the Shilstone Method or the KU Mix Method.
Note: Manufactured sands used to obtain optimum gradations have caused difficulties in pumping, placing or finishing. Natural coarse sands and pea gravels used to obtain optimum gradations have worked well in concretes that were pumped.

(b) Deleterious Substances. Maximum allowed deleterious substances by weight are:

- Material passing the No. 200 sieve (KT-2)……..….. 2.5%
- Shale or Shale-like material (KT-8)……..……. 0.5%
- Clay lumps and friable particles (KT-7)…… 1.0%
- Sticks (wet) (KT-35)……..…… 0.1%
- Coal (AASHTO T 113)…..……..……. 0.5%

(c) Uniformity of Supply. Designate or determine the fineness modulus (grading factor) according to the procedure listed in the Construction Manual Part V, Section 17 before delivery, or from the first 10 samples tested and accepted. Provide aggregate that is within ±0.20 of the average fineness modulus.

(4) Handling Mixed Aggregates.

(a) Segregation. Before acceptance testing, remix all aggregate segregated by transit or stockpiling.

(b) Stockpiling.

- Keep aggregates from different sources, with different gradings or with a significantly different specific gravity separated.
- Transport aggregate in a manner that insures uniform grading.
- Do not use aggregates that have become mixed with earth or foreign material.
- Stockpile or bin all washed aggregate produced or handled by hydraulic methods for 12 hours (minimum) before batching. Rail shipment exceeding 12 hours is acceptable for binning provided the car bodies permit free drainage.
- Provide additional stockpiling or binning in cases of high or non-uniform moisture.

3.0 TEST METHODS

Test aggregates according to the applicable provisions of **SECTION 1117**.

4.0 PREQUALIFICATION

Aggregates for concrete must be prequalified according to **subsection 1101.2**.

5.0 BASIS OF ACCEPTANCE

The Engineer will accept aggregates for concrete base on the prequalification required by this specification, and **subsection 1101.4**.

KANSAS DEPARTMENT OF TRANSPORTATION
SPECIAL PROVISION TO THE
STANDARD SPECIFICATIONS 2007 EDITION

Add a new SECTION to DIVISION 400:

LOW-CRACKING HIGH-PERFORMANCE CONCRETE

1.0 DESCRIPTION

Provide the grades of low-cracking high-performance concrete (LC-HPC) specified in the Contract Documents.

2.0 MATERIALS

Coarse, Fine & Mixed Aggregate..... **07-PS0165, latest version**
Admixtures..... **DIVISION 1400**
Cement ..... **DIVISION 2000**
Water ..... **DIVISION 2400**

3.0 CONCRETE MIX DESIGN

a. General. Design the concrete mixes specified in the Contract Documents.

Provide aggregate gradations that comply with **07-PS0165, latest version** and Contract Documents.

If desired, contact the DME for available information to help determine approximate proportions to produce concrete having the required characteristics on the project.

Take full responsibility for the actual proportions of the concrete mix, even if the Engineer assists in the design of the concrete mix.

Submit all concrete mix designs to the Engineer for review and approval. Submit completed volumetric mix designs on KDOT Form No. 694 (or other forms approved by the DME).

Do not place any concrete on the project until the Engineer approves the concrete mix designs. Once the Engineer approves the concrete mix design, do not make changes without the Engineer's approval.

Design concrete mixes that comply with these requirements:

b. Air-Entrained Concrete for Bridge Decks. Design air-entrained concrete for structures according to **TABLE 1-1**.

TABLE 1-1: AIR ENTRAINED CONCRETE FOR BRIDGE DECKS

Grade of Concrete Type of Aggregate (SECTION 1100)	lb of Cementitious per cu yd of Concrete, min/max	lb of Water per lb of Cementitious*	Designated Air Content Percent by Volume**	Specified 28-day Compressive Strength Range, psi
Grade 3.5 (AE) (LC-HPC)				
MA-4	500 / 540	0.44 – 0.45	8.0 ± 1.0	3500 – 5500

*Limits of lb. of water per lb. of cementitious. Includes free water in aggregates, but excludes water of absorption of the aggregates. With approval of the Engineer, may be decreased to 0.43 on-site.

**Concrete with an air content less than 6.5% or greater than 9.5% shall be rejected. The Engineer will sample concrete for tests at the discharge end of the conveyor, bucket or if pumped, the piping.

c. Portland Cement. Select the type of portland cement specified in the Contract Documents. Mineral admixtures are prohibited for Grade 3.5 (AE) (LC-HPC) concrete.

d. Design Air Content. Use the middle of the specified air content range for the design of air-entrained concrete.

e. Admixtures for Air-Entrainment and Water Reduction. Verify that the admixtures used are compatible and will work as intended without detrimental effects. Use the dosages recommended by the admixture manufacturers to determine the quantity of each admixture for the concrete mix design. Incorporate and mix the admixtures into the concrete mixtures according to the manufacturer's recommendations.

Set retarding or accelerating admixtures are prohibited for use in Grade 3.5 (AE) (LC-HPC) concrete. These include Type B, C, D, E, and G chemical admixtures as defined by ASTM C 494/C 494M – 08. Do not use admixtures containing chloride ion (CL) in excess of 0.1 percent by mass of the admixture in Grade 3.5 (AE) (LC-HPC) concrete.

(1) Air-Entraining Admixture. If specified, use an air-entraining admixture in the concrete mixture. If another admixture is added to an air-entrained concrete mixture, determine if it is necessary to adjust the air-entraining admixture dosage to maintain the specified air content. Use only a vinsol resin or tall oil based air-entraining admixture.

(2) Water-Reducing Admixture. Use a Type A water reducer or a dual rated Type A water reducer – Type F high-range water reducer, when necessary to obtain compliance with the specified fresh and hardened concrete properties.

Include a batching sequence in the concrete mix design. Consider the location of the concrete plant in relation to the job site, and identify the approximate quantity, when and at what location the water-reducing admixture is added to the concrete mixture.

The manufacturer may recommend mixing revolutions beyond the limits specified in **subsection 5.0**. If necessary and with the approval of the Engineer, address the additional mixing revolutions (the Engineer will allow up to 60 additional revolutions) in the concrete mix design.

Slump control may be accomplished in the field only by redosing with a water-reducing admixture. If time and temperature limits are not exceeded, and if at least 30 mixing revolutions remain, the Engineer will allow redosing with up to 50% of the original dose.

(3) Adjust the mix designs during the course of the work when necessary to achieve compliance with the specified fresh and hardened concrete properties. Only permit such modifications after trial batches to demonstrate that the adjusted mix design will result in concrete that complies with the specified concrete properties.

The Engineer will allow adjustments to the dose rate of air entraining and water-reducing chemical admixtures to compensate for environmental changes during placement without a new concrete mix design or qualification batch.

f. Designated Slump. Designate a slump for each concrete mix design within the limits in **TABLE 1-2**.

TABLE 1-2: DESIGNATED SLUMP*

Type of Work	Designated Slump (inches)
Grade 3.5 (AE) (LC-HPC)	1 ½ - 3

* The Engineer will obtain sample concrete at the discharge end of the conveyor, bucket or if pumped, the piping.

If potential problems are apparent at the discharge of any truck, and the concrete is tested at the truck discharge (according to **subsection 6.0**), the Engineer will reject concrete with a slump greater than 3 ½ inches at the truck discharge, 3 inches if being placed by a bucket.

4.0 REQUIREMENTS FOR COMBINED MATERIALS

a. Measurements for Proportioning Materials.

(1) Cement. Measure cement as packed by the manufacturer. A sack of cement is considered as 0.04 cubic yards weighing 94 pounds net. Measure bulk cement by weight. In either case, the measurement must be accurate to within 0.5% throughout the range of use.

(2) Water. Measure the mixing water by weight or volume. In either case, the measurement must be accurate to within 1% throughout the range of use.

(3) Aggregates. Measure the aggregates by weight. The measurement must be accurate to within 0.5% throughout the range of use.

(4) Admixtures. Measure liquid admixtures by weight or volume. If liquid admixtures are used in small quantities in proportion to the cement as in the case of air-entraining agents, use readily adjustable mechanical

dispensing equipment capable of being set to deliver the required quantity and to cut off the flow automatically when this quantity is discharged. The measurement must be accurate to within 3% of the quantity required.

b. Testing of Aggregates. Testing Aggregates at the Batch Site. Provide the Engineer with reasonable facilities at the batch site for obtaining samples of the aggregates. Provide adequate and safe laboratory facilities at the batch site allowing the Engineer to test the aggregates for compliance with the specified requirements.

KDOT will sample and test aggregates from each source to determine their compliance with specifications. Do not batch the concrete mixture until the Engineer has determined that the aggregates comply with the specifications. KDOT will conduct sampling at the batching site, and test samples according to the Sampling and Testing Frequency Chart in Part V. For QC/QA Contracts, establish testing intervals within the specified minimum frequency.

After initial testing is complete and the Engineer has determined that the aggregate process control is satisfactory, use the aggregates concurrently with sampling and testing as long as tests indicate compliance with specifications. When batching, sample the aggregates as near the point of batching as feasible. Sample from the stream as the storage bins or weigh hoppers are loaded. If samples cannot be taken from the stream, take them from approved stockpiles, or use a template and sample from the conveyor belt. If test results indicate an aggregate does not comply with specifications, cease concrete production using that aggregate. Unless a tested and approved stockpile for that aggregate is available at the batch plant, do not use any additional aggregate from that source and specified grading until subsequent sampling and testing of that aggregate indicate compliance with specifications. When tests are completed and the Engineer is satisfied that process control is again adequate, production of concrete using aggregates tested concurrently with production may resume.

c. Handling of Materials.

(1) Aggregate Stockpiles. Approved stockpiles are permitted only at the batch plant and only for small concrete placements or for the purpose of maintaining concrete production. Mark the approved stockpile with an "Approved Materials" sign. Provide a suitable stockpile area at the batch plant so that aggregates are stored without detrimental segregation or contamination. At the plant, limit stockpiles of tested and approved coarse aggregate and fine aggregate to 250 tons each, unless approved for more by the Engineer. If mixed aggregate is used, limit the approved stockpile to 500 tons, the size of each being proportional to the amount of each aggregate to be used in the mix.

Load aggregates into the mixer so no material foreign to the concrete or material capable of changing the desired proportions is included. When 2 or more sizes or types of coarse or fine aggregates are used on the same project, only 1 size or type of each aggregate may be used for any one continuous concrete placement.

(2) Segregation. Do not use segregated aggregates. Previously segregated materials may be thoroughly re-mixed and used when representative samples taken anywhere in the stockpile indicated a uniform gradation exists.

(3) Cement. Protect cement in storage or stockpiled on the site from any damage by climatic conditions which would change the characteristics or usability of the material.

(4) Moisture. Provide aggregate with a moisture content of ± 0.5% from the average of that day. If the moisture content in the aggregate varies by more than the above tolerance, take whatever corrective measures are necessary to bring the moisture to a constant and uniform consistency before placing concrete. This may be accomplished by handling or manipulating the stockpiles to reduce the moisture content, or by adding moisture to the stockpiles in a manner producing uniform moisture content through all portions of the stockpile.

For plants equipped with an approved accurate moisture-determining device capable of determining the free moisture in the aggregates, and provisions made for batch to batch correction of the amount of water and the weight of aggregates added, the requirements relative to manipulating the stockpiles for moisture control will be waived. Any procedure used will not relieve the producer of the responsibility for delivery of concrete meeting the specified water-cement ratio and slump requirements.

Do not use aggregate in the form of frozen lumps in the manufacture of concrete.

(5) Separation of Materials in Tested and Approved Stockpiles. Only use KDOT Approved Materials. Provide separate means for storing materials approved by KDOT. If the producer elects to use KDOT Approved Materials for non-KDOT work, during the progress of a project requiring KDOT Approved Materials, inform the Engineer and agree to pay all costs for additional materials testing.

Clean all conveyors, bins and hoppers of unapproved materials before beginning the manufacture of concrete for KDOT work.

5.0 MIXING, DELIVERY, AND PLACEMENT LIMITATIONS

a. Concrete Batching, Mixing, and Delivery. Batch and mix the concrete in a central-mix plant, in a truck mixer, or in a drum mixer at the work site. Provide plant capacity and delivery capacity sufficient to maintain continuous delivery at the rate required. The delivery rate of concrete during concreting operations must provide for the proper handling, placing and finishing of the concrete.

Seek the Engineer's approval of the concrete plant/batch site before any concrete is produced for the project. The Engineer will inspect the equipment, the method of storing and handling of materials, the production procedures, and the transportation and rate of delivery of concrete from the plant to the point of use. The Engineer will grant approval of the concrete plant/batch site based on compliance with the specified requirements. The Engineer may, at any time, rescind permission to use concrete from a previously approved concrete plant/batch site upon failure to comply with the specified requirements.

Clean the mixing drum before it is charged with the concrete mixture. Charge the batch into the mixing drum so that a portion of the water is in the drum before the aggregates and cementitious. Uniformly flow materials into the drum throughout the batching operation. Add all mixing water in the drum by the end of the first 15 seconds of the mixing cycle. Keep the throat of the drum free of accumulations that restrict the flow of materials into the drum.

Do not exceed the rated capacity (cubic yards shown on the manufacturer's plate on the mixer) of the mixer when batching the concrete. The Engineer will allow an overload of up to 10% above the rated capacity for central-mix plants and drum mixers at the work site, provided the concrete test data for strength, segregation and uniform consistency are satisfactory, and no concrete is spilled during the mixing cycle.

Operate the mixing drum at the speed specified by the mixer's manufacturer (shown on the manufacturer's plate on the mixer).

