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Guide for Concrete Highway Bridge Deck Construction

Reported by ACI Committee 345

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Guide for Concrete Highway Bridge Deck Construction

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Guide for Concrete Highway Bridge Deck Construction

Reported by ACI Committee 345

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The service-life performance of concrete bridge decks, including maintenance, repair, and rehabilitation needs, is directly related to the care exercised from the preconstruction through post-construction period. This guide provides recommendations for bridge deck construction based on considerations of durability, concrete materials, reinforcement, placing, finishing and curing, and overlays.

Keywords: admixtures; aggregate; air entrainment; bridge decks; concrete curing; concrete finishing; concrete overlays; concrete placing; cracking; durability; polymer concrete; reinforcing bars; scaling; shrinkage; skid resistance.

CONTENTS

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

The deck of a highway bridge serves both structural and functional purposes for the structure. As a structural component, it provides the load path to safely transfer forces from wheel loads to the supporting superstructure and substructure elements. It may also contribute, through composite action, to the performance of primary superstructure components. Equally, the construction and condition of a deck directly impacts serviceability or the ability of the structure to safely and efficiently carry highway traffic by providing smoothness, skid resistance, and resistance to deflections under wheel loads. The riding surface of a highway bridge deck should provide a continuation of the pavement segments that it connects. The surface should be free from characteristics or profile deviations that impart objectionable or unsafe riding qualities. The desirable qualities should persist with minimum maintenance throughout the projected service life of the structure.

Roughness, cracking, spalling, scaling, and poor skid resistance are defects that result when the many details that influence their occurrence are not given sufficient attention. Recognition of the interaction of design, materials, and construction practices, as well as environmental factors, is the important first step in achieving smooth and durable decks.

Many decks remain smooth and free from surface deterioration and retain skid resistance for many years. When deficiencies occur, they usually take one of the forms described in this guide. The contribution of various aspects of deck construction to defects is discussed and guidelines based on theory and experience presented that should reduce the probabilities of occurrence to acceptable levels.

1.2—Scope

This guide presents considerations to take in the design and a summary of construction practices for conventionally reinforced concrete highway bridge decks. Such decks are typically supported by multiple simple- or continuous-span steel or prestressed concrete girders, or integral reinforced concrete members. The service-life performance of concrete bridge decks, including maintenance, repair, and rehabilitation needs, is directly related to the care exercised from preconstruction through the post-construction period. Recommendations are presented for design and durability considerations, concrete materials, reinforcement, placing, finishing and curing, and the use of overlays.

Although some performance and durability factors discussed may be applicable, design and construction of prestressed bridge decks are presently beyond the scope of this guide. Thus, prestressing steel is not included in the reinforcement section. Guidance for the design of prestressed bridge decks is being developed elsewhere (Swartz and Schokker 2008).

CHAPTER 2—DEFINITIONS

2.1—Definitions

ACI provides a comprehensive list of definitions through an online resource, "ACI Concrete Terminology," at http://terminology.concrete.org. Definitions provided herein complement that resource.

crack, reflective—a crack that forms in a bonded overlay or wearing course caused by upward extension of moving crack or joint in the substrate.

washboarding—undulations in the finished surface of a deck that cause vibrations of undesirable frequency and amplitude in passing vehicles.

CHAPTER 3—DESIGN AND DURABILITY CONSIDERATIONS

3.1—General

Chapter 3 emphasizes design factors that may affect the resistance of a bridge deck to chemical and environmental exposure, including potential for freezing and thawing, deleterious chemical reactions with concrete constituents, or chloride-induced corrosion damage. The design considerations of this chapter are not concerned with the structural analysis of the bridge deck. Structural aspects of the design, however, can have implications in the development of internal stresses and subsequent cracking in bridge decks, which may negatively impact durability. The items discussed in this chapter are generally within the purview of the bridge designer, and should receive due consideration.

3.2—Concrete and reinforcement materials

Although the specific topics of material selection for concrete mixture proportioning and bridge deck reinforcement are covered in greater detail in Chapters 4 and 5, respectively, it is important to emphasize the influence of material selection during the design process on the long-term durability of a bridge deck. Most modern bridge deck designs generally employ some strategy for deterring corrosion and enhancing exposure-related durability. These may include the use of epoxy-coated, galvanized, or metallic-clad reinforcement; alternative reinforcement materials such as various grades of stainless steel, specialized steel alloy formulations; or fiber-reinforced polymer (FRP) reinforcement.

The use of better-quality concrete mixtures has gained favor, either separately from, or in conjunction with, alternative reinforcement strategies. Such strategies may include minimizing the water-cementitious material ratio (*w/cm*) of a concrete mixture or the use of mineral admixtures, such as fly ash, silica fume, slag cement, or metakaolin, to reduce permeability characteristics of the concrete. Many other admixtures are commercially available to address workability and placement characteristics, resistance to freezing and thawing, and increased corrosion resistance. Other products are available to reduce susceptibility to plastic and drying shrinkage.

Careful consideration should be given to the selection of deck materials. One common myth is that compressive strength is the single most important factor in specifying quality deck concrete. In fact, concrete bridge decks composed of concrete with excessively high compressive strength tend to be less flexible, have greater shrinkage potential, and have less ability to redistribute load and thermal- or shrinkage-induced strains. The result is a greater tendency toward cracking, which leads to premature deterioration from the ingress of moisture and aggressive chemicals, such as deicing salts. Recently, many agencies have considered performance-based specifications that rely more on measures of permeability than strength as criteria for acceptance.

Alternatively, reinforcing materials such as FRP bars, which are not affected by chlorides, can be considered viable alternatives to ferrous reinforcing bars. The use of FRP bars is governed by the American Association of State Highway Transportation Officials (AASHTO) LRFD design guidelines (AASHTO 1998) and by the Canadian Highway Bridge Design Code (CAN/CSA-S6-06) (Canadian Standards Association 2006).

3.3—Positive protective systems

3.3.1 *Overlays*—The common forms of bridge deck deterioration, such as scaling, some types of cracking, and surface spalling, generally occur within the top 2 in. (50 mm) of a deck. Improper concrete placing and finishing practices often result in a lower-quality concrete in this area. Because it is subjected to the most severe exposure and service conditions, the top portion of the deck slab should have the best possible concrete quality. Consideration should be given to placing an overlay on the bridge deck when it is constructed. Many different types of overlays have been used successfully. Chapter 7 discusses several types of overlays in detail.

3.3.2 *Other positive protective systems*—Because of the high cost of repairing corrosion-induced damage, several different protective systems are being used for bridge decks in severe deicing salt areas and for structures in marine environments. Other systems used to enhance durability or protect decks, some of which have been mentioned already, may include:

1. High-performance concretes that employ fly ash, silica fume, and slag cement as mineral additives for reduced permeability and protection against sulfate attack and alkali-silica reaction (ASR);
2. Shrinkage-compensating cements or shrinkage-reducing admixtures (SRA) in concrete for crack reduction;
3. Calcium nitrite, or other (anodic) corrosion-inhibiting admixtures, for increasing the threshold value of chloride concentration required for corrosion;
4. Waterproofing membranes with or without a bituminous concrete wearing surface for protection against chloride ion penetration;
5. Passive-current or impressed-current cathodic protection for preventing corrosion and stopping the corrosion of active systems;
6. Reinforcing steel coatings or cladding such as galvanizing, fusion-bonded epoxy, and stainless steel for extending the time to corrosion damage; and
7. Alternative reinforcing materials, such as solid stainless steel and nonmetallic FRPs, for extending the corrosion-resistant service life.

The performance of several different protection systems was evaluated as part of Strategic Highway Research Program (SHRP) and Federal Highway Administration (FHWA) studies (Pfeifer et al. 1987; Bennett et al. 1993; Weyers et al. 1993). As noted previously, selection of appropriate concrete mixtures is discussed in detail in Chapter 4. The ability of various types of reinforcement to resist corrosion is discussed in Chapter 5.

3.4—Arrangement and cover of reinforcement

3.4.1 In the most common type of bridge deck—the slab-on-beam bridge using a 7 to 9 in. (175 to 230 mm) thick slab spanning between longitudinal girders—the primary reinforcement is placed transverse to the girders. To use this reinforcement most effectively from a structural point of view, practice places the reinforcement closest to the top and bottom slab surfaces. The “AASHTO Standard Specifications for Highway Bridges” (AASHTO 1998) provides simple empirical equations to represent the Westergaard analysis of bridge deck behavior. The primary reinforcement is selected on the basis of one-way slab action and pure flexure. Shear, bond, and fatigue are not considered in the procedure. None of the bridge deck durability studies have

Fig. 3.4.3—Transverse cracking (from below).

indicated any structural deficiencies in the deck design procedure with the level of stresses generally permitted. Further, the AASHTO document "AASHTO LRFD Bridge Design Specifications (5th Edition) with 2010 Interims" (AASHTO 2010) permits design of concrete decks using an empirical method or by traditional method, based on flexure. Included commentary indicates that slabs have been found to resist concentrated wheel loads via a complex internal membrane stress (internal arching) rather than pure flexure. Traditional flexural or empirical design methods are stated to have high (8.0 to 10.0 or greater) factors of safety. The empirical method is governed by requirement for composite action and minimum overhang, as well as constraints on effective span, total slab and core depth, and material strengths, though the method specifically excludes the design of the cantilever or overhang components. To support the traditional design method, an appendix provides simplified tabulation of design moments for different girder arrangements. Moments are calculated using the equivalent strip method as applied to concrete slabs supported on three or more parallel girders. The moment values account for multiple [vehicle] presence factors and dynamic load allowance. In most designs, the primary slab reinforcement generally consists of No. 5 or 6 (No. 16 or 19) bars placed from about 5 to 9 in. (125 to 230 mm) on center.

3.4.2 Distribution reinforcement, generally consisting of No. 4 or 5 (No. 13 or 16) bars, is placed transverse to the primary reinforcement to provide for the two-way behavior of the deck. The amount of distribution reinforcement is determined as a percentage of the primary reinforcement, with more being placed in the middle half of the slab span than over the beams.

3.4.3 Shrinkage and temperature reinforcement is placed transverse to the primary reinforcement near the top of the slab to control potential cracking resulting from drying shrinkage and temperature changes in the concrete. In current practice, No. 4 or 5 (No. 13 or 16) bars are spaced from 12 to 18 in. (300 to 450 mm) on center and placed underneath the top primary slab reinforcement. Transverse cracks, the most common kinds of cracks found in bridge decks, especially if simply-supported, tend to form parallel to, and directly over, the top primary reinforcing bars, exposing them to attack from chlorides, moisture, and air (Fig. 3.4.3). Furthermore, the tensile stresses caused by drying shrinkage are not uniform through the depth of a concrete slab, but are largest near the drying faces. Because flexural strength is not generally the dominant factor in reinforced concrete deck design, a more effective way to control or reduce the widths of drying shrinkage cracking is to place the shrinkage and temperature reinforcement in a more strategic location, which is above the primary slab reinforcement while providing minimum 2 in. (50 mm) clear cover.

3.4.4 Prestressed box beam bridges generally experience reduced tendencies toward transverse cracking because of their stiffness. Adjacent box beam superstructures with no space between the beams, however, often have thin, non-reinforced decks that frequently exhibit undesirable longitudinal reflection cracks over the joints between adjacent beams. One solution is to post-tension the beams together transversely and use a reinforced concrete deck on top.

3.4.5 A most important consideration for durability in bridge deck design is the thickness of protective concrete cover over the top reinforcement. It is recommended that 2 in. (50 mm) of concrete, measured from top of bar, be the minimum specified protective cover over the uppermost reinforcement in bridge decks, with provisions for variability during placement (Pfeifer et al. 1987). AASHTO (2010) requires minimum 2.5 in. (65 mm) clear cover for decks exposed to deicing or subject to tire stud or chain wear and 3.0 in. (75 mm) in coastal exposures. ACI 117-10 and the discussion on reinforcement in this guide provide recommended construction tolerances. Spalling generally occurs readily on decks having inadequate cover over the bars. Similar requirements for top, bottom, and side faces for reinforcing bar cover should be considered for highly corrosive environments. It should be recognized, however, that specified cover depths are to be a function of in-place concrete properties, intended service life, and loading and environmental conditions.