Mixing time is measured from the time all materials, except water, are in the drum. If it is necessary to increase the mixing time to obtain the specified percent of air in air-entrained concrete, the Engineer will determine the mixing time.

If the concrete is mixed in a central-mix plant or a drum mixer at the work site, mix the batch between 1 to 5 minutes at mixing speed. Do not exceed the maximum total 60 mixing revolutions. Mixing time begins after all materials, except water, are in the drum, and ends when the discharge chute opens. Transfer time in multiple drum mixers is included in mixing time. Mix time may be reduced for plants utilizing high performance mixing drums provided thoroughly mixed and uniform concrete is being produced with the proposed mix time. Performance of the plant must comply with Table A1.1, of ASTM C 94, Standard Specification for Ready Mixed Concrete. Five of the six tests listed in Table A1.1 must be within the limits of the specification to indicate that uniform concrete is being produced.

If the concrete is mixed in a truck mixer, mix the batch between 70 and 100 revolutions of the drum or blades at mixing speed. After the mixing is completed, set the truck mixer drum at agitating speed. Unless the mixing unit is equipped with an accurate device indicating and controlling the number of revolutions at mixing speed, perform the mixing at the batch plant and operate the mixing unit at agitating speed while traveling from the plant to the work site. Do not exceed 350 total revolutions (mixing and agitating).

If a truck mixer or truck agitator is used to transport concrete that was completely mixed in a stationary central mixer, agitate the concrete while transporting at the agitating speed specified by the manufacturer of the equipment (shown on the manufacturer's plate on the equipment). Do not exceed 250 total revolutions (additional re-mixing and agitating).

Provide a batch slip including batch weights of every constituent of the concrete and time for each batch of concrete delivered at the work site, issued at the batching plant that bears the time of charging of the mixer drum with cementitious and aggregates. Include quantities, type, product name and manufacturer of all admixtures on the batch ticket.

If non-agitating equipment is used for transportation of concrete, provide approved covers for protection against the weather when required by the Engineer.

Place non-agitated concrete within 30 minutes of adding the cement to the water.

Do not use concrete that has developed its initial set. Regardless of the speed of delivery and placement, the Engineer will suspend the concreting operations until corrective measures are taken if there is evidence that the concrete cannot be adequately consolidated.

Adding water to concrete after the initial mixing is prohibited. Add all water at the plant. If needed, adjust slump through the addition of a water reducer according to **subsection 3.0e.(2)**.

b. Placement Limitations.

(1) Concrete Temperature. Unless otherwise authorized by the Engineer, the temperature of the mixed concrete immediately before placement is a minimum of 55°F, and a maximum of 70°F. With approval by the Engineer, the temperature of the concrete may be adjusted 5°F above or below this range.

(2) Qualification Batch. For Grade 3.5 (AE) (LC-HPC) concrete, qualify a field batch (one truckload or at least 6 cubic yards) at least 35 days prior to commencement of placement of the bridge decks. Produce the qualification batch from the same plant that will supply the job concrete. Simulate haul time to the jobsite prior to discharge of the concrete for testing. Prior to placing concrete in the qualification slab and on the job, submit documentation to the Engineer verifying that the qualification batch concrete meets the requirements for air content, slump, temperature of plastic concrete, compressive strength, unit weight and other testing as required by the Engineer.

Before the concrete mixture with plasticizing admixture is used on the project, determine the air content of the qualification batch. Monitor the slump, air content, temperature and workability at initial batching and estimated time of concrete placement. If these properties are not adequate, repeat the qualification batch until it can be demonstrated that the mix is within acceptable limits as specified in this specification.

(3) Placing Concrete at Night. Do not mix, place or finish concrete without sufficient natural light, unless an adequate and artificial lighting system approved by the Engineer is provided.

(4) Placing Concrete in Cold Weather. Unless authorized otherwise by the Engineer, mixing and concreting operations shall not proceed once the descending ambient air temperature reaches 40°F, and may not be initiated until an ascending ambient air temperature reaches 40°F. The ascending ambient air temperature for initiating concreting operations shall increase to 45°F if the maximum ambient air temperature is expected to be between 55°F and 60°F during or within 24 hours of placement and to 50°F if the ambient air temperature is expected to equal or exceed 60°F during or within 24 hours of placement.

If the Engineer permits placing concrete during cold weather, aggregates may be heated by either steam or dry heat before placing them in the mixer. Use an apparatus that heats the weight uniformly and is so arranged as to preclude the possible occurrence of overheated areas which might injure the materials. Do not heat aggregates directly by gas or oil flame or on sheet metal over fire. Aggregates that are heated in bins, by steam-coil or water-coil heating, or by other methods not detrimental to the aggregates may be used. The use of live steam on or through binned aggregates is prohibited. Unless otherwise authorized, maintain the temperature of the mixed concrete between 55°F to 70°F at the time of placing it in the forms. With approval by the Engineer, the temperature of the concrete may be adjusted up to 5°F above or below this range. Do not place concrete when there is a probability of air temperatures being more than 25°F below the temperature of the concrete during the first 24 hours after placement unless insulation is provided for both the deck and the girders. Do not, under any circumstances, continue concrete operations if the ambient air temperature is less than 20°F.

If the ambient air temperature is 40°F or less at the time the concrete is placed, the Engineer may permit the water and the aggregates be heated to at least 70°F, but not more than 120°F.

Do not place concrete on frozen subgrade or use frozen aggregates in the concrete.

(5) Placing Concrete in Hot Weather. When the ambient temperature is above 90ºF, cool the forms, reinforcing steel, steel beam flanges, and other surfaces which will come in contact with the mix to below 90ºF by means of a water spray or other approved methods. For Grade 3.5 (AE) (LC-HPC) concrete, cool the concrete mixture to maintain the temperature immediately before placement between 55°F and 70°F. With approval by the Engineer, the temperature of the concrete may be up to 5°F below or above this range.

Maintain the temperature of the concrete at time of placement within the specified temperature range by any combination of the following:

Shading the materials storage areas or the production equipment.

Cooling the aggregates by sprinkling with potable water.

Cooling the aggregates or water by refrigeration or replacing a portion or all of the mix water with ice that is flaked or crushed to the extent that the ice will completely melt during mixing of the concrete.

- Liquid nitrogen injection.

6.0 INSPECTION AND TESTING

The Engineer will test the first truckload of concrete by obtaining a sample of fresh concrete at truck discharge and by obtaining a sample of fresh concrete at the discharge end of the conveyor, bucket or if pumped, the piping. The Engineer will obtain subsequent sample concrete for tests at the discharge end of the conveyor, bucket or if pumped, the discharge end of the piping. If potential problems are apparent at the discharge of any truck, the Engineer will test the concrete at truck discharge prior to deposit on the bridge deck.

The Engineer will cast, store, and test strength test specimens in sets of 5. See **TABLE 1-3**.

KDOT will conduct the sampling and test the samples according to **SECTION 2500** and **TABLE 1-3**. The Contractor may be directed by the Engineer to assist KDOT in obtaining the fresh concrete samples during the placement operation.

A plan will be finalized prior to the construction date as to how out-of-specification concrete will be handled.

TABLE 1-3: SAMPLING AND TESTING FREQUENCY CHART

Tests Required (Record to)	Test Method	CMS	Verification Samples and Tests	Acceptance Samples and Tests
Slump (0.25 inch)	KT-21	a	Each of first 3 truckloads for any individual placement, then 1 of every 3 truckloads	
Temperature (1°F)	KT-17	a	Every truckload, measured at the truck discharge, and from each sample made for slump determination.	
Mass (0.1 lb)	KT-20	a	One of every 6 truckloads	
Air Content (0.25%)	KT-18 or KT-19	a	Each of first 3 truckloads for any individual placement, then 1 of every 6 truckloads	
Cylinders (1 lbf; 0.1 in; 1 psi)	KT-22 and AASHTO T 22	VER	Make at least 2 groups of 5 cylinders per pour or major mix design change with concrete sampled from at least 2 different truckloads evenly spaced throughout the pour, with a minimum of 1 set for every 100 cu yd. Include in each group 3 test cylinders to be cured according to KT-22 and 2 test cylinders to be field-cured. Store the field-cured cylinders on or adjacent to the bridge. Protect all surfaces of the cylinders from the elements in as near as possible the same way as the deck concrete. Test the field-cured cylinders at the same age as the standard-cured cylinders.	
Density of Fresh Concrete (0.1 lb/cu ft or 0.1% of optimum density)	KT-36	ACI		b,c: 1 per 100 cu yd for thin overlays and bridge deck surfacing.

Note a: "Type Insp" must = "ACC" when the assignment of a pay quantity is being made. "ACI" when recording test values for additional acceptance information.
Note b: Normal operation. Minimum frequency for exceptional conditions may be reduced by the DME on a project basis, written justification shall be made to the Chief of the Bureau of Materials and Research and placed in the project documents. (Multi-Level Frequency Chart (see page 17, Appendix A of Construction Manual, Part V).
Note c: Applicable only when specifications contain those requirements.

The Engineer will reject concrete that does not comply with specified requirements.

The Engineer will permit occasional deviations below the specified cementitious content, if it is due to the air content of the concrete exceeding the designated air content, but only up to the maximum tolerance in the air content. Continuous operation below the specified cement content for any reason is prohibited.

As the work progresses, the Engineer reserves the right to require the Contractor to change the proportions if conditions warrant such changes to produce a satisfactory mix. Any such changes may be made within the limits of the Specifications at no additional compensation to the Contractor.

KANSAS DEPARTMENT OF TRANSPORTATION
SPECIAL PROVISION TO THE
STANDARD SPECIFICATIONS, 2007 EDITION

Add a new SECTION to DIVISION 700:

LOW-CRACKING HIGH-PERFORMANCE CONCRETE – CONSTRUCTION

1.0 DESCRIPTION

Construct the low-cracking high-performance concrete (LC-HPC) structures according to the Contract Documents and this specification.

BID ITEMS	UNITS
Qualification Slab	Cubic Yard
Concrete (*) (AE) (LC-HPC)	Cubic Yard
*Grade of Concrete	

2.0 MATERIALS

Provide materials that comply with the applicable requirements.

LC-HPC ..... **07-PS0166, latest version**

Concrete Curing Materials ..... **DIVISION 1400**

3.0 CONSTRUCTION REQUIREMENTS

a. Qualification Batch and Slab. For each LC-HPC bridge deck, produce a qualification batch of LC-HPC that is to be placed in the deck and complies with **07-PS0166, latest version**, and construct a qualification slab that complies with this specification to demonstrate the ability to handle, place, finish and cure the LC-HPC bridge deck.

After the qualification batch of LC-HPC complies with **07-PS0166, latest version**, construct a qualification slab 15 to 45 days prior to placing LC-HPC in the bridge deck. Construct the qualification slab to comply with the Contract Documents, using the same LC-HPC that is to be placed in the deck and that was approved in the qualification batch. Submit the location of the qualification slab for approval by the Engineer. Place, finish and cure the qualification slab according to the Contract Documents, using the same personnel, methods and equipment (including the concrete pump, if used) that will be used on the bridge deck.

A minimum of 1 day after construction of the qualification slab, core 4 full-depth 4 inch diameter cores, one from each quadrant of the qualification slab, and forward them to the Engineer for visual inspection of degree of consolidation.

Do not commence placement of LC-HPC in the deck until approval is given by the Engineer. Approval to place concrete on the deck will be based on satisfactory placement, consolidation, finishing and curing of the qualification slab and cores, and will be given or denied within 24 hours of receiving the cores from the Contractor. If an additional qualification slab is deemed necessary by the Engineer, it will be paid for at the contract unit price for Qualification Slab.

b. Falsework and Forms. Construct falsework and forms according to **SECTION 708**.

c. Handling and Placing LC-HPC.

(1) Quality Control Plan (QCP). At a project progress meeting prior to placing LC-HPC, discuss with the Engineer the method and equipment used for deck placement. Submit an acceptable QCP according to the Contractor's Concrete Structures Quality Control Plan, Part V. Detail the equipment (for both determining and controlling the evaporation rate and LC-HPC temperature), procedures used to minimize the evaporation rate, plans for maintaining a continuous rate of finishing the deck without delaying the application of curing materials within the time specified in **subsection 3.0f.**, including maintaining a continuous supply of LC-HPC throughout the placement with an adequate quantity of LC-HPC to complete the deck and filling diaphragms and end walls in advance of deck placement, and plans for placing the curing materials within the time specified in **subsection 3.0f**. In the plan, also include input from the LC-

HPC supplier as to how variations in the moisture content of the aggregate will be handled, should they occur during construction.

(2) Use a method and sequence of placing LC-HPC approved by the Engineer. Do not place LC-HPC until the forms and reinforcing steel have been checked and approved. Before placing LC-HPC, clean all forms of debris.

(3) Finishing Machine Setup. On bridges skewed greater than 10º, place LC-HPC on the deck forms across the deck on the same skew as the bridge, unless approved otherwise by State Bridge Office (SBO). Operate the bridge deck finishing machine on the same skew as the bridge, unless approved otherwise by the SBO. Before placing LP-HPC, position the finish machine throughout the proposed placement area to allow the Engineer to verify the reinforcing steel positioning.

(4) Environmental Conditions. Maintain environmental conditions on the entire bridge deck so the evaporation rate is less than 0.2 lb/sq ft/hr. The temperature of the mixed LC-HPC immediately before placement must be a minimum of 55°F and a maximum of 70°F. With approval by the Engineer, the temperature of the LC-HPC may be adjusted 5°F above or below this range. This may require placing the deck at night, in the early morning or on another day. The evaporation rate (as determined in the American Concrete Institute Manual of Concrete Practice 305R, Chapter 2) is a function of air temperature, LC-HPC temperature, wind speed and relative humidity. The effects of any fogging required by the Engineer will not be considered in the estimation of the evaporation rate (**subsection 3.0c.(5)**).

Just prior to and at least once per hour during placement of the LC-HPC, the Engineer will measure and record the air temperature, LC-HPC temperature, wind speed, and relative humidity on the bridge deck. The Engineer will take the air temperature, wind, and relative humidity measurements approximately 12 inches above the surface of the deck. With this information, the Engineer will determine the evaporation rate using KDOT software or **FIGURE 710-1**.

When the evaporation rate is equal to or above 0.2 lb/ft^2/hr, take actions (such as cooling the LC-HPC, installing wind breaks, sun screens etc.) to create and maintain an evaporation rate less than 0.2 lb/ft^2/hr on the entire bridge deck.

(5) Fogging of Deck Placements. Fogging using hand-held equipment may be required by the Engineer during unanticipated delays in the placing, finishing or curing operations. If fogging is required by the Engineer, do not allow water to drip, flow or puddle on the concrete surface during fogging, placement of absorptive material, or at any time before the concrete has achieved final set.

(6) Placement and Equipment. Place LC-HPC by conveyor belt or concrete bucket. Pumping of LC-HPC will be allowed if the Contractor can show proficiency when placing the approved mix during construction of the qualification slab using the same pump as will be used on the job. Placement by pump will also be allowed with prior approval of the Engineer contingent upon successful placement by pump of the approved mix, using the same pump as will be used for the deck placement, at least 15 days prior to placing LC-HPC in the bridge deck. To limit the loss of air, the maximum drop from the end of a conveyor belt or from a concrete bucket is 5 feet and pumps must be fitted with an air cuff/bladder valve. Do not use chutes, troughs or pipes made of aluminum.

Place LC-HPC to avoid segregation of the materials and displacement of the reinforcement. Do not deposit LC-HPC in large quantities at any point in the forms, and then run or work the LC-HPC along the forms.

Fill each part of the form by depositing the LC-HPC as near to the final position as possible.

The Engineer will obtain sample LC-HPC for tests and cylinders at the discharge end of the conveyor, bucket, or if pumped, the piping.

(7) Consolidation.

- Accomplish consolidation of the LC-HPC on all span bridges that require finishing machines by means of a mechanical device on which internal (spud or tube type) concrete vibrators of the same type and size are mounted (**subsection 154.2**).
- Observe special requirements for vibrators in contact with epoxy coated reinforcing steel as specified in **subsection 154.2**.
- Provide stand-by vibrators for emergency use to avoid delays in case of failure.
- Operate the mechanical device so vibrator insertions are made on a maximum spacing of 12 inch centers over the entire deck surface.
- Provide a uniform time per insertion of all vibrators of 3 to 15 seconds, unless otherwise designated by the Engineer.
- Provide positive control of vibrators using a timed light, buzzer, automatic control or other approved method.
- Extract the vibrators from the LC-HPC at a rate to avoid leaving any large voids or holes in the LC-HPC.
- Do not drag the vibrators horizontally through the LC-HPC.
- Use hand held vibrators (**subsection 154.2**) in inaccessible and confined areas such as along bridge rail or curb.

- When required, supplement vibrating by hand spading with suitable tools to provide required consolidation.
- Reconsolidate any voids left by workers.

Continuously place LC-HPC in any floor slab until complete, unless shown otherwise in the Contract Documents.

d. Construction Joints, Expansion Joints and End of Wearing Surface (EWS) Treatment. Locate the construction joints as shown in the Contract Documents. If construction joints are not shown in the Contract Documents, submit proposed locations for approval by the Engineer.

If the work of placing LC-HPC is delayed and the LC-HPC has taken its initial set, stop the placement, saw the nearest construction joint approved by the Engineer, and remove all LC-HPC beyond the construction joint.

Construct keyed joints by embedding water-soaked beveled timbers of a size shown on the Contract Documents, into the soft LC-HPC. Remove the timber when the LC-HPC has set. When resuming work, thoroughly clean the surface of the LC-HPC previously placed, and when required by the Engineer, roughen the key with a steel tool. Before placing LC-HPC against the keyed construction joint, thoroughly wash the surface of the keyed joint with clean water.

e. Finishing. Strike off bridge decks with a vibrating screed or single-drum roller screed, either self-propelled or manually operated by winches and approved by the Engineer. Use a self-oscillating screed on the finish machine, and operate or finish from a position either on the skew or transverse to the bridge roadway centerline. See **subsection 3.0c.(3)**. Do not mount tamping devices or fixtures to drum roller screeds; augers are allowed.

Irregular sections may be finished by other methods approved by the Engineer and detailed in the required QCP. See **subsection 3.0c.(1)**.

Finish the surface by a burlap drag, metal pan or both, mounted to the finishing equipment. Use a float or other approved device behind the burlap drag or metal pan, as necessary, to remove any local irregularities. Do not add water to the surface of LC-HPC. Do not use a finishing aid.

Tining of plastic LC-HPC is prohibited. All LC-HPC surfaces must be reasonably true and even, free from stone pockets, excessive depressions or projections beyond the surface.

Finish all top surfaces, such as the top of retaining walls, curbs, abutments and rails, with a wooden float by tamping and floating, flushing the mortar to the surface and provide a uniform surface, free from pits or porous places. Trowel the surface producing a smooth surface, and brush lightly with a damp brush to remove the glazed surface.

f. Curing and Protection.

(1) General. Cure all newly placed LC-HPC immediately after finishing, and continue uninterrupted for a minimum of 14 days. Cure all pedestrian walkway surfaces in the same manner as the bridge deck. Curing compounds are prohibited during the 14 day curing period.

(2) Cover With Wet Burlap. Soak the burlap a minimum of 12 hours prior to placement on the deck. Rewet the burlap if it has dried more one hour before it is applied to the surface of bridge deck. Apply 1 layer of wet burlap within 10 minutes of LC-HPC strike-off from the screed, followed by a second layer of wet burlap within 5 minutes. Do not allow the surface to dry after the strike-off, or at any time during the cure period. In the required QCP, address the rate of LC-HPC placement and finishing methods that will affect the period between strike-off and burlap placement. See **subsection 3.0c.(1)**. During times of delay expected to exceed 10 minutes, cover all concrete that has been placed, but not finished, with wet burlap.

Maintain the wet burlap in a fully wet condition using misting hoses, self-propelled, machine-mounted fogging equipment with effective fogging area spanning the deck width moving continuously across the entire burlap-covered surface, or other approved devices until the LC-HPC has set sufficiently to allow foot traffic. At that time, place soaker hoses on the burlap, and supply running water continuously to maintain continuous saturation of all burlap material to the entire LC-HPC surface. For bridge decks with superelevation, place a minimum of 1 soaker hose along the high edge of the deck to keep the entire deck wet during the curing period.

(3) Waterproof Cover. Place white polyethylene film on top of the soaker hoses, covering the entire LC-HPC surface after soaker hoses have been placed, a maximum of 12 hours after the placement of the LC-HPC. Use as wide of sheets as practicable, and overlap 2 feet on all edges to form a complete waterproof cover of the entire LC-HPC surface. Secure the polyethylene film so that wind will not displace it. Should any portion of the sheets be broken or damaged before expiration of the curing period, immediately repair the broken or damaged portions. Replace sections that have lost their waterproof qualities.

If burlap and/or polyethylene film is temporarily removed for any reason during the curing period, use soaker hoses to keep the entire exposed area continuously wet. Replace saturated burlap and polyethylene film, resuming the specified curing conditions, as soon as possible.

Inspect the LC-HPC surface once every 6 hours for the entirety of the 14 day curing period, so that all areas remain wet for the entire curing period and all curing requirements are satisfied.

(4) Documentation. Provide the Engineer with a daily inspection set that includes:

- documentation that identifies any deficiencies found (including location of deficiency);
- documentation of corrective measures taken;
- a statement of certification that the entire bridge deck is wet and all curing material is in place;
- documentation showing the time and date of all inspections and the inspector's signature.
- documentation of any temporary removal of curing materials including location, date and time, length of time curing was removed, and means taken to keep the exposed area continuously wet.

(5) Cold Weather Curing. When LC-HPC is being placed in cold weather, also adhere to **07-PS0166, latest version**.

When LC-HPC is being placed and the ambient air temperature may be expected to drop below 40ºF during the curing period or when the ambient air temperature is expected to drop more than 25°F below the temperature of the LC-HPC during the first 24 hours after placement, provide suitable measures such as straw, additional burlap, or other suitable blanketing materials, and/or housing and artificial heat to maintain the LC-HPC and girder temperatures between 40ºF and 75ºF as measured on the upper and lower surfaces of the LC-HPC. Enclose the area underneath the deck and heat so that the temperature of the surrounding air is as close as possible to the temperature of LC-HPC and between 40ºF and 75ºF. When artificial heating is used to maintain the LC-HPC and girder temperatures, provide adequate ventilation to limit exposure to carbon dioxide if necessary. Maintain wet burlap and polyethylene cover during the entire 14 day curing period. Heating may be stopped after the first 72 hours if the time of curing is lengthened to account for periods when the ambient air temperature is below 40ºF. For every day the ambient air temperature is below 40ºF, an additional day of curing with a minimum ambient air temperature of 50ºF will be required. After completion of the required curing period, remove the curing and protection so that the temperature of the LC-HPC during the first 24 hours does not fall more than 25°F.

(6) Curing Membrane. At the end of the 14-day curing period remove the wet burlap and polyethylene and within 30 minutes, apply 2 coats of an opaque curing membrane to the LC-HPC. Apply the curing membrane when no free water remains on the surface but while the surface is still wet. Apply each coat of curing membrane according to the manufacturer's instructions with a minimum spreading rate per coat of 1 gallon per 80 square yards of LC-HPC surface. If the LC-HPC is dry or becomes dry, thoroughly wet it with water applied as a fog spray by means of approved equipment. Spray the second coat immediately after and at right angles to the first application.

Protect the curing membrane against marring for a minimum of 7 days. Give any marred or disturbed membrane an additional coating. Should the curing membrane be subjected to continuous injury, the Engineer may limit work on the deck until the 7-day period is complete. Because the purpose of the curing membrane is to allow for slow drying of the bridge deck, extension of the initial curing period beyond 14 days, while permitted, shall not be used to reduce the 7-day period during which the curing membrane is applied and protected.

(7) Construction Loads. Adhere to **TABLE 710-2**.

If the Contractor needs to drive on the bridge before the approach slabs can be placed and cured, construct a temporary bridge from the approach over the EWS capable of supporting the anticipated loads. Do not bend the reinforcing steel which will tie the approach slab to the EWS or damage the LC-HPC at the EWS. The method of bridging must be approved by the Engineer.

TABLE 710-2: CONCRETE LOAD LIMITATIONS ON BRIDGE DECKS		
Days after concrete is placed	**Element**	**Allowable Loads**
1*	Subdeck, one-course deck or concrete overlay	Foot traffic only.
3*	One-course deck or concrete overlay	Work to place reinforcing steel or forms for the bridge rail or barrier.
7*	Concrete overlays	Legal Loads; Heavy stationary loads with the Engineer's approval.***
10 (15)**	Subdeck, one-course deck or post-tensioned haunched slab bridges**	Light truck traffic (gross vehicle weight less than 5 tons).****
14 (21)**	Subdeck, one-course deck or post-tensioned haunched slab bridges**	Legal Loads; Heavy stationary loads with the Engineer's approval.***Overlays on new decks.
28	Bridge decks	Overloads, only with the State Bridge Engineer's approval.***

*Maintain a 7 day wet cure at all times (14-day wet cure for decks with LC-HPC).
** Conventional haunched slabs.
*** Submit the load information to the appropriate Engineer. Required information: the weight of the material and the footprint of the load, or the axle (or truck) spacing and the width, the size of each tire (or track length and width) and their weight.
****An overlay may be placed using pumps or conveyors until legal loads are allowed on the bridge.

g. Grinding and Grooving. Correct surface variations exceeding 1/8 inch in 10 feet by use of an approved profiling device, or other methods approved by the Engineer after the curing period. Perform grinding on hardened LC-HPC after the 7 day curing membrane period to achieve a plane surface and grooving of the final wearing surface as shown in the Contract Documents.

Use a self-propelled grinding machine with diamond blades mounted on a multi-blade arbor. Avoid using equipment that causes excessive ravels, aggregate fractures or spalls. Use vacuum equipment or other continuous methods to remove grinding slurry and residue.

After any required grinding is complete, give the surface a suitable texture by transverse grooving. Use diamond blades mounted on a self-propelled machine that is designed for texturing pavement. Transverse grooving of the finished surface may be done with equipment that is not self-propelled providing that the Contractor can show proficiency with the equipment. Use equipment that does not cause strain, excessive raveling, aggregate fracture, spalls, disturbance of the transverse or longitudinal joint, or damage to the existing LC-HPC surface. Make the grooving approximately 3/16 inch in width at 3/4 inch centers and the groove depth approximately 1/8 inch. For bridges with drains, terminate the transverse grooving approximately 2 feet in from the gutter line at the base of the curb. Continuously remove all slurry residues resulting from the texturing operation.

h. Post Construction Conference. At the completion of the deck placement, curing, grinding and grooving for a bridge using LC-HPC, a post-construction conference will be held with all parties that participated in the planning and construction present. The Engineer will record the discussion of all problems and successes for the project.

i. Removal of Forms and Falsework. Do not remove forms and falsework without the Engineer's approval. Remove deck forms approximately 2 weeks (a maximum of 4 weeks) after the end of the curing period (removal of burlap), unless approved by the Engineer. The purpose of 4 week maximum is to limit the moisture gradient between the bottom and the top of the deck.