Deviations from the specified cover should be expected to occur in construction. The designer should try to anticipate conditions that could make accurate reinforcing bar placement difficult, or where the desired concrete surface might be undercut by the action of the strikeoff, as at non-uniform sections of complicated geometrical transitions, and compensate with an increased cover requirement. Furthermore, field investigations have documented that clear cover depths in cast-in-place bridge decks vary consistently, even under favorable conditions, with a standard deviation of approximately 3/8 in. (10 mm) (Weyers et al. 2003).

When FRP bars are used, issues of concrete cover and crack widths are less critical. For GFRP reinforcing bars, minimum concrete cover is dictated by issues of potential reflective cracking due to differences in transverse thermal expansion with the surrounding concrete. A concrete clear cover of only two bar diameters is sufficient to avoid this phenomenon. Further consideration needs to be made to future rehabilitation, and milling of the concrete wearing surface.

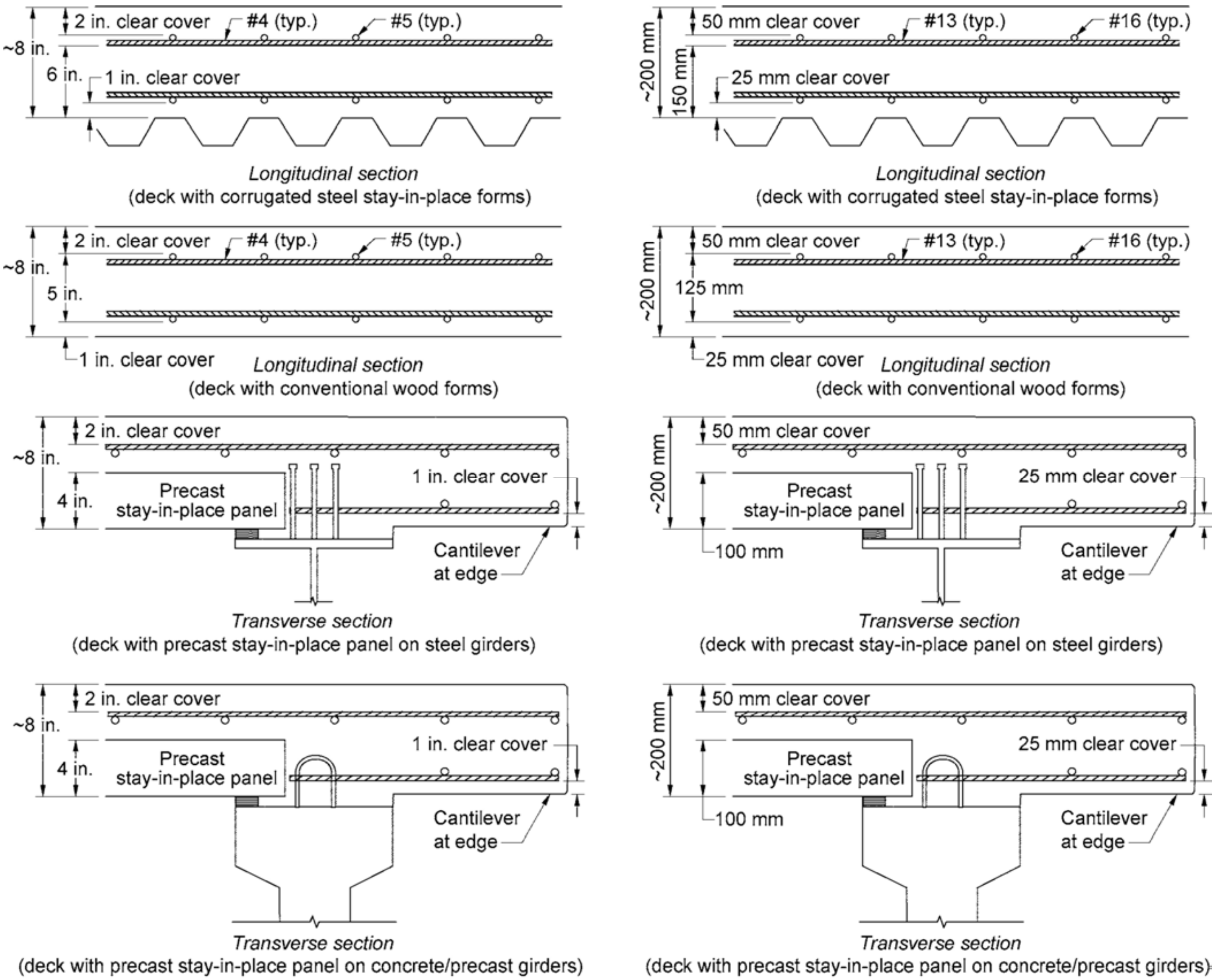

Fig. 3.5.1—Sections of several common reinforced concrete bridge configurations—U.S. Customary units (left) and SI units (right).

3.5—Deck thickness

3.5.1 Bridge design agencies usually establish standard details specifying deck thickness and reinforcement arrangement for different bridge deck spans. A nominal minimum deck thickness of 7 in. (175 mm) is recommended (AASHTO 1998). More commonly, bridge decks range from 8 to 9 in. (200 to 230 mm) in thickness, depending upon the particular deck system and application (Fig. 3.5.1).

3.5.2 The high quality of deck concrete that is needed to achieve durability usually results in much higher concrete strengths than needed for the structural capacity of the deck. The advent of higher strength grades of reinforcement also necessitates a reevaluation of established standard details. The temptation exists to use thinner deck slabs to use these materials more efficiently. While there is no direct evidence that deterioration is more likely to occur in thinner, more flexible decks than in thicker, stiffer decks, there is evidence that once deterioration has begun, it is likely to progress more rapidly in thinner decks due to less deck mass, which increases the amplitude movement during traffic (Ramey 2001). Thinner decks also result in greater congestion of reinforcement because the two layers of reinforcement are closer together. Poor consolidation of concrete can occur in areas of congested reinforcement.

3.5.3 As with all construction, tolerances should be allowed in design dimensions to ensure achieving all critical minimum values. Reports confirm that the placing of top deck reinforcement often varies widely. Average cover was found to be typically equal to the design or plan cover, with a standard deviation of approximately 0.3 in. (8 mm) (Weyers et al. 1994, 2003; Bergren and Brown 1975). Thus, to ensure that 97% of the reinforcement has at least the minimum 2 in. (50 mm) cover recommended in Section 3.4, an average and plan cover of 2.6 in. (66 mm) would be required. When these tolerances are added to the thickness occupied by the reinforcing bars and to the required clearances between bars and slab faces, the required minimum thickness is close to 8 in. (200 mm). Figure 3.5.3 shows the relationship of several component dimensions to the total deck thickness assuming the bar sizes most commonly used.

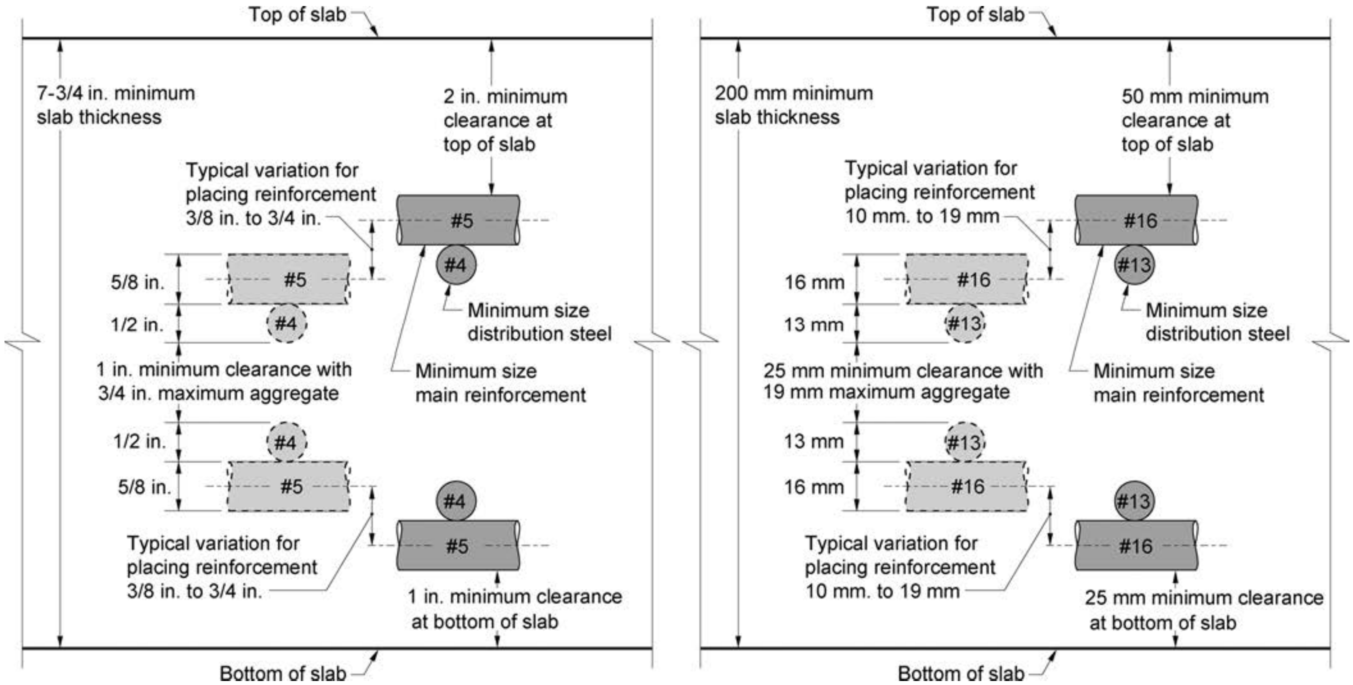

Fig. 3.5.3—Guidelines for vertical placement of reinforcement in bridge decks— U.S. Customary units (left) and SI units (right).

3.5.4 The use of stay-in-place (SIP) forms affects the thickness of the deck, and should be taken into consideration. Stay-in-place forming methods for decks provide a site construction advantage where form stripping is difficult.

If corrugated metal SIP forms are used, slightly greater slab thicknesses are required as the profile positions of the layers of reinforcing bars and the minimum cover over the reinforcement should be maintained. Figure 3.5.1 shows one type of deck design where the use of corrugated forms results in an additional 1 in. (25 mm) of concrete. This design simplifies form placement, particularly on radial structures.

If precast concrete deck panels are used, only a single mat of top reinforcing bar is required in the deck concrete. The designer should provide appropriate seating details for the precast panels that span between girder flanges. Details should include sufficient bearing width for the panels at the girders and sufficient clearance above the girder flanges to permit placement of cast-in-place concrete topping without leaving voids or honeycombed concrete. Bridge decks constructed with SIP precast concrete deck panels frequently develop reflective cracking of the cast-in-place concrete cover over the edges of the panels in the transverse direction, and sometimes the longitudinal direction. Post-tensioning of the panels might be employed to prevent such cracks from developing.

3.5.5 Adequate provision for deck haunches, or fillets or bolsters, is a design feature associated with deck thickness. The designer should select bearing elevations so that the steel or precast concrete girder does not penetrate into the deck slab thickness at any point along its length. The designer should consider the differences between the roadway profile and the girder profile—including the possible deviations from expected girder camber—at various points along the girder length. Small concrete haunches are formed in that portion of the deck where the top surface of the girder is lower than the bottom of the slab. Designers should include provisions for deck haunches, such that slab thickness is not reduced and the placement of reinforcement is not affected where the girder might project into the slab.