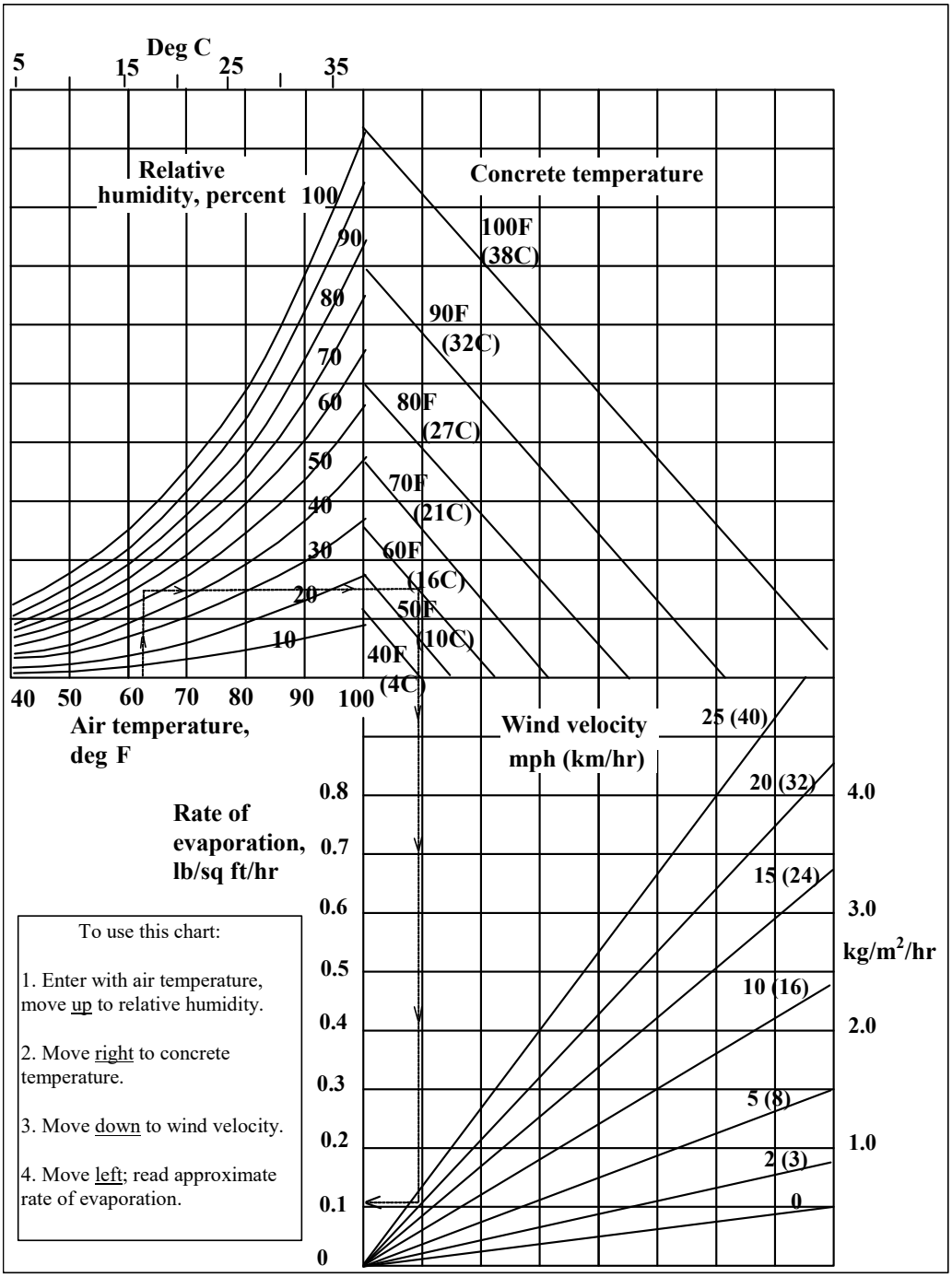
For additional requirements regarding forms and falsework, see **SECTION 708**.

4.0 MEASUREMENT AND PAYMENT

The Engineer will measure the qualification slab and the various grades of (AE) (LC-HPC) concrete placed in the structure by the cubic yard. No deductions are made for reinforcing steel and pile heads extending into the LP-HPC. The Engineer will not separately measure reinforcing steel in the qualification slab.

Payment for the "Qualification Slab" and the various grades of "(AE) (LC-HPC) Concrete" at the contract unit prices is full compensation for the specified work.

FIGURE 710-1: STANDARD PRACTICE FOR CURING CONCRETE



Effect of concrete and air temperatures, relative humidity, and wind velocity on the rate of evaporation of surface moisture from concrete. This chart provides a graphic method of estimating the loss of surface moisture for various weather conditions. To use the chart, follow the four steps outlined above. When the evaporation rate exceeds 0.2 lb/ft^2/hr (1.0 kg/ m^2/hr), measures shall be taken to prevent excessive moisture loss from the surface of unhardened concrete; when the rate is less than 0.2 lb/ft^2/hr (1.0 kg/m^2/hr) such measures may be needed. When excessive moisture loss is not prevented, plastic cracking is likely to occur.

APPENDIX B : BRIDGE DECK SURVEY SPECIFICATIONS

1.0 DESCRIPTION.

This specification covers the procedures and requirements to perform bridge deck surveys of reinforced concrete bridge decks.

2.0 SURVEY REQUIREMENTS.

a. Pre-Survey Preparation.

(1) Prior to performing the crack survey, related construction documents need to be gathered to produce a scaled drawing of the bridge deck. The scale must be exactly 1 in. = 10 ft (for use with the scanning software), and the drawing only needs to include the boundaries of the deck surface.

NOTE 1 – In the event that it is not possible to produce a scaled drawing prior to arriving at the bridge deck, a hand-drawn crack map (1 in.= 10 ft) created on engineering paper using measurements taken in the field is acceptable.

(2) The scaled drawing should also include compass and traffic directions in addition to deck stationing. A scaled 5 ft by 5 ft grid is also required to aid in transferring the cracks observed on the bridge deck to the scaled drawing. The grid shall be drawn separately and attached to the underside of the crack map such that the grid can easily be seen through the crack map.

NOTE 2 – Maps created in the field on engineering paper need not include an additional grid.

(3) For curved bridges, the scaled drawing need not be curved, i.e., the curve may be approximated using straight lines.

(4) Coordinate with traffic control so that at least one side (or one lane) of the bridge can be closed during the time that the crack survey is being performed.

b. Preparation of Surface.

(1) After traffic has been closed, station the bridge in the longitudinal direction at ten feet intervals. The stationing shall be done as close to the centerline as possible. For curved bridges, the stationing shall follow the curve.

(2) Prior to beginning the crack survey, mark a 5 ft by 5 ft grid using lumber crayons or chalk on the portion of the bridge closed to traffic corresponding to the grid on the scaled drawing. Measure and document any drains, repaired areas, unusual cracking, or any other items of interest.

(3) Starting with one end of the closed portion of the deck, using a lumber crayon or chalk, begin tracing cracks that can be seen while bending at the waist. After beginning to trace cracks, continue to the end of the crack, even if this includes portions of the crack that were not initially seen while bending at the waist. Areas covered by sand or other debris need not be surveyed. Trace the cracks using a different color crayon than was used to mark the grid and stationing.

(4) At least one person shall recheck the marked portion of the deck for any additional cracks. The goal is not to mark every crack on the deck, only those cracks that can initially be seen while bending at the waist.

NOTE 3 – An adequate supply of lumber crayons or chalk should be on hand for the survey. Crayon or chalk colors should be selected to be readily visible when used to mark the concrete.

c. Weather Limitations.

(1) Surveys are limited to days when the expected temperature during the survey will not be below 60 °F.

(2) Surveys are further limited to days that are forecasted to be at least mostly sunny for a majority of the day.

(3) Regardless of the weather conditions, the bridge deck must be completely dry before the survey can begin.

3.0 BRIDGE SURVEY.

a. Crack Surveys.

Using the grid as a guide, transfer the cracks from the deck to the scaled drawing. Areas that are not surveyed should be marked on the scaled drawing. Spalls, regions of scaling, and other areas of special interest need not be included on the scale drawings but should be noted.

b. Delamination Survey.

At any time during or after the crack survey, bridge decks shall be checked for delamination. Any areas of delamination shall be noted and drawn on a separate drawing of the bridge. This second drawing need not be to scale.

c. Under Deck Survey.

Following the crack and delamination survey, the underside of the deck shall be examined and any unusual or excessive cracking noted.

APPENDIX C: LENGTH–CHANGE MEASUREMENTS FOR MIXTURES USED IN CHAPTER 2

TABLE C.1 : LENGTH-CHANGE MEASURENETS FOR CONTROL MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)		
		Control		
		A	C	Average
0	0.00	0	0	0
0	0.03	-10	-40	-25
0	0.08	30	0	15
0	0.13	20	-20	0
0	0.16	20	-20	0
0	0.78	30	-30	0
0	1.00	32	-28	2
0	2.00	40	-20	10
0	3.00	70	50	60
0	4.00	60	40	50
0	5.00	50	30	40
0	6.00	60	40	50
0	7.00	60	50	55
0	8.00	60	40	50
0	9.00	60	40	50
0	10.00	60	40	50
0	11.00	60	40	50
0	12.00	60	40	50
0	13.00	60	50	55
0	14.00	60	50	55
1	15.00	0	-20	-10
2	16.00	-40	-70	-55
3	17.00	-60	-120	-90
4	18.00	-90	-150	-120
5	19.00	-130	-170	-150
6	20.00	-140	-190	-165
7	21.00	-150	-200	-175
8	22.00	-150	-200	-175
9	23.00	-200	-240	-220
10	24.00	-210	-260	-235
11	25.00	-210	-260	-235
12	26.00	-220	-270	-245
13	27.00	-220	-270	-245

TABLE C.1 (con't) : LENGTH-CHANGE MEASURENETS FOR CONTROL MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)		
		Control		
		A	C	Average
14	28.00	-220	-280	-250
15	29.00	-250	-300	-275
16	30.00	-250	-300	-275
17	31.00	-270	-320	-295
18	32.00	-270	-320	-295
19	33.00	-270	-320	-295
20	34.00	-270	-320	-295
21	35.00	-280	-330	-305
22	36.00	-280	-330	-305
23	37.00	-280	-330	-305
24	38.00	-280	-320	-300
25	39.00	-280	-320	-300
26	40.00	-280	-330	-305
27	41.00	-280	-330	-305
28	42.00	-280	-340	-310
29	43.00	-290	-350	-320
30	44.00	-300	-350	-325
32	46.00	-310	-360	-335
34	48.00	-310	-360	-335
36	50.00	-310	-360	-335
38	52.00	-320	-370	-345
40	54.00	-320	-380	-350
42	56.00	-330	-390	-360
44	58.00	-330	-390	-360
46	60.00	-340	-400	-370
48	62.00	-350	-410	-380
50	64.00	-340	-400	-370
52	66.00	-350	-400	-375
54	68.00	-360	-410	-385
56	70.00	-360	-410	-385
58	72.00	-370	-410	-390
60	74.00	-370	-410	-390
62	76.00	-370	-410	-390
64	78.00	-400	-440	-420
66	80.00	-400	-440	-420

TABLE C.2 (con't) : LENGTH-CHANGE MEASURENETS FOR CONTROL MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)		
		Control		
		A	C	Average
68	82.00	-400	-440	-420
70	84.00	-400	-440	-420
72	86.00	-410	-450	-430
74	88.00	-410	-450	-430
76	90.00	-410	-450	-430
78	92.00	-410	-450	-430
80	94.00	-410	-450	-430
82	96.00	-400	-450	-425
84	98.00	-400	-450	-425
86	100.00	-420	-470	-445
88	102.00	-430	-480	-455
90	104.00	-450	-490	-470
92	106.00	-450	-490	-470
94	108.00	-440	-480	-460
101	115.00	-430	-480	-455
108	122.00	-450	-490	-470
115	129.00	-450	-490	-470
122	136.00	-440	-490	-465
129	143.00	-450	-510	-480
136	150.00	-450	-510	-480
143	157.00	-450	-520	-485
150	164.00	-470	-540	-505
157	171.00	-480	-550	-515
164	178.00	-470	-530	-500
171	185.00	-470	-530	-500
178	192.00	-470	-520	-495
180	194.00	-470	-525	-498

TABLE C.2 : LENGTH-CHANGE MEASURENETS FOR SCA 1 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCA 1			
		A	B	C	Average
0	0	0	0	0	0
0	0.0	20	10	10	13
0	0.1	40	20	20	27
0	0.1	40	20	30	30
0	0.2	60	30	40	43
0	0.2	50	20	50	40
0	0.3	20	0	30	17
0	0.8	70	60	50	60
0	1	74	64	54	64
0	2	90	80	70	80
0	3	110	90	80	93
0	4	120	120	100	113
0	5	140	130	120	130
0	6	160	150	140	150
0	7	170	150	150	157
0	8	170	170	160	167
0	9	190	180	180	183
0	10	190	180	180	183
0	11	200	190	190	193
0	12	210	200	190	200
0	13	210	210	190	203
0	14	220	210	200	210
1	15	180	160	160	167
2	16	140	130	120	130
3	17	130	110	110	117
4	18	120	100	100	107
5	19	110	90	90	97
6	20	90	80	70	80
7	21	80	60	60	67
8	22	70	60	60	63

TABLE C.2 (con't) : LENGTH-CHANGE MEASURENETS FOR SCA 1 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCA 1			
		A	B	C	Average
9	23	60	50	50	53
10	24	50	40	30	40
11	25	40	20	20	27
12	26	30	10	10	17
13	27	30	10	10	17
14	28	20	10	10	13
15	29	20	0	0	7
16	30	20	0	-10	3
17	31	10	-10	-20	-7
18	32	0	-20	-30	-17
19	33	-10	-20	-40	-23
20	34	-20	-40	-50	-37
21	35	-30	-50	-40	-40
22	36	-30	-40	-40	-37
23	37	-30	-40	-40	-37
24	38	-20	-40	-40	-33
25	39	-10	-30	-30	-23
26	40	-10	-30	-30	-23
27	41	-10	-30	-40	-27
28	42	-10	-30	-40	-27
29	43	-10	-30	-40	-27
30	44	-10	-30	-40	-27
32	46	-10	-30	-40	-27
34	48	-10	-40	-40	-30
36	50	-40	-50	-50	-47
38	52	-40	-50	-50	-47
40	54	-50	-60	-50	-53
42	56	-50	-60	-50	-53
44	58	-40	-70	-60	-57
46	60	-60	-80	-80	-73
48	62	-70	-90	-90	-83
50	64	-80	-100	-100	-93
52	66	-80	-100	-100	-93
54	68	-90	-100	-100	-97
56	70	-90	-100	-100	-97
58	72	-90	-110	-120	-107

TABLE C.2 (con't) : LENGTH-CHANGE MEASURENETS FOR SCA 1 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCA 1			
		A	B	C	Average
60	74	-90	-100	-120	-103
62	76	-90	-90	-110	-97
64	78	-90	-90	-110	-97
66	80	-80	-90	-110	-93
68	82	-80	-90	-100	-90
70	84	-90	-100	-110	-100
72	86	-100	-110	-110	-107
74	88	-100	-110	-110	-107
76	90	-100	-110	-110	-107
78	92	-100	-110	-110	-107
80	94	-100	-110	-110	-107
82	96	-100	-110	-110	-107
84	98	-100	-120	-110	-110
86	100	-100	-120	-120	-113
88	102	-100	-120	-120	-113
90	104	-100	-120	-120	-113
92	106	-100	-120	-120	-113
94	108	-110	-130	-130	-123
101	115	-110	-140	-140	-130
108	122	-120	-150	-150	-140
115	129	-130	-150	-150	-143
122	136	-130	-160	-150	-147
129	143	-140	-170	-160	-157
136	150	-140	-170	-160	-157
143	157	-140	-160	-160	-153
150	164	-140	-160	-170	-157
157	171	-160	-200	-190	-183
164	178	-160	-200	-190	-183
171	185	-160	-190	-190	-180
178	192	-160	-180	-180	-173
180	194	-160	-180	-180	-173

TABLE C.3 : LENGTH-CHANGE MEASURENETS FOR IC-SCM-SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC-SCM-SCA 2			
		A	B	C	Average
0	0	0	0	0	0
0	0.0	30	50	30	37
0	0.1	70	80	70	73
0	0.1	110	140	100	117
0	0.2	210	260	230	233
0	0.3	300	280	280	287
0	0.8	400	390	390	393
0	1	411	398	398	402
0	2	470	440	440	450
0	3	500	480	470	483
0	4	520	490	470	493
0	5	530	500	470	500
0	6	540	510	470	507
0	7	540	540	510	530
0	8	560	550	520	543
0	9	560	560	510	543
0	10	540	540	490	523
0	11	540	540	490	523
0	12	540	540	500	527
0	13	540	540	500	527
0	14	540	530	500	523
1	15	520	500	460	493
2	16	510	500	460	490
3	17	510	510	460	493
4	18	500	500	450	483
5	19	490	490	440	473
6	20	470	490	420	460
7	21	430	450	390	423
8	22	470	480	430	460
9	23	460	450	410	440

TABLE C.3 (con't) : LENGTH-CHANGE MEASURENETS FOR IC-SCM-SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC-SCM-SCA 2			
		A	B	C	Average
10	24	470	460	410	447
11	25	450	440	390	427
12	26	430	420	370	407
13	27	420	410	360	397
14	28	400	400	350	383
15	29	390	390	340	373
16	30	390	370	330	363
17	31	390	370	330	363
18	32	380	370	330	360
19	33	370	360	330	353
20	34	370	360	330	353
21	35	360	350	320	343
22	36	370	360	320	350
23	37	360	350	310	340
24	38	340	340	290	323
25	39	340	340	290	323
26	40	330	340	280	317
27	41	330	340	280	317
28	42	330	340	280	317
29	43	330	340	280	317
30	44	320	330	270	307
32	46	320	320	270	303
34	48	320	320	270	303
36	50	290	310	250	283
38	52	290	310	240	280
40	54	280	300	230	270
42	56	270	290	230	263
44	58	250	280	210	247
46	60	250	270	200	240
48	62	240	270	190	233
50	64	240	270	190	233
52	66	230	270	190	230
54	68	220	250	180	217
56	70	210	240	180	210

TABLE C.3 (con't) : LENGTH-CHANGE MEASURENETS FOR IC-SCM-SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC-SCM-SCA 2			
		A	B	C	Average
58	72	210	230	170	203
60	74	210	230	170	203
62	76	210	230	170	203
64	78	210	230	170	203
66	80	210	240	170	207
68	82	200	220	150	190
70	84	190	210	140	180
72	86	200	220	160	193
74	88	220	230	180	210
76	90	240	250	200	230
78	92	220	230	190	213
80	94	210	220	180	203
82	96	200	230	170	200
84	98	200	240	160	200
86	100	220	250	180	217
88	102	220	250	180	217
90	104	220	250	180	217
92	106	210	220	180	203
94	108	220	230	190	213
101	115	220	230	190	213
108	122	210	220	190	207
115	129	180	190	160	177
122	136	190	200	150	180
129	143	180	190	140	170
136	150	170	180	120	157
143	157	150	170	120	147
150	164	150	160	130	147
157	171	180	180	140	167
164	178	160	170	130	153
171	185	150	170	120	147
178	192	140	160	110	137
180	194	110	120	90	107

TABLE C.4 : LENGTH-CHANGE MEASURENETS FOR IC-SCM-SCA 1 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC-SCM-SCA 1			
		A	B	C	Average
0	0	0	0	0	0
0	0.1	-60	-80	10	-43
0	0.1	0	10	50	20
0	0.2	10	20	60	30
0	0.3	20	20	70	37
0	0.8	80	40	90	70
0	1	84	46	96	75
0	2	100	70	120	97
0	3	100	80	120	100
0	4	120	90	140	117
0	5	140	100	160	133
0	6	150	110	170	143
0	7	160	120	190	157
0	8	180	140	190	170
0	9	170	130	180	160
0	10	190	150	210	183
0	11	190	150	210	183
0	12	200	160	220	193
0	13	200	160	220	193
0	14	220	160	250	210
1	15	180	130	230	180
2	16	170	130	210	170
3	17	150	110	200	153
4	18	140	100	190	143
5	19	130	90	180	133
6	20	120	90	180	130
7	21	120	100	180	133
8	22	120	100	180	133
9	23	120	80	160	120
10	24	100	80	160	113

TABLE C.4 (con't) : LENGTH-CHANGE MEASURENETS FOR IC-SCM-SCA 1 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC-SCM-SCA 1			
		A	B	C	Average
11	25	100	80	160	113
12	26	90	70	150	103
13	27	90	70	150	103
14	28	80	60	140	93
15	29	70	50	130	83
16	30	70	50	120	80
17	31	70	50	120	80
18	32	60	50	120	77
19	33	50	40	110	67
20	34	50	30	100	60
21	35	40	20	90	50
22	36	20	0	70	30
23	37	10	-10	50	17
24	38	10	-10	40	13
25	39	10	-20	30	7
26	40	10	-30	20	0
27	41	10	-30	20	0
28	42	20	-20	20	7
29	43	20	-20	20	7
30	44	20	-10	20	10
32	46	-10	-30	10	-10
34	48	-30	-40	10	-20
36	50	-50	-60	0	-37
38	52	-40	-50	10	-27
40	54	-50	-60	0	-37
42	56	-50	-60	0	-37
44	58	-50	-60	0	-37
46	60	-60	-70	-20	-50
48	62	-70	-90	-30	-63
50	64	-60	-70	-20	-50
52	66	-50	-60	-10	-40
54	68	-50	-50	0	-33
56	70	-40	-40	10	-23
58	72	-30	-40	10	-20
60	74	-40	-50	0	-30

TABLE C.4 (con't) : LENGTH-CHANGE MEASURENETS FOR IC-SCM-SCA 1 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC-SCM-SCA 1			
		A	B	C	Average
62	76	-50	-50	-10	-37
64	78	-50	-40	-10	-33
66	80	-40	-40	-10	-30
68	82	-40	-50	-20	-37
70	84	-50	-50	-20	-40
72	86	-20	-60	-30	-37
74	88	-20	-60	-30	-37
76	90	-20	-50	-30	-33
78	92	-40	-50	-20	-37
80	94	-30	-50	-10	-30
82	96	-40	-60	-20	-40
84	98	-70	-70	-30	-57
86	100	-70	-60	-10	-47
88	102	-70	-70	-20	-53
90	104	-70	-70	-30	-57
92	106	-70	-80	-30	-60
94	108	-80	-90	-50	-73
101	115	-90	-90	-50	-77
108	122	-90	-90	-50	-77
115	129	-110	-130	-80	-107
122	136	-120	-120	-80	-107
129	143	-120	-130	-90	-113
136	150	-90	-110	-60	-87
143	157	-110	-110	-70	-97
150	164	-120	-120	-80	-107
157	171	-120	-130	-90	-113
164	178	-120	-130	-100	-117
171	185	-110	-120	-90	-107
178	192	-120	-120	-90	-110
180	194	-120	-120	-90	-110

TABLE C.5 : LENGTH-CHANGE MEASURENETS FOR SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCA 2			
		A	B	C	Average
0	0	0	0	0	0
0	0.0	20	80	-120	-7
0	0.1	70	100	-80	30
0	0.1	120	130	-30	73
0	0.2	150	160	20	110
0	0.8	240	280	130	217
0	1	247	293	139	227
0	2	280	350	180	270
0	3	330	390	210	310
0	4	330	400	220	317
0	5	340	410	230	327
0	6	340	410	230	327
0	7	340	410	220	323
0	8	340	410	220	323
0	9	330	400	230	320
0	10	340	400	230	323
0	11	350	410	230	330
0	12	360	410	230	333
0	13	350	410	220	327
0	14	360	420	220	333
1	15	280	330	150	253
2	16	250	300	110	220
3	17	210	260	70	180
4	18	200	240	60	167
5	19	190	230	50	157
6	20	180	210	40	143
7	21	160	200	20	127
8	22	140	180	0	107
9	23	130	170	-10	97
10	24	110	150	-40	73

TABLE C.5 (con't) : LENGTH-CHANGE MEASURENETS FOR SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCA 2			
		A	B	C	Average
11	25	100	140	-40	67
12	26	90	140	-50	60
13	27	90	130	-50	57
14	28	80	120	-60	47
15	29	70	110	-60	40
16	30	50	90	-100	13
17	31	50	90	-100	13
18	32	50	90	-100	13
19	33	40	80	-110	3
20	34	40	80	-110	3
21	35	40	80	-110	3
22	36	20	60	-130	-17
23	37	20	60	-130	-17
24	38	10	50	-140	-27
25	39	10	50	-140	-27
26	40	10	50	-140	-27
27	41	10	50	-140	-27
28	42	10	50	-140	-27
29	43	10	40	-140	-30
30	44	10	40	-140	-30
32	46	0	30	-140	-37
34	48	0	30	-140	-37
36	50	-10	20	-160	-50
38	52	-20	20	-180	-60
40	54	-20	20	-180	-60
42	56	-20	20	-190	-63
44	58	-30	20	-190	-67
46	60	-40	10	-200	-77
48	62	-40	0	-210	-83

TABLE C.5 (con't) : LENGTH-CHANGE MEASURENETS FOR SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCA 2			
		A	B	C	Average
50	64	-40	0	-210	-83
52	66	-40	0	-210	-83
54	68	-50	-10	-210	-90
56	70	-50	-10	-210	-90
58	72	-50	-10	-210	-90
60	74	-60	-20	-220	-100
62	76	-60	-20	-230	-103
64	78	-70	-30	-230	-110
66	80	-80	-40	-240	-120
68	82	-90	-50	-250	-130
70	84	-100	-60	-260	-140
72	86	-100	-60	-260	-140
74	88	-100	-60	-260	-140
76	90	-100	-60	-270	-143
78	92	-110	-60	-270	-147
80	94	-110	-60	-280	-150
82	96	-110	-60	-280	-150
84	98	-110	-60	-280	-150
86	100	-110	-60	-270	-147
88	102	-110	-60	-270	-147
90	104	-110	-70	-270	-150
92	106	-110	-70	-270	-150
94	108	-100	-80	-270	-150
101	115	-130	-90	-310	-177
108	122	-130	-90	-310	-177
115	129	-140	-100	-310	-183
122	136	-140	-90	-310	-180
129	143	-130	-90	-300	-173
136	150	-150	-110	-310	-190
143	157	-150	-100	-310	-187
150	164	-150	-110	-320	-193
157	171	-160	-120	-340	-207
164	178	-170	-130	-360	-220
171	185	-170	-120	-340	-210
178	192	-160	-120	-340	-207
180	194	-157	-117	-337	-204