3.6—Deck drainage

3.6.1 It is vital to establish grades that will ensure proper drainage of the deck. Typical cross slopes are 2% or 1/4 in./ft (20 mm/m). In addition to provisions for storm water removal, attention should be given to the problem of draining the small quantities of water from melting snow and brine from deicing chemicals. The shallow slopes and crowns sometimes found on bridge decks, the small inaccuracies in finish of the wearing surface, the confining effect of the curb or barrier, and the accumulation of dirt and debris in gutters often result in inadequate deck drainage (Fig. 3.6.1(a)). Ponding of water and brine on an inadequately drained deck is a basic cause of bridge deck deterioration. This deterioration may take the form of popouts and spalls due to expanding aggregate or corrosion of reinforcement due to penetration of chlorides. Poor drainage may lead to premature failure of joints, and result in deterioration of superstructure and substructure components (Fig. 3.6.1(b)).

3.6.2 Drains should be sized and located so that drain water may be removed quickly and will not be emptied onto, or blown against, the concrete or steel below. An adequate number of small deck drains should be provided in flat surface areas. Materials used in drains and conduits should be resistant to the corrosive effect of deicing chemicals. Consideration should be given to directing drain water to avoid erosion of head-slopes. Some regulations may require

Fig. 3.6.1(a)—Poor deck drainage causes water to collect at joint.

Fig. 3.6.1(b)—Water leakage through failed joint causes corrosion and deterioration of pier cap.

collection of deck drainage to prevent contamination of downstream water resources.

3.6.3 Inlets should be sized to prohibit large particles, such as beverage cans, from lodging in the drain conduit and causing stoppages (Fig. 3.6.3). Sharp-angle turns should be avoided in drainage conduits, and outfalls should be readily accessible to facilitate cleaning.

Fig. 3.6.3—Poor design and lack of maintenance permit debris to accumulate and block deck drains.

3.7—Joint-forming materials and locations

Deck joints are a common point of failure and a source of significant maintenance costs in highway structures. From a durability aspect, it is desirable to reduce the number of joints in bridge decks to the greatest extent possible. One method of eliminating joints at the ends of a structure is by employing integral or semi-integral abutments. Using continuous beams and deck slabs in place of simply-supported spans can eliminate joints and, in addition, provide structural redundancy, although not all joints can be eliminated, particularly on long structures. Adequate watertight deck joints should be provided to accommodate movements. The type(s) of joints required for a bridge will depend on other design factors, including the type of support system, simple or continuous design, the length of the span(s) between joints, whether the joints will be subject to deicing operations (blade impact), and traffic type (percentage of trucks) and volume. Types may include strip seal expansion joints, pourable joint seals, compression joint seals, assembly joint/seal (modular), or open expansion joints. Compression seals can accommodate movements from 0.25 to 2.5 in. (6 to 65 mm) and are most commonly recommended for contraction joints and fixed joints (Purvis 2003). Design, selection, installation, and maintenance of joints and joint-forming materials can be found in ACI 504R-90. The following are brief descriptions of some common joint seal configurations, excerpted in part from a recent report by French and McKeel (2003).

Proper preparation of adjacent vertical surfaces is essential to the successful installation of joint systems that rely on surface bond or compression to achieve a watertight seal. Cleaning of the joint faces, priming of the surfaces, and placement of the sealant should be sequenced quickly to minimize the chances of contamination. Adherence to recommended installation procedures is absolutely essential to attaining satisfactory service from a joint sealing system (French and McKeel 2003). For further guidance on the application and performance of joint systems for concrete bridge decks, refer to Purvis (2003).

3.7.1 *Field molded seal*—This commonly used system consists of a self-leveling sealing material that is poured into the joint (Fig. 3.7.1). A closed-cell foam backer rod placed in the joint below the sealer supports it until it has cured. After curing, the sealing material remains flexible to accommodate horizontal and vertical movements. A cold-poured silicone rubber material is generally used. The system is commonly used where joint movement is 3/16 in. (5 mm) or less, although manufacturers claim suitability for larger movements.

3.7.2 *Open-cell compression seal*—There are several proprietary configurations of open-cell compression seals. They are generally neoprene rubber strip members that are rectangular in cross section, with various configurations of internal diagonal and vertical webs. The seals are placed in the joint while in compression with the aid of a lubricating adhesive, which cures to bond the sides of the seal to the joint

Fig. 3.7.1—Field-molded (pourable) joint seal system.

Fig. 3.7.4—Typical strip seal joint system in service.

faces. Compression seals can accommodate joint movements ranging from 1/4 to 2-1/2 in. (6 to 65 mm).

3.7.3 *Closed-cell compression seal*—A low-density closed cell foam rectangular-shaped strip member is compressed into the joint with an elastomeric primer to function in a manner similar to the open-cell compression seal.

3.7.4 *Strip seal*—Strip seals are V-shaped strips of elastomeric materials that are generally mechanically locked to metal retainer members at the edges of the expansion joint (Fig. 3.7.4). One strip seal evaluated in the study was bonded to the sides of the joint. Strip seals, which are highly regarded nationally, accommodate movements up to 4 in. (100 mm).

3.7.5 *Plug seal*—Plug seals are deformable polymer-modified asphalt concrete material placed in a cutout area over the expansion joint at the deck surface. A backer rod is compressed into the joint opening below the cutout, and the entire blocked out area is sealed with the binder material used in the mixture. A plate placed over the joint opening and sealed with the binder material supports the elastomeric asphalt layer, which accommodates the movement of the deck. Plug seals are appropriate for a maximum joint movement of 2 in. (50 mm).

3.7.6 *Inflatable neoprene seal*—The system consists of a preformed open-cell neoprene strip member bonded to the edges of the expansion joint with a structural epoxy adhesive. The sides of the seal and the joint face are coated with an epoxy adhesive, and the seal, which is sized to match the midrange joint opening, is then inflated to ensure a positive seal with the joint face. Inflation is maintained during the entire curing time of the adhesive and is then allowed to deflate as the air bleeds out.

Fig. 3.7.7—Example of modular joint system.

3.7.7 *Modular joint systems*—A variety of proprietary modular joint systems (Fig. 3.7.7) are commercially available. These systems tend to be used in larger width joints that should withstand heavy vehicular traffic and possibly frequent snow removal operations. Such systems, which tend to be preformed and mechanically anchored rather than adhered or compressed within the joint, may be cast into place during initial deck construction or retrofit during rehabilitation operations. These systems could incorporate cover plates.

3.7.8 *Open joint systems*—Open joint systems, including finger-joint systems, are not designed to prevent water infiltration, but to accommodate smooth vehicular travel over the joint while permitting water and contaminants to pass. These are generally employed under very heavy traffic conditions in joints that have expected horizontal movements that are greater than other systems can accommodate. These systems provide no protection for underlying superstructure and substructure components, relative to moisture and aggressive contaminants that flow from the deck. Drain troughs and conduits may be needed below such joints on large bridges.

3.8—Types and causes of deck cracking

Cracks in reinforced concrete compromise the durability of the materials by providing a path for ingress of moisture and other contaminants that may contribute to corrosion or other forms of material degradation. The designer should consider causes of the various types of cracking to avoid such conditions wherever possible. Cracks may be classified according to their orientation in relation to the direction of traffic as longitudinal, transverse, diagonal, or random. In addition, the terms "pattern cracking" and "crazing" are used to refer to characteristic defects as defined in ACI 201.1R-08. Examples of several types of cracking are shown in Fig. 3.8(a) through (c). The severity of cracking is conventionally expressed qualitatively as fine, medium, and wide, based on crack width.

ACI 201.1R-08 defines cracking severity as:

1. Fine: Generally less than 0.04 in. (l mm) wide;
2. Medium: Between 0.04 in. (l mm) and 0.08 in. (2 mm) wide; and
3. Wide: Over 0.08 in. (2 mm) wide.

A survey by the Portland Cement Association (PCA) (1970) of randomly selected bridge decks in eight states provides some insight to frequency and causes of various categories of cracking, recognizing that most cracks are caused by a number of interacting factors. The survey found comparatively little longitudinal and diagonal cracking.

3.8.1 Diagonal cracking occurred most often in the acute angle corner near abutments of skewed bridges, or over single-column piers of concrete box girder, deck girder, or hollow slab bridges.

3.8.2 Transverse cracking was observed on about one-half of the 2300 spans inspected. No one factor can be singled out as the cause of transverse cracking. Among the more important factors were:

- External and internal restraint on the early and long-term shrinkage of the slab; and
- A combination of dead-load and live-load stresses in negative moment regions.

In general, the observed crack pattern suggests that live-load stresses alone play a relatively minor role in transverse cracking.

A study of 72 North Carolina highway bridges was completed in 1985, shortly after their construction. The study sought to determine the frequency, extent, and cause(s) of transverse cracking in decks on steel and prestressed concrete girder superstructures of both simple and continuous design. In the first of two reports, the impact of construction and materials was investigated (Cheng and Johnston 1985) and in the second, the influence of superstructure type, deck casting sequence, and superstructure vibrations under load (Perfetti et al. 1985) was discussed. The study found the most frequent transverse cracking occurred on continuous structures, most particularly those comprising concrete decks on steel girders. The casting sequence was found to have some influence and seemed to relate to the development of residual stresses after placement, but could not be fully correlated with observed cracking. As one might expect, weather conditions conducive to high evaporation rates and thermal contraction contributed to higher incidence of cracking. Individual contractor practices were also a factor, as certain contractors' work appeared to be more prone to cracking, though causality was not clearly established.

A comprehensive investigation (Krauss and Rogalla 1996) of major factors that influence transverse cracking concluded that multi-span continuous composite steel girder bridges exhibited the highest severity of transverse cracking. Also, post-tensioned bridges had the least susceptibility to deck cracking. The deck and girder shrink together, as well as the post-tensioning, inducing compressive stresses in the deck. It was concluded that simply supported spans can

Fig. 3.8(a)—Pattern cracking.

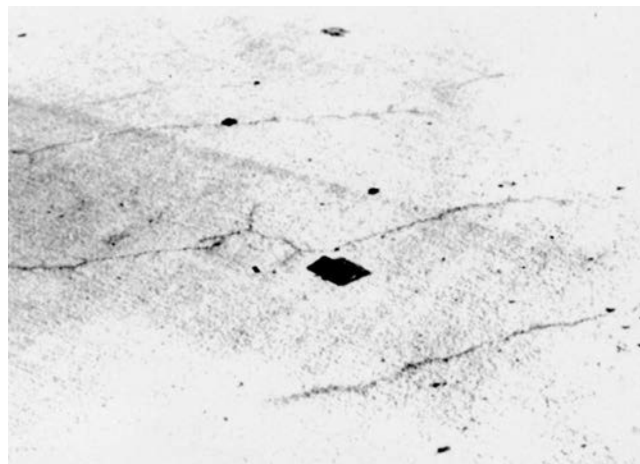

Fig. 3.8(b)—Diagonal cracking.

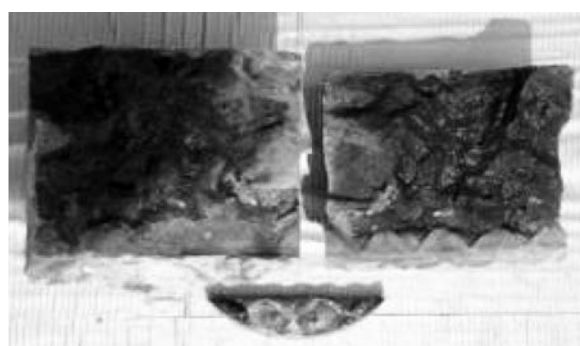

Fig. 3.8(c)—Faces of crack in core from a concrete deck.

make bridge decks less susceptible to transverse cracking. Research (Buckler et al. 2002) has shown that there is a strong correlation between the increase in transverse beam spacing and an increase in the severity of longitudinal deck cracking. The increased cracking is attributed to the additional stresses from flexural bending.