TABLE C.6 : LENGTH-CHANGE MEASURENETS FOR IC-SCM MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)		
		IC-SCM		
		A	C	Average
0	0	0	0	0
0	0.1	80	40	60
0	0.1	70	50	60
0	0.1	70	40	55
0	0.2	70	40	55
0	0.3	80	60	70
0	0.4	80	60	70
0	0.7	70	70	70
0	1	68	68	68
0	2	60	60	60
0	3	70	70	70
0	4	90	90	90
0	5	80	80	80
0	6	70	70	70
0	7	70	70	70
0	8	70	70	70
0	9	70	70	70
0	10	70	70	70
0	11	70	70	70
0	12	70	70	70
0	13	80	80	80
0	14	80	80	80
1	15	70	70	70
2	16	60	60	60
3	17	50	50	50
4	18	40	40	40
5	19	30	30	30
6	20	10	30	20
7	21	0	20	10
8	22	-10	0	-5
9	23	-20	-10	-15

TABLE C.6 (con't) : LENGTH-CHANGE MEASURENETS FOR IC-SCM MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)		
		IC-SCM		
		A	C	Average
10	24	-30	-20	-25
11	25	-40	-30	-35
12	26	-50	-30	-40
13	27	-50	-50	-50
14	28	-70	-70	-70
15	29	-50	-50	-50
16	30	-60	-60	-60
17	31	-70	-70	-70
18	32	-80	-80	-80
19	33	-90	-80	-85
20	34	-100	-100	-100
21	35	-80	-80	-80
22	36	-110	-110	-110
23	37	-110	-120	-115
24	38	-120	-130	-125
25	39	-130	-140	-135
26	40	-140	-150	-145
27	41	-130	-150	-140
28	42	-130	-150	-140
29	43	-150	-150	-150
30	44	-150	-170	-160
32	46	-150	-170	-160
34	48	-160	-180	-170
36	50	-190	-200	-195
38	52	-190	-210	-200
40	54	-190	-210	-200
42	56	-220	-220	-220
44	58	-240	-240	-240
46	60	-250	-250	-250
48	62	-210	-220	-215
50	64	-220	-230	-225
52	66	-240	-250	-245
54	68	-240	-260	-250

TABLE C.6 (con't) : LENGTH-CHANGE MEASURENETS FOR IC-SCM MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)		
		IC-SCM		
		A	C	Average
56	70	-260	-270	-265
58	72	-260	-270	-265
60	74	-260	-270	-265
62	76	-250	-270	-260
64	78	-270	-280	-275
66	80	-270	-290	-280
68	82	-280	-300	-290
70	84	-280	-300	-290
72	86	-280	-300	-290
74	88	-280	-300	-290
76	90	-290	-320	-305
78	92	-280	-310	-295
80	94	-280	-310	-295
82	96	-290	-310	-300
84	98	-320	-340	-330
86	100	-330	-340	-335
88	102	-330	-350	-340
90	104	-320	-350	-335
92	106	-310	-340	-325
94	108	-310	-350	-330
101	115	-310	-350	-330
108	122	-310	-350	-330
115	129	-310	-360	-335
122	136	-320	-360	-340
129	143	-320	-360	-340
136	150	-320	-360	-340
143	157	-320	-360	-340
150	164	-320	-360	-340
157	171	-320	-360	-340
164	178	-320	-360	-340
171	185	-320	-360	-340
178	192	-320	-360	-340
180	194	-320	-360	-340

TABLE C.7 : LENGTH-CHANGE MEASURENETS FOR SCM MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCM			
		A	B	C	Average
0	0	0	0	0	0
0	0.1	30	10	0	13
0	0.2	30	-20	30	13
0	1	50	40	60	50
0	2	50	50	60	53
0	3	50	50	70	57
0	4	50	50	70	57
0	5	60	50	70	60
0	6	60	50	70	60
0	7	60	50	70	60
0	8	60	50	70	60
0	9	60	50	70	60
0	10	60	50	70	60
0	11	60	50	70	60
0	12	60	50	80	63
0	13	60	50	80	63
0	14	60	50	80	63
1	15	40	20	50	37
2	16	20	0	40	20
3	17	0	-20	30	3
4	18	-10	-30	20	-7
5	19	-80	-70	-40	-63
6	20	-120	-70	-60	-83
7	21	-90	-100	-70	-87
8	22	-90	-100	-80	-90
9	23	-100	-110	-90	-100
10	24	-110	-120	-100	-110
11	25	-120	-130	-110	-120
12	26	-130	-130	-120	-127
13	27	-140	-140	-120	-133

TABLE C.7 (con't) : LENGTH-CHANGE MEASURENETS FOR SCM MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCM			
		A	B	C	Average
14	28	-160	-160	-130	-150
15	29	-160	-170	-130	-153
16	30	-160	-170	-140	-157
17	31	-160	-170	-140	-157
18	32	-160	-180	-150	-163
19	33	-180	-180	-150	-170
20	34	-160	-180	-170	-170
21	35	-160	-180	-160	-167
22	36	-150	-190	-160	-167
23	37	-160	-190	-170	-173
24	38	-170	-200	-170	-180
25	39	-180	-200	-180	-187
26	40	-190	-210	-180	-193
27	41	-190	-220	-200	-203
28	42	-200	-210	-190	-200
29	43	-200	-210	-200	-203
30	44	-200	-210	-210	-207
32	46	-210	-220	-220	-217
34	48	-220	-220	-230	-223
36	50	-230	-220	-230	-227
38	52	-240	-230	-240	-237
40	54	-240	-240	-250	-243
42	56	-260	-280	-250	-263
44	58	-270	-290	-260	-273
46	60	-280	-300	-270	-283
48	62	-270	-290	-270	-277
50	64	-270	-290	-260	-273
52	66	-270	-300	-260	-277
54	68	-280	-290	-260	-277
56	70	-280	-290	-270	-280
58	72	-270	-280	-290	-280
60	74	-270	-280	-270	-273
62	76	-280	-280	-270	-277
64	78	-280	-300	-270	-283

TABLE C.7 (con't) : LENGTH-CHANGE MEASURENETS FOR SCM MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCM			
		A	B	C	Average
66	80	-270	-300	-270	-280
68	82	-270	-300	-270	-280
70	84	-270	-300	-270	-280
72	86	-280	-300	-270	-283
74	88	-280	-300	-270	-283
76	90	-280	-300	-280	-287
78	92	-280	-300	-280	-287
80	94	-280	-300	-280	-287
82	96	-290	-300	-280	-290
84	98	-290	-300	-280	-290
86	100	-280	-300	-280	-287
88	102	-290	-300	-280	-290
90	104	-290	-300	-280	-290
92	106	-290	-300	-280	-290
94	108	-290	-300	-280	-290
101	115	-280	-290	-270	-280
108	122	-300	-310	-270	-293
115	129	-300	-320	-290	-303
122	136	-310	-330	-310	-317
129	143	-310	-330	-300	-313
136	150	-320	-330	-310	-320
143	157	-330	-320	-320	-323
150	164	-320	-330	-320	-323
157	171	-330	-340	-320	-330
164	178	-330	-340	-330	-333
171	185	-340	-350	-320	-337
178	192	-350	-360	-330	-347
180	194	-350	-360	-330	-347

TABLE C.8 : LENGTH-CHANGE MEASURENETS FOR IC-SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)		
		IC-SCA 2		
		A	C	Average
0	0	0	0	0
0	0.1	40	70	55
0	0.2	130	170	150
0	0.4	170	210	190
0	0.8	250	270	260
0	1	250	280	265
0	2	280	310	295
0	3	290	320	305
0	4	300	340	320
0	5	300	340	320
0	6	300	350	325
0	7	310	360	335
0	8	320	360	340
0	9	320	360	340
0	10	330	370	350
0	11	330	370	350
0	12	330	370	350
0	13	330	370	350
0	14	340	380	360
1	15	290	310	300
2	16	270	290	280
3	17	250	260	255
4	18	240	240	240
5	19	210	220	215
6	20	210	220	215
7	21	190	200	195
8	22	180	190	185
9	23	160	180	170
10	24	140	160	150
11	25	130	150	140
12	26	130	130	130

TABLE C.8 (con't) : LENGTH-CHANGE MEASURENETS FOR IC-SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)		
		IC-SCA 2		
		A	C	Average
13	27	120	130	125
14	28	100	110	105
15	29	90	100	95
16	30	80	90	85
17	31	60	70	65
18	32	50	60	55
19	33	50	60	55
20	34	50	70	60
21	35	40	50	45
22	36	30	40	35
23	37	20	30	25
24	38	10	20	15
25	39	0	20	10
26	40	-10	20	5
27	41	-10	20	5
28	42	-10	10	0
29	43	-10	0	-5
30	44	-10	0	-5
32	46	-20	0	-10
34	48	-40	-30	-35
36	50	-50	-30	-40
38	52	-60	-30	-45
40	54	-70	-40	-55
42	56	-70	-50	-60
44	58	-80	-50	-65
46	60	-80	-50	-65
48	62	-100	-70	-85
50	64	-100	-70	-85
52	66	-110	-80	-95
54	68	-110	-80	-95
56	70	-110	-80	-95
58	72	-110	-80	-95
60	74	-110	-80	-95
62	76	-120	-100	-110

TABLE C.8 (con't) : LENGTH-CHANGE MEASURENETS FOR IC-SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)		
		IC-SCA 2		
		A	C	Average
64	78	-120	-110	-115
66	80	-130	-110	-120
68	82	-130	-110	-120
70	84	-130	-110	-120
72	86	-140	-110	-125
74	88	-140	-120	-130
76	90	-140	-120	-130
78	92	-150	-130	-140
80	94	-150	-140	-145
82	96	-160	-140	-150
84	98	-160	-140	-150
86	100	-160	-140	-150
88	102	-160	-140	-150
90	104	-160	-140	-150
92	106	-160	-150	-155
94	108	-160	-150	-155
101	115	-160	-150	-155
108	122	-170	-160	-165
115	129	-180	-160	-170
122	136	-200	-170	-185
129	143	-210	-180	-195
136	150	-210	-180	-195
143	157	-230	-190	-210
150	164	-250	-200	-225
157	171	-260	-210	-235
164	178	-250	-210	-230
171	185	-260	-220	-240
178	192	-260	-220	-240
180	194	-260	-220	-240

TABLE C.9 : LENGTH-CHANGE MEASURENETS FOR IC-SCA 1 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC-SCA 1			
		A	B	C	Average
0	0.0	0	0	0	0
0	0.1	30	20	20	23
0	0.2	80	40	50	57
0	0.4	90	50	60	67
0	0.7	110	80	80	90
0	1.0	110	70	80	87
	1.0	111	71	81	88
0	2.0	130	100	100	110
0	3.0	150	110	110	123
0	4.0	150	140	120	137
0	5.0	160	140	130	143
0	6.0	170	140	140	150
0	7.0	180	150	150	160
0	8.0	190	160	160	170
0	9.0	200	170	170	180
0	10.0	210	180	180	190
0	11.0	210	180	180	190
0	12.0	210	200	190	200
0	13.0	220	200	200	207
0	14.0	220	220	220	220
1	15.0	180	180	170	177
2	16.0	170	170	160	167
3	17.0	160	160	150	157
4	18.0	150	150	150	150
5	19.0	140	160	140	147
6	20.0	140	150	140	143
7	21.0	140	140	140	140
8	22.0	130	140	130	133
9	23.0	120	130	120	123

TABLE C.9 (con't) : LENGTH-CHANGE MEASURENETS FOR IC-SCA 1 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC-SCA 1			
		A	B	C	Average
10	24.0	110	120	110	113
11	25.0	110	110	110	110
12	26.0	100	110	80	97
13	27.0	110	110	100	107
14	28.0	90	90	80	87
15	29.0	80	80	70	77
16	30.0	80	70	60	70
17	31.0	70	60	50	60
18	32.0	70	50	50	57
19	33.0	70	50	50	57
20	34.0	80	60	60	67
21	35.0	70	60	60	63
22	36.0	60	40	50	50
23	37.0	50	30	40	40
24	38.0	40	20	30	30
25	39.0	30	20	30	27
26	40.0	30	20	20	23
27	41.0	30	10	20	20
28	42.0	20	10	10	13
29	43.0	20	10	0	10
30	44.0	10	0	0	3
32	46.0	0	0	-10	-3
34	48.0	0	-10	-20	-10
36	50.0	-10	-20	-20	-17
38	52.0	-20	-30	-30	-27
40	54.0	-30	-40	-40	-37
42	56.0	-30	-50	-40	-40
44	58.0	-30	-60	-50	-47
46	60.0	-40	-70	-50	-53
48	62.0	-50	-60	-60	-57
50	64.0	-50	-70	-60	-60
52	66.0	-60	-80	-60	-67
54	68.0	-60	-90	-80	-77
56	70.0	-70	-90	-80	-80

TABLE C.9 (con't) : LENGTH-CHANGE MEASURENETS FOR IC-SCA 1 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC-SCA 1			
		A	B	C	Average
58	72.0	-70	-80	-80	-77
60	74.0	-60	-80	-80	-73
62	76.0	-60	-90	-80	-77
64	78.0	-60	-90	-80	-77
66	80.0	-70	-90	-80	-80
68	82.0	-80	-100	-80	-87
70	84.0	-80	-110	-80	-90
72	86.0	-90	-120	-90	-100
74	88.0	-90	-120	-90	-100
76	90.0	-90	-130	-100	-107
78	92.0	-90	-130	-110	-110
80	94.0	-90	-130	-110	-110
82	96.0	-100	-140	-120	-120
84	98.0	-100	-150	-120	-123
86	100.0	-110	-150	-120	-127
88	102.0	-120	-150	-120	-130
90	104.0	-120	-150	-120	-130
92	106.0	-130	-150	-130	-137
94	108.0	-130	-150	-130	-137
101	115.0	-130	-150	-130	-137
108	122.0	-130	-160	-140	-143
115	129.0	-140	-160	-140	-147
122	136.0	-170	-190	-150	-170
129	143.0	-170	-190	-160	-173
136	150.0	-170	-190	-150	-170
143	157.0	-170	-200	-170	-180
150	164.0	-180	-220	-180	-193
157	171.0	-200	-230	-190	-207
164	178.0	-200	-240	-190	-210
171	185.0	-210	-240	-200	-217
178	192.0	-210	-240	-190	-213
180	194.0	-210	-240	-190	-213

TABLE C.10 : LENGTH-CHANGE MEASURENETS FOR SCM-SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCM-SCA 2			
		A	B	C	Average
0	0.0	0	0	0	0
0	0.1	60	60	10	43
0	0.2	230	200	150	193
0	0.8	360	330	370	353
0	1.0	365	337	373	358
0	2.0	390	370	390	383
0	3.0	410	380	410	400
0	4.0	410	380	420	403
0	5.0	410	380	420	403
0	6.0	410	380	420	403
0	7.0	410	370	410	397
0	8.0	410	370	410	397
0	9.0	410	380	420	403
0	10.0	410	380	420	403
0	11.0	410	380	420	403
0	12.0	410	380	420	403
0	13.0	410	380	420	403
0	14.0	420	390	430	413
1	15.0	370	340	380	363
2	16.0	340	310	350	333
3	17.0	310	290	320	307
4	18.0	280	260	300	280
5	19.0	270	240	290	267
6	20.0	250	230	270	250
7	21.0	240	210	250	233
8	22.0	240	200	230	223
9	23.0	220	200	230	217
10	24.0	200	180	200	193
11	25.0	190	170	200	187
12	26.0	170	140	180	163

TABLE C.10 (con't) : LENGTH-CHANGE MEASURENETS FOR SCM-SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCM-SCA 2			
		A	B	C	Average
13	27.0	160	150	180	163
14	28.0	160	150	170	160
15	29.0	160	150	160	157
16	30.0	160	140	150	150
17	31.0	160	140	150	150
18	32.0	150	140	150	147
19	33.0	130	120	150	133
20	34.0	120	100	140	120
21	35.0	130	110	130	123
22	36.0	130	110	130	123
23	37.0	130	110	130	123
24	38.0	130	110	130	123
25	39.0	130	110	130	123
26	40.0	110	100	120	110
27	41.0	100	90	110	100
28	42.0	90	80	110	93
29	43.0	90	70	100	87
30	44.0	90	70	100	87
32	46.0	90	70	100	87
34	48.0	80	70	90	80
36	50.0	70	70	90	77
38	52.0	60	60	60	60
40	54.0	60	60	60	60
42	56.0	60	50	80	63
44	58.0	60	50	70	60
46	60.0	50	40	80	57
48	62.0	50	40	80	57
50	64.0	40	40	70	50
52	66.0	40	40	70	50
54	68.0	30	30	60	40
56	70.0	30	30	50	37
58	72.0	30	30	50	37
60	74.0	20	20	40	27
62	76.0	0	20	20	13

TABLE C.10 (con't) : LENGTH-CHANGE MEASURENETS FOR SCM-SCA 2 MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		SCM-SCA 2			
		A	B	C	Average
64	78.0	0	10	10	7
66	80.0	0	10	10	7
68	82.0	10	10	20	13
70	84.0	0	10	10	7
72	86.0	0	0	0	0
74	88.0	0	0	-10	-3
76	90.0	0	0	-10	-3
78	92.0	-20	0	-30	-17
80	94.0	-30	-10	-30	-23
82	96.0	-40	-10	-30	-27
84	98.0	-30	-10	-30	-23
86	100.0	-30	-10	-30	-23
88	102.0	-30	-20	-30	-27
90	104.0	-30	-20	-30	-27
92	106.0	-30	-20	-30	-27
94	108.0	-30	-30	-40	-33
101	115.0	-50	-40	-50	-47
108	122.0	-60	-50	-60	-57
115	129.0	-60	-50	-70	-60
122	136.0	-70	-60	-70	-67
129	143.0	-80	-70	-80	-77
136	150.0	-98	-80	-100	-93
143	157.0	-100	-80	-110	-97
150	164.0	-100	-90	-110	-100
157	171.0	-100	-80	-110	-97
164	178.0	-100	-80	-110	-97
171	185.0	-110	-90	-120	-107
178	192.0	-120	-80	-120	-107
180	194.0	-120	-80	-120	-107

TABLE C.11 : LENGTH-CHANGE MEASURENETS FOR IC MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC			
		A	B	C	Average
0	0	0	0	0	0
0	0.1	80	70	60	70
0	0.8	90	100	100	97
0	1	90	100	100	97
0	2	90	100	100	97
0	3	100	100	100	100
0	4	100	100	100	100
0	5	90	100	100	97
0	6	90	100	100	97
0	7	90	90	100	93
0	8	90	90	100	93
0	9	90	90	100	93
0	10	90	90	100	93
0	11	90	90	100	93
0	12	100	100	100	100
0	13	100	100	100	100
0	14	100	100	100	100
1	15	70	80	70	73
2	16	40	50	50	47
3	17	10	10	20	13
4	18	-20	-10	-10	-13
5	19	-30	-40	-40	-37
6	20	-60	-70	-40	-57
7	21	-70	-90	-60	-73
8	22	-80	-90	-80	-83
9	23	-100	-100	-90	-97
10	24	-110	-120	-110	-113
11	25	-130	-140	-130	-133
12	26	-150	-170	-150	-157
13	27	-150	-180	-150	-160

TABLE C.11 (con't) : LENGTH-CHANGE MEASURENETS FOR IC MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC			
		A	B	C	Average
14	28	-150	-190	-160	-167
15	29	-160	-200	-170	-177
16	30	-170	-200	-180	-183
17	31	-180	-200	-190	-190
18	32	-190	-200	-200	-197
19	33	-200	-220	-210	-210
20	34	-220	-230	-230	-227
21	35	-230	-230	-230	-230
22	36	-230	-230	-230	-230
23	37	-230	-240	-230	-233
24	38	-240	-250	-240	-243
25	39	-240	-260	-240	-247
26	40	-250	-280	-250	-260
27	41	-270	-280	-270	-273
28	42	-270	-290	-270	-277
29	43	-280	-300	-280	-287
30	44	-290	-310	-290	-297
32	46	-310	-320	-300	-310
34	48	-320	-330	-300	-317
36	50	-320	-330	-310	-320
38	52	-330	-350	-320	-333
40	54	-340	-340	-320	-333
42	56	-340	-340	-320	-333
44	58	-350	-340	-330	-340
46	60	-360	-350	-320	-343
48	62	-360	-350	-320	-343
50	64	-370	-350	-330	-350
52	66	-370	-350	-330	-350
54	68	-380	-360	-340	-360
56	70	-390	-370	-350	-370
58	72	-390	-370	-360	-373
60	74	-400	-380	-360	-380
62	76	-410	-410	-380	-400
64	78	-400	-410	-390	-400

TABLE C.11 (con't) : LENGTH-CHANGE MEASURENETS FOR IC MIXTURE USED IN CHAPTER 2

Days After Drying	Days After Cast	Shrinkage (microstrain)			
		IC			
		A	B	C	Average
66	80	-410	-410	-390	-403
68	82	-410	-410	-390	-403
70	84	-410	-420	-400	-410
72	86	-430	-430	-400	-420
74	88	-440	-440	-410	-430
76	90	-440	-450	-420	-437
78	92	-450	-460	-430	-447
80	94	-450	-460	-440	-450
82	96	-460	-460	-450	-457
84	98	-470	-460	-440	-457
86	100	-470	-460	-440	-457
88	102	-470	-460	-450	-460
90	104	-470	-460	-450	-460
92	106	-470	-460	-440	-457
94	108	-480	-460	-440	-460
101	115	-500	-480	-450	-477
108	122	-500	-480	-450	-477
115	129	-500	-480	-450	-477
122	136	-520	-490	-470	-493
129	143	-510	-500	-470	-493
136	150	-520	-520	-480	-507
143	157	-520	-520	-500	-513
150	164	-530	-520	-500	-517
157	171	-530	-520	-500	-517
164	178	-530	-520	-510	-520
171	185	-530	-530	-520	-527
178	192	-540	-520	-520	-527
180	194	-540	-520	-520	-527

APPENDIX D: CRACK SURVEY RESULTS FOR CONV, LC-HPC, AND EXTRA CONTROL BRIDGES AND FILED INFORMATION FOR CHAPTER 4 BRIDGES

TABLE D.1 : INDIVIDUAL CRACK SURVEY RESULTS FOR CONV BRIDGES

Bridge Number	Bridge & Placement	Age (Months)	Crack Density (m/m^2)
3-046	East Deck	210	0.418
		102	0.402
	West Deck	210	0.539
		102	0.362
	Ctr. Deck	210	0.334
		102	0.153
75-044	Deck	155	0.304
		48	0.179
75-045	Deck	154	0.433
		47	0.468
89-204	Deck	132	0.926
		34	0.732
3-045	West Deck	223	0.192
		112	0.122
	East Deck	223	0.368
		112	0.196
	W. Ctr. Deck	223	0.203
		112	0.188
	Ctr. Deck	220	0.273
		112	0.215
	E. Ctr. Deck	220	0.333
		112	0.163
56-142	+ Moment	189	0.200
		80	0.108
	N. Pier	188	0.163
		80	0.093
56-148	Deck	133	0.487
		36	0.259
70-095	Deck	212	0.136
		106	0.069
70-103	Right	219	0.647
		102	0.395
	Left	219	0.852
		102	0.557
70-104	Deck	212	0.104
		106	0.083

TABLE D.1 (con't) : INDIVIDUAL CRACK SURVEY RESULTS FOR CONV BRIDGES

Bridge Number	Bridge & Placement	Age (Months)	Crack Density (m/m^2)
70-107	**Deck**	130	0.702
		34	0.322
99-076	**Placement 4**	163	1.022
		42	0.872
	Placement 5	163	1.052
		42	0.861
	North (West Ln.)	161	0.947
		42	0.801
	North (East Ln.)	157	0.663
		42	0.412
	Placement 2	165	1.006
		42	1.536
	Placement 3	164	0.881
		42	0.739
89-208	**Deck**	73	0.106
		36	0.009

TABLE D.2 : INDIVIDUAL CRACK SURVEY RESULTS FOR LC-HPC AND EXTRA CONTROL BRIDGES

Bridge Number	Bridge & Placement	Age (Months)	Crack Density (m/m^2)
105-034	LC-HPC 1-p1	6	0.012
		18	0.047
		32	0.044
		44	0.060
		56	0.032
		68	0.045
		71	0.061
		79	0.096
		91	0.043
		103	0.043
		115	0.037
	LC-HPC 1-p2	5	0.003
		18	0.006
		31	0.024
		43	0.125
		55	0.023
		67	0.034
		70	0.103
		78	0.081
		91	0.040
		103	0.024
		115	0.055
105-310	LC-HPC 2	7	0.013
		21	0.028
		32	0.085
		44	0.059
		59	0.143
		68	0.197
		80	0.151
		92	0.116
		104	0.220

TABLE D.2 (con't) : INDIVIDUAL CRACK SURVEY RESULTS FOR LC-HPC AND EXTRA CONTROL BRIDGES

Bridge Number	Bridge & Placement	Age (Months)	Crack Density (m/m^2)
46-338	**LC-HPC 3**	7	0.028
		19	0.110
		31	0.108
		43	0.315
		54	0.173
		66	0.174
		79	0.660
		92	0.487
		105	0.453
46-339	**LC-HPC 4-p2**	9	0.004
		21	0.079
		33	0.094
		45	0.080
		56	0.092
		68	0.147
		80	0.173
		93	0.181
46-340 # 1	**LC-HPC 5**	8	0.059
		19	0.123
		31	0.128
		43	0.190
		54	0.158
		67	0.140
		79	0.229
		92	0.247
46-340 # 2	**LC-HPC 6**	7	0.063
		20	0.238
		31	0.231
		43	0.336
		55	0.362
		67	0.303
		80	0.356
		92	0.386

TABLE D.2 (con't) : INDIVIDUAL CRACK SURVEY RESULTS FOR LC-HPC AND EXTRA CONTROL BRIDGES

Bridge Number	Bridge & Placement	Age (Months)	Crack Density (m/m^2)
43-33	LC-HPC 7	11	0.003
		24	0.019
		35	0.012
		47	0.005
		59	0.055
		71	0.065
		83	0.077
		96	0.087
		107	0.036
54-57	LC-HPC 9	14	0.130
		26	0.248
		38	0.344
		49	0.305
		62	0.454
		74	0.430
		115	0.485
78-119	LC-HPC 11 (North Lane)	23	0.060
		36	0.165
		48	0.269
		61	0.218
		72	0.358
		85	0.482
		111	0.504
46-351	LC-HPC 15	19	0.211
		31	0.161
		43	0.317
		56	0.299
		78	0.293
		95	0.360
46-352	LC-HPC 16	8	0.092
		19	0.249
		31	0.211
		44	0.311
		55	0.397
		79	0.356
		95	0.420

TABLE D.2 (con't) : INDIVIDUAL CRACK SURVEY RESULTS FOR LC-HPC AND EXTRA CONTROL BRIDGES

Bridge Number	Bridge & Placement	Age (Months)	Crack Density (m/m^2)
46-373	LC-HPC 17	9	0.226
		21	0.243
		33	0.274
		46	0.308
		68	0.327
		84	0.374
56-49	Extra Control	12	0.077
		26	0.230
		37	0.219
		48	0.265
		61	0.316
		73	0.358
		85	0.395
		109	0.401