Brown and Weyers (2003) reported that the total transverse and diagonal cracking in bridge decks is a function of the increase in transverse beam spacing. A correlation was also established between the annual average daily truck traffic and the increase in transverse and diagonal cracking for decks built with epoxy-coated reinforcing steel.

A survey of 10 bridge decks totaling 35,500 ft^2 (3300 m^2) showed somewhat different results (Brown and Weyers 2003). Eighty-six percent of the 4203 ft (1281 m) of linear cracking was longitudinal cracking. Of the longitudinal cracking, 85% was less than 0.012 in. (0.30 mm) wide. The longitudinal cracking was typically over and parallel to the flanges of the superstructure beams. More recent design methods of longer, continuous, and wider spacing between the deck supports could have contributed to a change in the orientation of the major percentage of cracking from transverse to longitudinal cracking.

3.8.3 Pattern and random cracking were usually shallow, and could be related to early or long-term drying. This minor cracking was a common defect. Occasionally, severe cases were encountered in which the probable causes were severe early drying (plastic shrinkage cracking) (Keller 2004) or unstable conditions associated with reactive aggregates (Lerch 1957).

CHAPTER 4—CONCRETE MATERIALS

4.1—General

Bridge deck exposure is recognized as severe, and selection of cementitious materials, aggregates, and admixtures for deck concrete is critical. Proper selection of materials is the prerequisite for a long service life. Other aspects required for achieving durable deck concrete include concrete proportioning, mixing, delivery, placing, finishing, curing, and maintenance. They are covered in ACI 201.2R-08, ACI 211.1-91, ACI 211.2-98, ACI 211.4R-08, ACI 223R-10, ACI 304R-00, ACI 304.2R-96, ACI 304.5R-91, ACI 308R-01, ACI 309R-05, and ACI 345.1R-06. This section is devoted to a discussion on concrete-making materials and their influence on concrete properties.

4.2—Concrete-making materials

4.2.1 *Cementitious materials*

4.2.1.1 *Portland cement*—Portland cement is hydraulic cement that sets and hardens by chemical interaction with water and is produced by pulverizing portland cement clinker, usually in combination with calcium sulfate. Several types of portland cement are specified in ASTM C150/C150M-11 or AASHTO M85-09. Both specifications include the following types of cement:

- Type I: Normal, general-purpose cement;
- Type IA: Normal, air-entraining;
- Type II: Moderate sulfate resistance;
- Type IIA: Moderate sulfate resistance, air-entraining;
- Type III: High early strength;
- Type IIIA: High early strength, air-entraining;
- Type IV: Low heat of hydration; and
- Type V: High sulfate resistance.

A Type I/II designation indicates that cement meets ASTM C150/C150M-11 requirements for Types I and II. Within all eight types of cement, low-alkali cements are of particular interest, especially when the use of alkali-reactive aggregate is a possibility. Low-alkali cements are characterized by less than 0.60% of equivalent alkali content by mass of cement. It should be emphasized, however, that the use of low-alkali cements alone will not necessarily prevent alkali-silica reaction (ASR) from occurring. Additional information on ASR is presented in ACI 201.2R-08.

Table 4.2.1.1—Comparison of AASHTO M85-09 and ASTM C150/C150M-11 cement specifications

	AASHTO M85-09	ASTM C150/C150M-11
Maximum C_3S	58% (for Type II)	Not specified (except 35% for Type IV)
Blaine fineness, average maximum	4000 cm^2/g (Types I and II)	Not specified
Maximum processing additions	1%	0%
Use of interground lime	Not allowed	5% maximum

Type III cement is similar to Type I; however, it is approximately 50% finer. The average Blaine fineness of Type III cement surveyed in 1999 was 2670 ft^2/lb (547 m^2/kg), versus 1800 ft^2/lb (369 m^2/kg) for Type I (Tennis 1999). For comparison, the average Blaine fineness values of Types II and V were 1840 and 1820 ft^2/lb (377 and 373 m^2/kg), respectively. ASTM specifications allow for a higher SO_3 content for Type III cements that enable the manufacturer to regulate the set time and influence the rate of strength development and volume change.

Type IV cement is not commonly produced. The low heat of hydration, typical for Type IV, can be achieved by using blended cements or by incorporating supplementary cementitious materials.

Air-entraining cements are not commonly used. Air entrainment is usually controlled by the use of air-entraining admixtures.

The specific gravity of portland cement is approximately 3.15 (Kosmatka et al. 2002). Extensive information on cement manufacturing and composition can be found elsewhere (Bhatty 2004; Bhatty et al. 2004).

ASTM C150/C150M-11 and AASHTO M85-09 specifications for cement are similar; however, there are some differences in the limits on tricalcium silicate (C_3S) content, Blaine fineness, processing additions, and the use of interground lime. Specific information is presented in Table 4.2.1.1.

In a study that compared ASTM Type II (ASTM C150/C150M-11 and AASHTO Type II (AASHTO M85-09) cements, fresh and hardened concrete properties were not that different, except for the lower early-age strength exhibited by concretes made with AASHTO cement (Burg and Panoussaki 1993). The authors also reported on the adoption of cement specification by the state departments of transportation.

4.2.1.2 *Blended cements*—Blending or intergrinding one or more supplementary cementitious materials (SCMs) with portland cement produces blended hydraulic cements. Blended cements are uncommon in North America due to the practice of directly adding SCM to the concrete mixture. The primary SCMs used in blended cement production include fly ash, slag cement, silica fume, and calcined clay. Characteristics of SCMs, when used in blended cements, and the mechanism of their influence on concrete properties is summarized by Detwiler et al. (1996).

Blended cements should conform to ASTM C595/C595M-11, C1157/C1157M-10, or AASHTO M240-10. ASTM C595/C595M-11 and AASHTO M240-10 identify five classes of blended cement:

- Type IS: Portland blast-furnace slag cement, which is usually used for general construction. The slag constituent is between 25 and 70% by mass of total cementitious content;
- Types IP and P: Portland-pozzolan cement, which is usually used for general construction. Type P is used when high strength at early ages is not required. The pozzolan content ranges between 15 and 40% by mass of total cementitious materials;
- Type I(PM): Pozzolan-modified portland cement, which is used for general construction. The I(PM) cement contains less than 15% of pozzolan by mass of total cementitious materials;
- Type I(SM): Slag-modified portland cement, which is used for general construction. The slag constituent is less than 25% by mass of total cementitious content; and
- Type S: Slag cement, which is generally not for use in structural concrete. The slag cement content is at least 70% by mass of total cementitious materials.

Additional requirements may also be specified. In such cases, suffixes are added to the cement designation as follows:

- (A) for air-entraining cement;
- (MS) for moderate sulfate resistance;
- (MH) for moderate heat of hydration; and
- (LH) for low heat of hydration (Type P only).

ASTM C1157/C1157-10 recognizes six different types of cement with uses similar to ASTM C150/C150M-11 Types I through V:

- Type GU: General-purpose with uses similar to ASTM C150/C150M-11 Type I;
- Type HE: High-early-strength with uses similar to ASTM C150/C150M-11 Type III;
- Type MS: Moderate sulfate resistance with uses similar to ASTM C150/C150M-11 Type II;
- Type HS: High sulfate resistance with uses similar to ASTM C150/C150M-11 Type V;
- Type MH: Moderate heat of hydration with uses similar to ASTM C150/C150M-11 Type II; and
- Type LH: Low heat of hydration with uses similar to ASTM C150/C150M-11 Type IV.

An additional option is low reactivity with alkali-reactive aggregates, designated by the letter "R" following the letter designation.

Unlike ASTM C150/C150M-11 or C595/C595M-11, ASTM C1157/C1157M-10 is a performance-based specification that has no restrictions on the composition of the cement or its constituents. As such, it allows for optimization in regards to ingredient proportioning and particular concrete (cement) properties. In 1993, ASTM C1157/C1157M had not yet been adopted by any of state departments of transportation (Burg and Panoussaki 1993). This standard is still not widely adopted. However, several states, such as California, Colorado, Utah, and Minnesota, permit use of this standard through their standard specifications or special provisions.

A comparison of properties of concretes made with ASTM C150/C150M-11 and blended cements should be made with caution due to limited information available (Johansen et al. 2005).

4.2.1.3 *Supplementary cementitious materials*—SCMs include natural pozzolans, fly ash, silica fume, and slag cement. SCMs are also referred to as mineral admixtures. All of these materials have been successfully used in concrete. Their effect on concrete properties is due to pozzolanic or pozzolanic and hydraulic reaction. In the reaction, calcium hydroxide (free lime) converts into calcium silicate hydrate (CSH).

Traditionally, SCMs were individually added to concrete, but have become more readily available. It is common to combine more than two of these materials to optimize concrete properties. When three cementitious materials are used, they are called "ternary" mixtures. Some researchers reported better properties were obtained with ternary mixtures than with binary mixtures, especially in sulfate-bearing environments (Shiathas et al. 2003; Antiohos et al. 2004).

According to the PCA (2000), more than 60% of ready mixed concrete uses SCMs in concrete production.

4.2.1.4 *Fly ash and natural pozzolans (ASTM C618-08a)*—The use of natural pozzolans has been documented for centuries. Their effect on concrete properties includes reduced permeability, improved sulfate attack resistance, reduced expansion due to ASR, and reduced heat development in concrete. Calcined clay, calcined shale, and metakaolin are processed materials, and are the most common natural pozzolans used today (Kosmatka et al. 2002). Metakaolin is calcined clay with an average particle size of about 0.04 to 0.08 mils (1 to 2 μm). Calcined shale possesses both pozzolanic and hydraulic cementing properties.

When used in concrete, calcined clays constitute 15 to 35% of cementitious materials. Metakaolin is used in the amount of up to 10% of cement mass. The specific gravity of calcined clays ranges between 2.4 and 2.6, and the Blaine fineness ranges from 3170 to 6590 ft^2/lb (650 to 1350 m^2/kg) (Kosmatka et al. 2002).

Natural pozzolans may cause either an increase or decrease in water demand; however, calcined clays and calcined shale have been found to have almost no effect on water demand in concrete (Kosmatka et al. 2002). Metakaolin is often used when high strength or low permeability is required.

Among other SCMs used in concrete, fly ash is used the most. It is a by-product of the coal industry. Its particles are glassy spheres with diameters ranging from less than 0.04 mil (1 μm) to more than 4 mils (100 μm); however, the majority of them are less than 0.8 mil (20 μm). Typical fineness of fly ash ranges between 1460 to 2440 ft^2/lb (300 and 500 m^2/kg), but can be as low as 980 ft^2/lb (200 m^2/kg) and as high as 3420 ft^2/lb (700 m^2/kg). The specific gravity ranges between 1.9 and 2.8, and the bulk density of compacted fly ash ranges from 70 to 94 lb/ft^3 (1120 to 1500 kg/m^3). Fly ash is gray or tan in color, and resembles portland cement.

Utilization of fly ash in concrete production is minimal. It has been reported that world's annual production of fly ash is 496 million tons (450 million tonnes); however, less than 8%, or 39 million tons (35 million tonnes), is used for concrete production (Mehdi 2001). Fly ash is used in approximately 50% of ready mixed concrete (PCA 2000).

In accordance with ASTM C618-08a and AASHTO M295-07, fly ashes and natural pozzolans are classified into the following three classes:

- Class N: Raw or calcined natural pozzolans, which are volcanic ash, pumice, calcined clay, and metakaolin;
- Class F: Fly ash with pozzolanic properties; and
- Class C: Fly ash with pozzolanic and cementitious properties.

Class F fly ashes are primarily pozzolanic in nature, and have no cementitious properties. Class F materials are characterized by a relatively high loss on ignition (LOI). The LOI is not always consistent, even for the same source fly ash. The LOI values are usually below 5%, but may also approach 10% (Kosmatka et al. 2002). This high LOI (carbon) content may cause increased dosage of air-entraining agent in concrete and air content instability.