TABLE D.3 : INDIVIDUAL CRACK SURVEY RESULTS FOR HARTFORT, RAILROAD AND TOPEKA BRIDGES

Bridge Number	Bridge & Placement	Age (Months)	Crack Density (m/m^2)
297	**Topeka Fiber 1**	16	0.157
		27	0.272
		38	0.287
		51	0.394
283	**Topeka Fiber 2 p1**	12	0.608
		24	0.645
		34	0.713
282	**Topeka Control p1**	14	0.739
		27	0.725
		36	0.766
282	**Topeka Control p2**	14	0.395
		27	0.411
		36	0.393
283	**Topeka Fiber 2 p2**	12	0.173
		24	0.300
		34	0.429
46-363	**OP-14 p1 (LC-HPC 14 p1)**	18	0.341
		30	0.502
		42	0.585
		67	1.000
46-364	**OP-14 p2 (LC-HPC 14 p2)**	14	0.640
		26	0.727
		38	1.304
		62	1.336
46-365	**OP-14 p3 (LC-HPC 14 p3)**	13	0.421
		25	0.871
		37	0.678
		62	1.395

TABLE D.3 (con't): INDIVIDUAL CRACK SURVEY RESULTS FOR HARTFORT, RAILROAD AND TOPEKA BRIDGES

Bridge Number	Bridge & Placement	Age (Months)	Crack Density (m/m^2)
54-66	Railroad Bridge (LC-HPC 13)	14	0.050
		25	0.129
		37	0.364
		49	0.342
		63	0.576
		75	0.471
		86	0.486
56-57	Hartford Bridge p1 (LC-HPC 12 p1)	16	0.271
		27	0.256
		39	0.313
		50	0.450
		62	0.478
		76	0.789
56-57	Hartford Bridge p2 (LC-HPC 12 p2)	5	0.254
		15	0.244
		27	0.268
		38	0.375
		51	0.381
		65	0.540

TABLE D.4 : WIND SPEED, RELATIVE HUMIDITY, AND AIR TEMP INFORMATION FOR CHAPTER 4 BRIDGES

Bridge Name	Wind Speed (mph)		Relative Humidity (%)		Air Temp (°F)	
	Low	High	Low	High	Low	High
LC-HPC-1 p1	1.5	2.0	72.0	74	52	59
LC-HPC-1 p2	4.0	8.0	52.0	68	52	72
LC-HPC-2	0.6	0.9	67.0	87	56	70
LC-HPC-3	0.0	1.0	51.0	72	43	54
LC-HPC-4-p2	1.0	1.0	69.0	72	66	67
LC-HPC-5	1.0	3.0	38.0	64	54	56
LC-HPC-6	1.0	2.0	46.0	73	36	55
LC-HPC-7	0.8	3.1	72.0	81	70	71
LC-HPC-9	0.0	7.3	35.0	51	56	72
LC-HPC-11	1.0	11.9	48.0	66	57	72
LC-HPC 15	0.3	4.2	44.7	71	58	71
LC-HPC 16	0.0	3.1	15.4	37	49	63
LC-HPC 17	0.0	1.1	22.6	90	57	96
Hartford P1 (LC-HPC-12 p1)	0.5	2.5	47.0	77	44	63
Hartford P2 (LC-HPC-12 p2)	5.3	16.0	28.5	76	53	65
Railroad (LC-HPC-13)	0.7	4.0	23.0	39	59	69
OP-14 p1 (LC-HPC-14-p1)	0.7	2.5	38.0	70	37	57
OP-14 p2 (LC-HPC-14-p2)	5.0	10.0	32.0	78	58	65
OP-14 p3 (LC-HPC-14-p3)	0.0	8.0	23.0	51	67	79
Topeka Fiber 1	0	13	19	77	40	78
Topeka Fiber 2 p1	0.6	2.6	62	89	73	88
Topeka Fiber 2 p2	1.2	5.9	72	86	71	77
Topeka Control p1	Date Not Recorded					
Topeka Control p2						
US-59 9						
US-59 10						
US-59 11						
US-59 12						

APPENDIX E: CRACK SURVEY (CRACK DENSITY AND WIDTH) RESULTS FOR CHAPTER 5 BRIDGES

TABLE E.1: INDIVIDUAL CRACK SURVEY RESULTS FOR US 59 BRIDGES

Bridge Number	Bridge Name	Age (Months)	Crack Density (m/m^2)
59-30-19.92	**US 59 1**	22	0.280
		31	0.385
		45	0.403
		55	0.417
		67	0.555
		90	0.543
59-30-19.91	**US 59 2**	22	0.140
		32	0.217
		46	0.306
		58	0.306
		70	0.373
		92	0.383
59-30-20.05	**US 59 3**	23	0.035
		32	0.051
		46	0.070
		57	0.117
		70	0.088
		81	0.056
		92	0.091
59-30-20.04	**US 59 4**	23	0.067
		33	0.056
		46	0.082
		57	0.145
		70	0.115
		82	0.153
		92	0.108
59-30-21.84	**US 59 5**	28	0.270
		38	0.320
		46	0.465
		61	0.484
		75	0.633
		99	0.559

TABLE E.1 (con't): INDIVIDUAL CRACK SURVEY RESULTS FOR US 59 BRIDGES

Bridge Number	Bridge Name	Age (Months)	Crack Density (m/m^2)
59-30-21.85	**US 59 6**	29	0.160
		39	0.198
		51	0.273
		62	0.298
		75	0.395
		100	0.412
59-30-18.76	**US 59 7**	31	0.010
		45	0.019
		56	0.046
		68	0.036
		81	0.031
		91	0.023
59-30-18.75	**US 59 8**	33	0.039
		45	0.049
		56	0.065
		69	0.073
		80	0.081
		93	0.064
59-30-24.51	**US 59 9**	33	0.719
		45	0.853
		81	0.838
		93	0.832
59-30-24.50	**US 59 10**	31	0.150
		43	0.217
		81	0.411
		93	0.372
59-30-24.82	**US 59 11**	33	0.213
		46	0.225
		83	0.314
		92	0.285
59-30-24.83	**US 59 12**	30	0.022
		43	0.075
		80	0.086
		89	0.104

TABLE E.2: INDIVIDUAL CRACK WIDTH MEASUREMENTS FOR US 59 BRIDGES

Individual Crack Width Values (10^{-3} in.)										
US-59 1	**US-59 2**	**US-59 3**	**US-59 4**	**US-59 5**	**US-59 6**	**US-59 7**	**US-59 8**	**US-59 9**	**US-59 10**	**US-59 12**
6	9	7	5	7	4	9	4	4	4	9
6	7	12	7	9	6	4	7	4	6	4
6	7	7	4	16	7	9	4	7	4	7
6	7	12	5	9	9	6	6	4	4	12
10	7	6	7	6	4	4	9	9	6	9
7	10	7	5	9	4	4	4	9	4	7
7	5	6	5	7	6	4	6	9	4	9
9	4	6	4	12	7	4	4	6	4	6
7	4	4	4	6	9	6	6	9	9	6
6	9	4	5	16	6	6	4	10	4	7
6	5	9	4	12	7	3	4	12	4	7
4	9	7	4	16	7	4	4	6	7	9
6	5	9	5	16	6	4	4	4	4	6
7	10	7	5	10	7	4	4	9	6	6
4	5	6	5	10	12	4	6	10	6	9
4	5	6	7	6	6	4	6	16	9	9
6	10	4	6	7	6	6	6	10	6	9
6	9	9	5	7	7	4	9	9	9	4
9	10	4	4	9	4	7	4	7	6	6
7	10	6	5	16	6	4	4	7	6	9
8	10	6	5	12	6	4	6	7	4	9
4	5	6	5	12	4	4	6	16	6	7
10	7	6	6	12	9	3	6	9	4	7
16	9	9	5	16	6	7	4	12	6	10
9	7	7	5	10	9	7	4	6	7	6
7	7	6	7	10	4	6	6	6	4	7
6	5	12	7	12	7	4	4	16	6	4
7	7	7	5	16	4	4	4	21	7	4
9	10	6	5	6	6	7	4	21	10	10
10	13	10	7	16	6	10	3	10	6	7
4	5	6	7	16	4	6	3	6	7	7

TABLE E.2 (con't): INDIVIDUAL CRACK WIDTH MEASUREMENTS FOR US 59 BRIDGES

Individual Crack Width Values (10^{-3} in.)										
US-59 1	**US-59 2**	**US-59 3**	**US-59 4**	**US-59 5**	**US-59 6**	**US-59 7**	**US-59 8**	**US-59 9**	**US-59 10**	**US-59 12**
20	16	7	10	16	9	4	4	9	6	10
12	9	6	5	16	6	4	9	10	6	7
25	9	7	9	20	12	4	4	9	6	7
25	10	10	10	10	12	7	6	16	4	7
12	10	7	9	10	7	6	6	6	6	6
25	10	7	5	6	6		7	10	4	
6	7	4	4	12	12		7	7	4	
25	5	6	5	10	6		6	10	4	
25	4	6	7	9	4		3	6	6	
20	9	7	4	10	9		4	4	10	
40	7	6	5	10	6		4	7	6	
12	9	9	7	7	9		6	6	4	
12	10	6	7	9	10		9	4	9	
25	10	9	5	12	4		6	6	4	
30	4	9	5	16	6		7	9	4	
35	5	7	5	16	4		6	6	4	
20	17	9	7	9	6		6	6	4	
20	7	7	7	10	9		4	9	4	
16	7	9	7	10	12		4	6	4	
12	7	7	5	7	7		4	6	9	
12	16	6		6	9		16	6	6	
16	7	6		6	7			10	4	
16	7			12	10			6	4	
20	10			10	9			6	9	
9	10			10	7			7	7	
16	7			10	6			6	4	
12	10			9	6			4	4	
12	7			16	6			16	6	
10	13			16	6			12	4	
9	9			7	4			25	4	
7	16			10	4			9	4	
16	9			12	6			10	6	
10	10			10	12			10	7	
12	10			7	6			7	7	
10	5			9	4			12	6	
7	7			10	10			6	6	

TABLE E.2 (con't): INDIVIDUAL CRACK WIDTH MEASUREMENTS FOR US 59 BRIDGES

Individual Crack Width Values (10^{-3} in.)										
US-59 1	**US-59 2**	**US-59 3**	**US-59 4**	**US-59 5**	**US-59 6**	**US-59 7**	**US-59 8**	**US-59 9**	**US-59 10**	**US-59 12**
10	7			16	9			6	6	
12	5			12	9			10	4	
12	5			9	4			6	6	
9	10			9	6			6	6	
25	5			9	4			4	6	
30	16			12	7			6	7	
7	5			9	4			6	7	
7	9			25	7			6	6	
10	7			12	9			10	4	
12	5			12	7			9	4	
12	7			10	6			12	9	
20	7			12	4			10	7	
20	13			12	9			9	4	
6	7			16	9			10	4	
20	9			9	6			10	6	
12	5			16	4			16	4	
9	5			12	9			10	4	
35	7			12	4			12	6	
7	5			9	7			6	9	
6	7			12	7			12	4	
16	5			6	7			10	4	
12	6			10	9			6	6	
20	10			12	7			10	6	
25	10			9	7			6	6	
7	5			12	7			6	4	
7	13			12	7			9	6	
12	9				7			6	6	
16	9				7			10	6	
6	9				6			9	6	
7	5				7			6	6	
9	10				12			6	4	
9	9				6			6	4	
10	5				4			6	6	
9	5				12			10	9	
30	5				10			6	4	
4	5				10			7	6	

TABLE E.2 (con't): INDIVIDUAL CRACK WIDTH MEASUREMENTS FOR US 59 BRIDGES

Individual Crack Width Values (10^{-3} in.)										
US-59 1	**US-59 2**	**US-59 3**	**US-59 4**	**US-59 5**	**US-59 6**	**US-59 7**	**US-59 8**	**US-59 9**	**US-59 10**	**US-59 12**
20	7				10			6	4	
25	10				6			6	6	
12	10				4			6	9	
7	7				10			9	7	
7	9				6			7	9	
10	7				7			7	7	
9	7				4			9	4	
12	16				12			6	6	
6	10				6			16	7	
7	7				6			12	4	
12	10				12			10	4	
16	10				12			9	6	
12	10				12			9	6	
10	7				6			9	6	
10	7				6			9	6	
12	5				4			9	6	
12	7				6			9	4	
16	5				4			9	7	
7	7				6			7	6	
6	10				4			7	7	
7	9				7			10	9	
	4				6			16	6	
	7				4			7	6	
	7				7			12	6	
	5				6			6	7	
	7				7			12	6	
	9				6			12	6	
	10				10			9	6	
	10				12			9	6	
	9				4			6	6	
	7				4			4	6	
					6			9	4	
					10			10	6	
					9			7	4	

TABLE E.2 (con't): INDIVIDUAL CRACK WIDTH MEASUREMENTS FOR US 59 BRIDGES

Individual Crack Width Values (10^{-3} in.)										
US-59 1	**US-59 2**	**US-59 3**	**US-59 4**	**US-59 5**	**US-59 6**	**US-59 7**	**US-59 8**	**US-59 9**	**US-59 10**	**US-59 12**
					6			4	4	
					7			12	6	
					9			9	4	
					4			4	4	
					6			12	4	
					4			6	6	
					9			12	6	
					9			9	7	
					4				12	
					6				6	
					6				4	
					9				4	
					12				4	
					6				4	