Class C fly ashes exhibit both pozzolanic and cementitious properties. They are characterized by low sulfur content, higher calcium level, and consistent LOI, which usually does not exceed 2%. This higher calcium level will increase the pH of the pore water and produce related ASR issues compared to Class F fly ash.

It is possible for an intermediate lime content fly ash to meet both Class F and Class C classifications. Class F and Class C fly ashes often constitute from 15 to 25% and from 15 to 40% of total mass of cementitious materials in the mixture, respectively. Their content in concrete is often expressed as a percentage of portland cement replacement.

Potential benefits of using fly ash in concrete include reduced permeability, reduced expansion due to ASR in the case of Class F fly ash, reduced heat of hydration, and lower concrete cost. Although it increases set time and reduces early strength, fly ash has a positive effect on long-term strength. Fly ash also can change the pore structure of the concrete by producing lower rapid chloride permeability values when tested in accordance with ASTM C1202-10 after long-term curing.

Concrete mixtures containing fly ash usually require up to 10% less water (Kosmatka et al. 2002; Helmuth 1987). Some Class F and Class C fly ashes, however, may increase water demand by up to 5% (Gebler and Klieger 1986). Fly ash was found to be beneficial in concrete pumping and during finishing operations.

Natural pozzolans and fly ashes used in concrete production should meet ASTM C311-11. Extensive reviews of natural pozzolans and fly ash are presented in ACI 232.1R-00 and ACI 232.2R-03, respectively.

4.2.1.5 *Silica fume (ASTM C1240-10a)*—Silica fume, also called microsilica or condensed silica fume, is a by-product of ferrosilicon industry. Its spherical particles are about 100 times smaller than cement particles. The average diameter is about 0.004 mil (0.1 μm), with a surface area of about 98,000 ft^2/lb (20,000 m^2/kg) by the nitrogen adsorption method. The specific gravity of silica fume usually ranges from 2.2 to 2.5. The bulk density of uncompacted silica fume varies from 8 to 27 lb/ft^3 (130 to 430 kg/m^3) (Kosmatka et al. 2002).

Silica fume use in concrete ranges from 5 to 10% of total cementitious materials, with a 7% maximum value typically used to control fresh and hardened properties. It usually comes in the form of a slurry or densified powder, available in bulk, which can be handled like cement through a silo, or prepackaged for addition to a pan-type mixer. Regardless of its form, it should be thoroughly mixed for adequate distribution in the concrete mixture. Silica fume increases water demand so the use of a high-range water reducer is highly recommended.

The benefits of significant reduction in permeability and increase in compressive strength, both early and long-term, outweigh the high cost of silica fume. Silica fume concrete is also known to be more difficult to finish, and is attributed to reducing, or even eliminating, bleed water. Finishing and curing operations should begin immediately after placement; otherwise, plastic shrinkage cracking may appear.

Silica fume used in concrete should meet the requirements of ASTM C1240-10a or AASHTO M307-07. Concrete specification information is summarized in Neville (2001). An extensive review of silica fume is presented in ACI 234R-06 and by Malhotra et al. (1987).

4.2.1.6 *Slag cement*—Slag cement is a by-product of the iron industry. Its particle size is less than 45 microns. Blaine fineness usually ranges from 1950 to 2930 ft^2/lb (400 to 600 m^2/kg), specific gravity ranges from 2.85 to 2.95, and the bulk density varies from 66 to 86 lb/ft^3 (1050 to 1375 kg/m^3) (Kosmatka et al. 2002). Annual production of slag worldwide is approximately 110 million tons (100 million tonnes), yet only a small percentage is being used in concrete (Mehdi 2001). Slag cement (water-cooled slag) has hydraulic properties, but air-cooled slag does not (PCA 2000). Slag is not used as the sole cementing material in concrete. When used, however, it usually accounts for 30 to 45% of the cementing materials by weight (PCA 2000).

In accordance with ASTM C989-10 and AASHTO M302-06, slag cement can be classified into three grades: 80, 100, and 120. The grade of a slag is based on its activity index, which is the ratio, expressed as a percentage, of the compressive strength of a mortar cube made with a 50% slag-cement blend to that of a mortar cube made with the reference cement alone (Whiting et al. 1993). Grade 80 is characterized by a low activity index, whereas Grade 100 and 120 are characterized by moderate and high activity indexes, respectively.

Some characteristics exhibited by concrete containing slag cement include reduced permeability, reduced expansions due to ASR, reduced water demand, improved sulfate resistance, increased set time, reduced heat of hydration, improved workability, improved finishing, and increased ultimate strength (ACI 233R-03). At temperatures above

84°F (29°C), set time was found to be unaffected (Hogan and Meusel 1981).

An extensive review of slag cement is presented in ACI 233R-03.

4.2.1.7 *Other cements (Types K and CAC)*—Other cements that have been used in bridge deck concrete include shrinkage-compensating cement. Specifications for the shrinkage-compensating cement (Type K) are covered under ASTM C845-04, and a thorough discussion of shrinkage-compensating concrete is provided in ACI 223R-10.

The main advantage in using Type K cement is to reduce shrinkage cracking. A study conducted by the Alabama Department of Transportation (DOT) compared the performance of Type K cement concrete with the standard concrete used on bridge decks and concrete containing shrinkage-reducing admixture (Cope and Ramey 2001). The results of the study showed that Type K cement concrete proved effective in the reduction of drying shrinkage, provided wet curing conditions were maintained. The permeability of Type K cement concrete, however, was comparable to the standard concrete. Concrete containing shrinkage-reducing admixtures (SRAs) have shown reduced shrinkage when compared with standard concrete, but not to the same extent as the Type K cement concrete. Also, the addition of SRA appeared to increase set time and cause an increase in dosage of the air-entraining admixture. Scaling and resistance to freezing and thawing of the SRA concrete were reported worse than for the standard mixture.

Gruner and Plain (1989, 1993) and Phillips et al. (1997) reported on the performance of shrinkage-compensating concrete in bridge decks constructed by the Ohio Turnpike Commission. It was found that no deterioration occurred on the Ohio Turnpike bridge decks for over 5 years of use on more than 300 bridge decks. In an article describing use of shrinkage-compensating concrete by the New York Thruway Authority (NYTA), the authors describe concrete characteristics and its performance as outstanding; however, significant scaling problems in the NYTA decks were reported (Ramey et al. 1999). Eventually, NYTA discontinued deck construction with shrinkage-compensating concrete. Gulyas et al. (2008) reported excellent performance of lightweight concrete bridge decks constructed with Type K cement in Cleveland after 15 years of exposure.

4.2.2 *Aggregate*—Aggregate for bridge deck concrete may be either 3/4 or 1 in. (20 or 25 mm) maximum sized normalweight aggregate conforming to ASTM C33/C33M-11 or lightweight aggregate conforming to ASTM C330/C330M-09. The influence of aggregate properties on fresh and hardened concrete properties is presented in ACI 221R-96.

The high unit cost of bridge decks and long service life expectancy require special attention to aggregate selection. Natural coarse aggregate is usually inadequate for deck concrete due to its poor skid resistance characteristics. Any aggregate used for making concrete is to be evaluated for deleterious materials, including organic impurities, soft particles, materials finer than No. 200 (75 μm) sieve, lightweight materials including coal and lignite, clay lumps and friable particles, and chert with specific gravity less than 2.4.

Table 4.2.2(a)—Deleterious materials in aggregate and their effects on concrete properties

Material	ASTM test	AASHTO test	Possible effect on concrete
Organic impurities	C40-04 C87-10	T21-05 T71-08	Setting time
Materials finer than No. 200 (75 μm) sieve	C117-04	T11-05	Higher water demand, lower bond
Lightweight materials including coal and lignite	C123-04	T113-06	Durability, staining, popouts
Clay lumps and friable particles	—	—	Popouts, workability, and durability
Chert with relative density less than 2.4	—	—	Popouts, durability
Alkali-reactive aggregate	C1260-07 C295-08 C227-10 C289-07 C586-05 C1293-08b	T303-00	Expansion, cracking

Table 4.2.2(b)—Other aggregate-related tests

Characteristic	ASTM test	AASHTO test
Aggregate grading	C117-04 C136-06	T11-05 T27-06
Fine aggregate degradation (due to attrition)	C1137-97	
Aggregate particle shape and surface texture	C295-08 D3398-00(2006)	
Aggregate abrasion resistance	C131-06 C539-84(2011)	T96-02
Aggregate bulk density and voids	C29/C29M-09	T19M/T19-09
Relative density and absorption	C127-07 C128-07a	T85-10 T84-10
Sulfate resistance	C88-05	T104-99
ASR	C1260-07 C295-08 C227-10 C289-07 C586-05 C1293-08b	T303-00
Freezing-and-thawing resistance	C666/C666M-03(2008)	T161-08

The aggregate is also to be assessed for alkali reactivity, especially with increasing alkalis found in present portland cements. Another important aggregate property for bridge deck construction is that the aggregate is to be a nonpolishing aggregate to maintain adequate skid resistance. ASTM and AASHTO designations for testing for the deleterious materials in aggregate, as well as their possible effects on concrete properties, have been compiled and are presented in Table 4.2.2(a). Other tests performed on aggregate have been compiled and are presented in Table 4.2.2(b).

Past performance is often the basis for aggregate acceptance. When such data are unavailable, an evaluation should be made by laboratory testing.

4.2.3 *Water*—Practically any water that is drinkable and has no pronounced taste or odor is satisfactory mixing water for concrete. No testing is required if such water is used. AASHTO T26-79 and ASTM C94/C94M-11 contain requirements for concrete mixing water. Due to greater demand for use of non-potable water, two ASTM standards,

Table 4.2.3—Test for water as specified in ASTM C1602/C1602M-06

Concrete property	Limits	Test method
Compressive strength, minimum percentage of control at 7 days	90	ASTM C109/C109M-11 or AASHTO T106M/T106-09
Time of set, deviation from control (hour:minute)	from 1:00 earlier to 1:30 later	ASTM C191-08 or AASHTO T131-10

ASTM C1602/C1602M-06 and ASTM C1603-10, were developed. These standards are applicable to water that is not intended for human consumption. The requirements covered in ASTM C1602/C1602M-06 are summarized in Table 4.2.3. ASTM C1603-10 requirements are optional.

Seawater and brackish water, even when meeting the aforementioned requirements, should not be used in concrete for bridge decks because of the increased possibility of reinforcing steel corrosion.

4.3—Chemical admixtures

A variety of chemical admixtures are used in bridge decks. For a detailed exposition regarding types and uses of admixtures, refer to ACI 212.3R-10 and ACI 212.4R-93. Of those discussed, useful admixtures for concrete bridge deck construction include air-entraining admixtures meeting ASTM C260/C260M-10a, and water-reducing, retarding, and accelerating admixtures meeting ASTM C494/C494M-10a Types A, B, and C. Combination water-reducing and retarding and water-reducing and accelerating admixtures are also included in ASTM C494/C494M-10a as Types D and E, respectively. High-range water reducer and retarding admixture requirements are also addressed in ASTM C494/C494M-10a as Types F and G, respectively.

The effectiveness of an admixture is influenced by numerous factors, including type and amount of cement, water content, aggregate gradation and shape, length of mixing period, time of addition to the mixture, consistency, and temperature of the concrete. Admixtures should be evaluated in trial mixtures, using the job materials under the temperature and humidity conditions anticipated for the job. Incompatibility between admixtures and other components, particularly the cement and certain fly ashes, may thus be revealed, allowing for changes to remedy the situation. The amount of the admixture used in such trials, or in the actual job when there is no provision for such trials, should be based on recommendations of the manufacturer.

Occasionally, the use of admixtures will produce side effects in concrete in addition to desired effects. For instance, although high-range water reducers increase the slump of concrete for a given water content, the rate of slump loss may be greater than for concrete without the high-range water reducer. Attention should be directed to this possibility because some changes may be required in the scheduling of mixing, placing, compacting, and finishing operations. Some water reducers may also cause significant increases in drying shrinkage of the concrete, even though their use may permit less total water to be used. This effect should be evaluated because an increase in shrinkage can influence the amount of cracking and subsequent performance of the deck.

Retarders are used to delay setting time of the concrete, allowing more time for placing and finishing, particularly when casting large deck areas in a continuous structure where setting before completion of placing and finishing operations could result in cracking due to deflections resulting from loads in adjacent spans. Retarders of the hydroxylated carboxylic acid types also generally increase the rate and capacity of bleeding. Changes in bleeding characteristics will require compensating changes in the timing of finishing operations and the provision of sun shades, windbreaks, or fogging.

Calcium chloride ($CaCl_2$), the most commonly used accelerator, generally increases drying shrinkage and may accelerate corrosion of the reinforcing steel. For this reason, calcium chloride should not be used in bridge deck concrete.

Concrete for bridge decks exposed to freezing-and-thawing cycles should have an adequate air-void system to provide escape path for freezing water. The use of air-entraining agents is highly recommended. It is well known that the higher the air content, the lower the concrete strength, especially in high-strength concrete. The amount of air entrainment in high-strength concrete that provides an adequate level of durability is contradictory. In one study (Saucier et al. 1965), air entrainment is recommended despite the loss of strength. In another study (Perenchio and Klieger 1978), high-strength concrete both with and without air entrainment was studied. In both cases, excellent freezing and thawing resistance was achieved. Freezing-and-thawing resistance (ASTM C666/C666M-03(2008) and salt scale resistance (ASTM C672/C672M-03) for low *w/cm* concrete is dependent on the *w/cm* as well as air void parameters. Several researchers recently reported results with respect to proper air void parameters required for concrete without a high-range water-reducing agent in high-strength concrete (Cohen et al. 1992; Attiogbe et al. 1992; Pinto and Hover 2001). When a high-range water-reducing admixture (HRWRA) is used, spacing factors can be greater or attained through lower concrete air content (Attiogbe et al. 1992).

There is a difference in the requirement for air content regardless of *w/cm* for salt scale resistance (ASTM C672/C672M-03) using the standard acceptance criteria. Air entrainment is indeed required for both high-strength concretes, 0.35 *w/cm,* and concrete with *w/cm* of 0.45 to 0.50, which can be salt scale resistant (Pinto and Hover 2001).

4.4—Effects on concrete properties

Those characteristics of the concrete that influence its water tightness, resistance to freezing and thawing, and abrasion are particularly important compared to those necessary for other applications of structural concrete. Even when the concrete is made with satisfactory materials, construction operations such as proportioning, transporting, placing, and finishing can detrimentally influence the deck performance unless the desired properties are obtained by diligent attention to the details of good concreting practice.

4.5—Workability and consistency

The workability of freshly mixed concrete as it is being placed in the bridge deck form should be such that the concrete can be readily compacted, struck off, and finished. Consistency measurements are helpful in control, but the aforementioned actual operations will reveal the need for possible changes in mixture proportions, aggregate grading, or some other aspect to enhance workability. Placement of the concrete during night hours might also be beneficial.

Workability usually improves when fly ash, slag, or calcined clay and shale are admixed with concrete. Silica fume, however, which has the opposite effect, could contribute to concrete stickiness leading to greater difficulties in finishing. In such cases, a high-range water reducer may be used to maintain the required workability. For concrete placed by pumping methods, the use of SCM, particularly silica fume, is beneficial.

Concrete slump should be kept to the minimum required for adequate compaction and finishing operations. Equally important is that the slump be uniform batch-to-batch for efficient and effective operations. When structural lightweight aggregate concrete is used, the slump can be reduced somewhat with little or no sacrifice in workability.

4.6—Bleeding

The bleeding of concrete is a matter of importance in bridge deck construction, particularly during hot weather, windy conditions, or in conditions of low relative humidity. Bleeding is controlled by the provision of adequate fines in the concrete; a relatively high cement content, fine aggregates containing the required amount of materials passing the No. 50 (300 μm) sieve, intentionally entrained air, and the minimum amount of water per unit volume that will provide the desired consistency. Concrete made with aggregate deficient in fines will benefit from the use of fly ash, which reduces water demand. A relationship between reduced bleeding rate and reduced water demand in concretes made with fly ash is presented by Gebler and Klieger (1986). Slag may increase or reduce bleeding in concrete depending on the slag's fineness. Reduced bleeding can be expected with finely ground slag, which is finer than cement. The effect of calcined clays, calcined shale, and metakaolin on bleeding is negligible. Significant reduction of bleeding and segregation can be accomplished when silica fume is used. Care should be exercised in the use of certain admixtures that may, as a side effect, increase the rate and capacity of bleeding.

As water is removed from concrete by bleeding, subsidence of the solid material takes place. Under certain conditions, early cracking at the surface of the concrete deck can result from the interaction of the subsidence of the plastic concrete and the restraint provided by the top reinforcing bar or other rigidly fixed items such as void forms.

Care should be taken to avoid rapid drying at the surface during the bleeding period, particularly when rate and capacity for bleeding are minimized. Exposure to sun and wind can result in the development of a surface crust beneath which bleeding water can collect and produce a zone of weakness, which is more prone to crack over the top reinforcing bar when under the influence of restraint to settlement forces. Plastic shrinkage cracking may also develop. Shading from the direct rays of the sun and the use of fine water spray by means of fog nozzles or monomolecular films may be required to avoid or minimize such developments (ACI 305R-10).

4.7—Air content

Field experience and laboratory studies have shown that the amount of entrained air required is a function of the maximum size of coarse aggregate used. Air-entraining admixtures that meet the requirements of ASTM C260/C260M-10a provide the proper size and distribution of air voids. Field control practice, however, involves only the measurement of the volume of air in the freshly mixed concrete. The volume of air entrained is primarily a function of the amount of air-entraining admixture used. Significant changes in air content, however, can result from changes in aggregate gradation and fine aggregate content, slump, concrete temperature, other admixtures, and mixing time.

Natural pozzolans and ground slag have almost no effect on the air-entraining dosage rates. Fly ashes and silica fume, however, require higher dosages of air-entraining agents. Class F ashes require more air-entraining agent than Class C fly ashes. Class F fly ashes are also known to lose more air during mixing.

4.8—Setting time

The setting time of concrete can easily be controlled with the use of chemical admixtures, accelerators, and retarders (Section 4.3). Retardation also occurs when fly ash or slag cement is used in the concrete mixture. Silica fume and natural pozzolans, such as calcined shale and calcined clay, usually have little effect on the set time. Regardless of the method used, delayed setting allows more time for placement and finishing, which may be beneficial during hot weather conditions.

4.9—Shrinkage

4.9.1 *Plastic shrinkage*—The probability of plastic shrinkage cracking increases with increasing evaporation rate from fresh concrete. This condition is independent from concrete constituents, with the exception of silica fume, which is known to produce low- or non-bleeding concrete. Without bleeding water, which replenishes near-surface water lost by evaporation, plastic shrinkage cracking is more likely to occur. Admixtures that delay the set time of concrete can also increase probability of plastic shrinkage cracking.

4.9.2 *Drying shrinkage*—Hardened concrete responds to changes in moisture content by expanding as moisture content increases and by shrinking as it dries. If kept continuously wet after casting, the amount of expansion is small, usually less than 0.015%, and can be accommodated with no problem. Shrinkage on drying, usually evaluated in plain concrete specimens with no reinforcement, generally ranges from about 400 to 800 millionths (0.04 to 0.08%) when exposed to drying at 50% relative humidity for 90 days or

Fig. 4.10—Surface scaling.

more. Reinforced concrete in field exposure generally shows lower shrinkage than the ones observed in the laboratory due to a larger volume-surface ratio and internal restraint. Even at low shrinkage levels, cracking may occur with restraint, and steps should be taken to minimize the amount of shrinkage on drying.

The most important controllable factor affecting shrinkage is the amount of water used per unit volume of concrete. Keeping the water content and the paste content as low as possible and the total aggregate content of the concrete as high as possible can minimize shrinkage. Using the largest size coarse aggregate consistent with reinforcement spacing maximizes total aggregate content. The use of low slumps and placing methods that minimize water requirements of the concrete are major factors in reducing shrinkage. High slumps, high cement contents, and high initial concrete temperatures will increase water requirements, and should be avoided.

Type K cement has been found useful in controlling shrinkage cracking (Section 4.2.1.7). The use of slag cement, and pozzolans such as fly ash and silica fume, can influence drying shrinkage. The impact, however, may or may not be significant, depending on proportions and other mixture design parameters, and whether sufficient restraint exists within the structure for shrinkage to contribute to stress development and cracking (Ramniceanu et al. 2010; Hossain et al. 2007; Tia et al. 2005; Mokarem et al. 2003).

4.10—Durability

The primary potentially deteriorating influences on concrete bridge decks are freezing and thawing, particularly in the presence of deicing chemicals and corrosion of the reinforcing steel. The resistance of concrete to freezing and thawing, even when various deicers are used, is significantly improved by the use of intentionally entrained air. Air-entraining admixtures meeting the requirements of ASTM C260/C260M-10a, when used to produce the recommended volume of entrained air, provide the proper size and distribution of air voids for effective protection. In addition, concrete should be air-dried before exposure to freezing and thawing. There are exceptions, however, when critically saturated conditions do not allow for this. Air void characteristics representative of an adequate system, as measured in hardened concrete by the linear traverse measurement technique in accordance with ASTM C457/C457M-10a, are as follows:

1. Calculated spacing factor less than about 0.008 in. (0.2 mm);
2. Surface area of the air voids greater than about 600 in.^{2}/in.^{3} (24 mm^2/mm^3) of air void volume; and
3. Number of air voids per linear inch of traverse is significantly greater, which is about twice the numerical value of the percentage of air in the concrete.

When ASTM C494/C494M-10a, Types F and G HRWRAs are used in concrete, these air void parameters still apply. Whiting and Schmitt (1987) reported that HRWRAs do not affect the durability of concrete.

Another concrete characteristic, especially important in the snow-belt areas, is its resistance to deicer scaling (Fig. 4.10). Scaling resistance can be improved when low *w/cm* is used and concrete has an adequate air-void system. Additional improvements can be accomplished by providing suitable finishing and curing, including a long period of air-drying before the first salt applications.

A low *w/cm* is helpful not only with scaling, but also with corrosion of the reinforcing steel. Most deicers are chloride-bearing salts—for example sodium, calcium, and magnesium—and their penetration to the reinforcing steel can cause rapid corrosion. High cement mixtures help by enhancing the probability for reduced *w/cm* and by increasing the capability for maintaining a high pH in the concrete—an environment that reduces the potential for steel corrosion.

Low *w/cm* are recommended because they provide concrete less permeable to water and deicer solution. For such concretes, the specified compressive strength, as defined in ACI 214R-11, should be at least 4500 psi (31 MPa) at 28 days. The maximum *w/cm* for bridge deck concrete should not exceed 0.45 by weight. ASTM C1202-10, often referred to as chloride permeability test, gives an indication of concrete permeability. Charge passed during the test of less than 1000 C (280 mA·h) is regarded as of high-quality concrete. Such values can easily be obtained with silica fume or metakaolin concrete. Other mineral admixtures, such as fly ashes, natural pozzolans, or slag cement, can also be used to reduce permeability. Regardless of the material used, low-permeability concrete can significantly extend service life of decks that are vulnerable to corrosion of reinforcing steel. For this reason, silica fume and metakaolin are often used in concrete used in deck overlays.

The benefit of mineral admixtures is not limited to permeability. They also help in reducing expansion due to alkali-silica reactivity of concrete with reactive aggregate. In general, Class F fly ashes are better than Type C fly ashes because they react with the by-product of the lime and reduce the pH of the pore water. The amount of mineral admixture should be carefully considered to avoid exacerbating the reaction. Tests on concrete, such as ASTM C1260-07, AASHTO T303-00 or ASTM C1293-08b, with different proportions of cement-mineral admixtures or petrographic examination of aggregate, are recommended. A more detailed description of tests and preventive techniques against alkali-

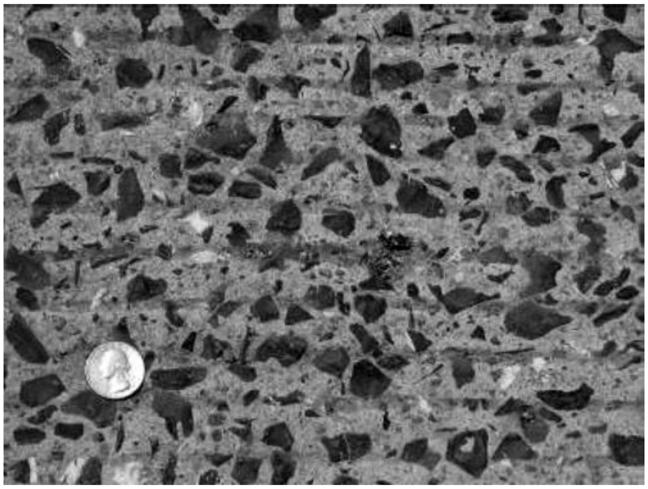

Fig. 4.12—Exposed coarse aggregate polished by tire wear contributes to low skid resistance.

silica reactivity is presented in ACI 221.1R-98, AASHTO (2000), and PCA Durability Subcommittee (1998).

4.11—Strength

Concrete strength is primarily a function of the *w/cm* and the extent of moist curing. Concrete proportions are selected on the basis of strength and durability requirements. For more detailed information, refer to ACI 211.1-91 and ACI 211.2-98 on proportioning, and ACI 201.1R-08 on durability.

In most instances, the requirements for durability previously discussed will govern the selection of *w/cm*, and the actual strength developed will be more than required from structural design considerations, which is limiting the maximum *w/cm* used.

Strength development can be controlled by the type and amount of cement, chemical admixtures, retarding or accelerating mineral admixtures, *w/cm*, and temperature. Silica fume is known to increase early strength due to its fineness and reactivity. Burg and Ost (1994) reported that ready mixed concrete with certain types of aggregates, silica fume, fly ash, and high-range water reducers can achieve strength levels of 20,000 psi (140 MPa).

4.12—Skid resistance

The skid resistance of a concrete bridge deck is influenced by the properties of the concrete, the properties of the component materials, and by the texture of the surface. The most important factor in skid resistance of concrete surfaces, especially at normal highway speeds, is surface texture. Satisfactory textures can be produced by brooming, wire drags, and flexible wire brushes. To promote retention of skid-resistant properties related to texture, deep texturing and practices that minimize wear are desirable. The latter includes low *w/cm* concrete mixtures, durable fine aggregates, avoidance of placing and finishing practices that tend to bring fines and water to the surface, and proper curing of the concrete surface. Pozzolans, silica fume, and slag cement have indirect effects, through the compressive strength of concrete, on skid resistance.

With increasing pavement wear or slower speeds, the characteristics of the fine aggregate become increasingly important in skid resistance of concrete surfaces. The silica content of the fine aggregate is the primary determinant in this instance, and aggregates possessing acid-insoluble (6N HCl) residue contents of 25% or greater provide good skid resistance. Coarse aggregate is relatively unimportant unless conditions have resulted in excessive wear and the coarse aggregate has become exposed at the surface (Fig. 4.12).

CHAPTER 5—REINFORCEMENT

5.1—General considerations

Reinforcing steel for bridge decks should meet the requirements of ASTM A615/A615M-09b, ASTM A706/A706M-09b, or AASHTO M31/M 31M-10. Stainless steel bars should meet the requirements of ASTM A955/A955M-11, and stainless steel cladding should meet the requirements of ASTM A276-10. Galvanized reinforcement should meet the requirements of ASTM A767/A767M-09. Organic-coated reinforcement should comply with ASTM A775/A775M-07b or AASHTO M284/M 284M-09. Fiber-reinforced polymer reinforcement should comply with the provisions of ASTM D7205/D7205M-06, and ACI 440.5-08 and ACI 440.6-08. Specifications, test methods, and design and construction using FRP reinforcement are provided in ACI 440R-07, AASHTO (2009), and CSA CAN/CSA-S6 (CSA 2006). Of equal importance, every effort should be made to ensure that bars are of proper size and length, placed and spliced in accordance with the plans, and have adequate concrete cover, especially over top bars. Adequate cover over bottom bars may be equally important in marine environments and at grade-separation bridges.

5.1.1 *Delamination and spalling*—Delaminations and surface spalls result from separation of a portion of the concrete surface, typically above reinforcement, by excessive internal pressure resulting from a combination of forces. An example of spalling is shown in Fig. 5.1.1. Spalling may expose reinforcement, decrease deck thickness, and subject the thinned section to impact. Joint spall is used to designate spalls adjacent to various types of joints. The incidence of spalling varies considerably, depending on location (PCA 1970), but where it occurs it is a serious and troublesome problem. It is related to the use of deicing chemicals, corrosion of reinforcement, traffic, and quantity and quality of concrete cover.

Damage caused by corrosion may result in costly repairs or replacement of bridge decks that have been in-service generally for 20 to 25 years, but in some instances as little as 10 to 15 years (Weyers et al. 1994). Because this early deck deterioration is primarily due to corroding of the concrete reinforcement, it is recommended that the reinforcement be either coated steel or a noncorrosive material when in a chloride environment. It is recognized in many countries that low-permeability concrete, in the form of a low *w/cm*, with or without secondary cementing materials, and sufficient clear concrete cover, is sufficient for the severity of the exposure conditions and design service life.

Fig. 5.1.1—Surface spalling.

5.2—Reinforcement placement

Because bridge decks depend on accurate placement of reinforcement for design performance and in-service durability, tolerances should be maintained during construction as shown in Chapter 8 of the Concrete Reinforcing Steel Institute (CRSI) *Manual of Standard Practice* (2009).

5.3—Reinforcement supports and ties

Reinforcement should be held securely by suitable supports and ties to prevent displacement during concrete placement. Plastic chairs or precast concrete blocks are sometimes used for support of the bars; more commonly, metallic reinforcement chairs, with or without plastic-protected ends, are used. Coated or stainless tie wire and reinforcement supports should be used with organic-coated reinforcement. For deep deck sections, welded support assemblies are sometimes used, or the primary reinforcement may be in the form of welded trusses that simplify accurate placement. Whatever the system used, there should be assurance that the supports will: be adequate to carry construction loads before and during placement, not stain concrete surfaces, displace excessive quantities of concrete, or allow reinforcing bars to move from their proper positions. Several suggested systems for support of deck reinforcement are shown in Chapter 3 of the CRSI *Manual of Standard Practice* (CRSI 2009).

While deck strength is not affected by the number of intersections tied, it is essential that sufficient ties and wire of adequate size are used to ensure that bars will be held in proper position during the concrete placement and consolidation operations. A safe rule would require that every other reinforcing bar intersection be tied and that wire not smaller than 15 gauge (1.83 mm) be used.

5.4—Concrete cover over reinforcement

5.4.1 As the first line of defense in delaying the onset of reinforcing steel corrosion, it is essential that the specified clear concrete cover thickness over the reinforcing bar be maintained. Concrete cover of the bottom mat is easily controlled by bar supports of the required height. Cover over the top mat is, however, much more difficult to control due to the inherent flexibility of the strikeoff screed system and possible differential deflections of adjacent girders.

5.4.2 Possible methods for checking expected top mat cover are as follows:

1. Obtain and plot elevations of the top reinforcement on a grid pattern and compare the results with elevations along the strikeoff screeds;
2. Stretch a string line between the screeds and measure down to the reinforcement; or
3. Run the strikeoff mechanism along the screed support rails and measure the space between the float board and reinforcing bar (dry-run), attach a block of wood to the float board and reinforcement, or attach a block of wood to the float board that has a thickness equal to the required cover.

In all three check-off methods, deflections and settlement of the screeds and screed supports should be considered. This includes differential deflections of exterior and interior steel girders and cantilevered forms due to concrete and strikeoff equipment loading. Check-off method no. 3 for using the strikeoff mechanism is preferred because it reduces the number of corrections to be applied.

5.4.3 To ensure that proper allowances were made for deflections and settlements, it is important to periodically measure the actual cover over the reinforcement during deck placement. Piercing the concrete above the reinforcing bar with a specially marked putty knife is a good checking technique. Metal detection instruments, specifically designed and calibrated for determining depth of cover of reinforcing steel, are commercially available and are suitable for use on fresh or hardened concrete. Most of these instruments function on magnetic principles and might not detect reinforcement comprised of materials other than mild steel.

5.4.4 Population clear cover depth standard deviation has been estimated to approximately 0.375 in. (10 mm), and the population of clear cover is normally distributed (Weed 1974; Pyc et al. 2000). Thus, for an average clear cover of 2.75 in. (70 mm), 2.5% of the reinforcing bar in a bridge deck has a clear cover depth of less than 2 in. (50 mm). The standard deviation of individual bridge decks ranges from 0.15 to 0.50 in. (3.8 to 12.5 mm) (Pyc et al. 2000). Thus, final cover depth acceptance should be based on a clear cover thickness survey. Survey procedures and acceptance limits have been developed (Ministry of Transportation Ontario (MTO) 1998). Payment guidelines can be based on the percent within limits (PWL) calculated from the measured mean and standard deviation as follows (MTO 1998):

> "All lots are greater than 95%; a payment bonus is awarded for lots greater than 98%. For lots less than 95% and greater than or equal to 80%, a payment reduction is instituted. For lots less than 80% and greater than or equal to 65%, additional payment reduction is assessed or lots are subjected to repair to provide adequate capacity and durability. PWL lots less than 65% are not accepted."

5.4.5 Cover depth selection should consider the influence of subsidence cracking over the reinforcing bar. Subsidence cracking occurs while the concrete is in the fresh state and is significantly influenced by cover depth and slump (Dakhil et al. 1975). Other factors that influence subsidence cracking are bar size and spacing (Weyers et al. 1982). For No. 5 (No. 16) bars, the probability in percentage of subsidence cracking is shown in Table 5.4.5 (Dakhil et al. 1975).

Limiting the minimum cover depth to 2 in. (50 mm) and the slump to 3 in. (75 mm) significantly reduces the probability of subsidence cracks directly over the reinforcing bar that, over time, will provide direct access to chloride and carbon dioxide.

5.5—Cleanliness

Before placing the concrete, reinforcement should be free from mud, oil, or other coatings that may adversely affect bonding capacity. Most bare reinforcing steel is coated with either mill scale or, to some degree, rust. Bare steel with rust, mill scale, or a combination of both, is acceptable, provided the minimum dimensions, including height of deformations and weight of a hand wire-brushed test specimen, are not less than the applicable ASTM specification requirements.

5.6—Reinforcement type

The second line of defense in delaying reinforcing steel corrosion is to change the bulk or surface corrosion-resistance properties of the reinforcing steel. Reinforcement for bridge decks can be characterized as black bar, zinc-coated, epoxy-coated, stainless steel (solid and clad), organically coated, solid FRP bars, and other metallic alloyed steel. Durability and structural performance aspects should be considered in the selection of reinforcement type. Carbonation and chloride-induced corrosion should be considered in specific applications. The merits of various metallic reinforcements in resisting chloride corrosion can be assessed with a service-life model. Guidelines and examples on the use of service-life models for bridge decks are present in ACI 365.1R-00 and elsewhere (Cady and Weyers 1983, 1984, 1992; Fitch et. al. 1995; Weyers 1998; Liu and Weyers 1998; Zemajtis et al. 1998; Weyers et al. 1993; Kirkpatrick et al. 2002a,b). The final selection of reinforcement type should be based on minimum life-cycle cost. A life-cycle cost model and examples are provided elsewhere (Weyers et al. 1984, 1993).

Life-cycle cost analysis requires estimated service lives of initial construction and maintenance, repair, and rehabilitation activities over the analysis time period, and the initial construction, maintenance, repair, and rehabilitation costs and an interest rate. The chloride corrosion service-life model requires the concrete chloride diffusion coefficient; chloride corrosion threshold concentration; environmental chloride exposure concentration; time-to-cracking time period from corrosion initiation to cracking and spalling of the cover concrete; and the clear cover depth of the reinforcement defined as the time-to-first maintenance and end-of-functional service life. The carbonation-induced corrosion service-life model requires the same parameters as the chloride corrosion model, except that the chloride parameter is replaced by the carbonation depth of penetration rate and the carbon dioxide concentration in the air at the bridge deck location.

5.6.1 *Bare reinforcement*—Bare reinforcement, or carbon steel, is passive in concrete when the pH of the concrete pore water is above 9 and the chloride content remains below the critical corrosion threshold concentration. An often-quoted acid-soluble chloride corrosion threshold concentration is 0.2% by mass of cement (1.2 lb/yd^3 [0.71 kg/m^3]) for typical bridge deck concrete (Clear 1975, 1976). Others have stated that a good criterion for minimizing the danger of corrosion is a maximum of 0.4% chloride by mass of the cement (Schiessl 1988). Because of the number of influencing factors, such as concrete moisture content, oxygen concentration, temperature, and thickness and integrity of the passive layer, there is no single threshold value applicable to all field structures. Ranges of chloride corrosion threshold levels from field studies ranged 1.0 to 8.6 lb/yd^3 (0.59 to 5.11 kg/m^3) (Stratfull et al. 1975; Vassie 1984; Brown and Weyers 2003) and a low risk of corrosion at 1.2 lb/yd^3 (0.71 kg/m^3), and a high risk at concentrations of 8.6 lb/yd^3 (5.1 kg/m^3). An analysis of field study data suggests that the standard deviation is 2.7 lb/yd^3 (1.6 kg/m^3) and the mean is 5.9 lb/yd^3 (3.5 kg/m^3). For a normal distribution, there would be a probability of corrosion of 1, 4, and 10% for field structures at chloride concentrations of 1.2, 1.5, and 2 lb/yd^3 (0.71, 0.88, and 1.2 kg/m^3), respectively (Vassie 1984; Brown and Weyers 2003). Results of a study of bridge decks suggest that a value of 1 lb/yd^3 (0.59 kg/m^3) of chloride be used (Stratfull et al. 1975). Thus, service-life estimates should consider an acceptable risk of corrosion initiation for the structure under consideration. Corrosion initiation of black bar reinforcement depending on the severity of environmental exposure conditions can be estimated at 10 to 30 years.

Table 5.4.5—Probability (%) of subsidence cracking for given slump and cover

	Slump in. (mm)		
Cover, in. (mm)	2 (50)	3 (75)	4 (100)
3/4 (19)	88	99	100
1 (20)	71	83	95
1-1/2 (40)	34	48	61
2 (50)	2	13	25

5.6.2 *Galvanized steel*—Bright galvanized layer on silicon-free steel rapidly cooled after hot-dipping typically consists of three distinct layers: an η or pure zinc surface layer, followed by increasing zinc-iron layers ζ (5 to 6% iron) and Δ layers (7 to 12% iron). The multi-layer coating is a metallurgically bonded coating that is tough and abrasion resistant. Passivation of the zinc layer occurs in the high alkaline environment of wet concrete when the pure zinc reacts to form calcium hydroxyzincate. A by-product of the reaction is the evolution of hydrogen gas. Chromate treatment of galvanized reinforcing steel prevents the evolution of hydrogen. Galvanized reinforcing steel remains passive at a lower pH than carbon steel, and thus provides protection against carbonation-induced corrosion (Maabn and Sorensen 1986).

Galvanized reinforcing steel extends the chloride-induced corrosion protection service life by extending the time-to-initiate corrosion and the corrosion period from initiation to cracking and spalling of the cover concrete. The corrosion initiation time period is increased by increasing the chloride corrosion initiation concentration. The chloride corrosion initiation concentration for galvanized reinforcing steel is at least 2.5 times (Yeomans 1994a,b), and may be four to five times (Zemajtis et al. 1999) that of bare steel. The corrosion period for galvanized reinforcement is extended because the corrosion products are not as voluminous as iron corrosion products, and the corrosion products may migrate farther from the reinforcement surface than carbon-steel corrosion products (Tonini and Dean 1976; Hoke et al. 1981). The corrosion period for galvanized reinforcing steel is four to five times the equivalent period for carbon steel (Yeomans 1994a). For bridge decks, the corrosion period for black bar—from corrosion initiation to cracking of the cover concrete—is typically 5 years (Liu and Weyers 1998). Thus, the corrosion period for galvanized reinforcement can be 20 to 25 years. For life-cycle cost analyses, repair and rehabilitation methods used for bridge decks constructed with bare steel are also applicable to galvanized steel structures.

5.6.3 *Epoxy-coated reinforcement*—Epoxy-coated reinforcing steel (ECR) was developed under a Federal Highway Administration (FHWA) research project (Clifton et al. 1974). Prescription-based ECR specifications were developed based on project recommendations (Clifton et al. 1974). Primary changes in the ECR prescription specifications have been an increase in coating thickness and a reduction in allowable coating discontinuities, holidays, and surface damage (Weyers 1995). All damage visible to the eye that occurs during fabrication, shipping, and handling at the job site should be repaired with patching material. Laboratory testing of ECR has demonstrated improved corrosion protection for thicker and continuous epoxy coatings (Weyers 1995; Pyc et al. 1998). Additional coating damage, however, takes place during concreting operations (Davis 1990; Clear 1992; Weyers et al. 1997; Pyc et al. 2000).

Bridge deck field studies demonstrated that the ECR prescription-based specifications do not address the corrosion protection failure mode of ECR (Weyers et al. 1997; Pyc et al. 2000). The epoxy coating on the steel reinforcement debonds from the steel surface in as little as 4 years. Chloride penetrates the cover concrete and initiates corrosion at coating damage areas. Corrosion of the reinforcing steel takes place in an acidic environment under the epoxy coating (Weyers et al. 1997; Pyc et al. 2000). The chloride corrosion initiation concentration is the same for bare steel and ECR (McDonald et al. 1998; Brown and Weyers 2003). Thus, ECR corrosion protection service-life extension is in extending the corrosion period from initiation to cracking and spalling of the cover concrete. Field studies have estimated the increase in corrosion period of ECR compared with carbon steel at 1 to 7 years (Weyers et al. 1997; Covino et al. 2000; Clear 1998). A laboratory study using new ECR with a well-bonded epoxy coating estimated the corrosion life extension for ECR at about 14 years (McDonald et al. 1998).

A life-cycle cost analysis for moderate chloride deicing salt exposure conditions demonstrated that ECR needs to extend the corrosion protection service by 10 years to be cost effective when compared with carbon steel (Pyc et al. 2000). If low-permeability concrete is used, ECR is not a cost-effective corrosion protection system (Pyc et al. 2000). The study period was 75 years, and it was assumed that repair and rehabilitation methods used for ECR decks is the same as for carbon steel reinforcement. No effective repair or rehabilitation methods have yet been identified for bridge decks built with ECR.

A research project that consisted of 141 field cores, including 113 ECR and 28 black bar specimens, demonstrated that chloride corrosion initiation and time-to-cracking rates are probability functions for black bar and ECR (Brown and Weyers 2003). Corrosion protection service-life extension provided by ECR in bridge decks was estimated at 4 to 5 years based on damage of 20% of the deck surface (Weyers et al. 2006). This deck damage level is in excess of that which requires an overlay to restore safe driving conditions. These conclusions validated early conclusions that ECR is not a cost-effective corrosion protection for bridge decks. Additionally, the report showed that the chloride concentration at corrosion initiation and cracking of the cover concrete is greater for ECR than black bar, the epoxy coating on the failed specimens was not fully cured in most cases, and the lack of curing correlated with the moisture content of the coating and the degree of cracking visually observed in the scanning electron microscope.

For lesser degrees of coating cure, greater coating moisture content and frequency and density of cracking in the coating were observed. The cracks in the coating occurred for 60% of tested bars, and the cracks were at least four orders of magnitude wider than a water or chloride molecule.

As stated previously, the corrosion mechanism of new ECR in laboratory studies is significantly different than the observed corrosion mechanism for ECR in field structures and field extracted specimens used in laboratory studies. Thus, the field-related studies of ECR corrosion protection performance are significantly more reliable than studies that use new ECR specimens. Not all field-related studies are in agreement with those previously presented. Field studies during the 1990s reported excellent corrosion protection performance for ECR for structures in service for less than 20 years (Gillis and Hagen 1994; Hasan et al. 1995; Perregaux and Brewster 1992; West Virginia DOT 1994; Fanous et al. 2000). Other examples of corrosion damage of structures built with ECR were reported with service-life corrosion protection periods of less than bare reinforcement to an additional 7 years more than bare reinforcement (Smith et al. 1993; Clear 1998). Thus, the designer's decision on whether to use ECR needs to be based on a risk-benefits assessment of the current state of field performance knowledge.

The risk-benefits assessment the designer has to make is the impact of a deck's performance over the life of the structure, which may be 100 years or less. Performance is to be measured in both life-cycle costs and user impact. Life-cycle costs include initial, maintenance, repair, and rehabilitation costs. User impacts include costs associated with vehicle delay and operating costs, and accident increase cost during construction activities. It has been shown for moderate chloride exposure climates, the cost of one concrete overlay within a deck's service life exceeds the life-cycle cost of using stainless steel reinforcement in the original construction (Weyers et al. 2006).

Life-cycle cost analyses for bridge decks built with ECR in more severe chloride exposure environments may or may not produce the same results. Results of these analyses need to be considered against the risks of using ECR compared with other corrosion protection systems.

5.6.4 *Stainless steel*—Stainless steel reinforcement is typically 304LN or 316LN (McDonald et al. 1995; Pedeferri et al. 1997). A number of different grades of stainless steel and other corrosion-resistant metallic reinforcement were developed and are being studied for potential implementation as corrosion-resistant reinforcement (Scully et al. 2003; Clemeña 2002). The chloride corrosion protection for 304LN and 316LN reinforcing bars in concrete was reported to be 3.5 to 5% and 3.5 to 8% by weight of cement, respectively (Pedeferri et al. 1997). For typical bridge deck concrete, this is 22 to 50 lb/yd^3 (13 to 29 kg/m^3). For most exposure conditions, the chloride content at a depth of 2 in. (50 mm) will not exceed 22 lb/yd^3 (13 kg/m^3) in 100 years. For mild chloride exposure conditions, 6.8 lb/yd^3 (4.0 kg/m^3) at a depth of 0.5 in. (13 mm) below the surface, the average bridge deck would have to be overlaid three times if it was constructed with bare steel and a *w/cm* of 0.45 (Pyc et al. 2000).

Stainless steel bars should not be welded. Welding scale significantly decreased the chloride-induced corrosion resistance of welded bar (Pedeferri et al. 1997; Pedeferri 1998). The consequences of galvanic coupling of carbon steel and stainless steel were also shown to be negligible under most situations found in real structures (Pedeferri 1998). Specifications governing stainless steel reinforcement are provided in ASTM A955/A955M-11. Their application in U.S. highway bridges is governed by AASHTO specifications, specifically AASHTO 18M/MP 18-09.

5.6.5 *Fiber-reinforced polymers (FRPs)*—Fiber-reinforced polymers used as concrete reinforcement include both carbon FRP (CFRP) and glass FRP (GFRP) fibers (Hassan et al. 2000; Bradberry 2001). There are a large number of resin types and sizing, or fiber coatings that may be used in the manufacture of FRP reinforcement. The polymer matrix that gives form and provides interlaminar shear strength may be epoxy, polyester, vinyl ester, or blends (Bradberry 2001). Fiber sizing is typically used to improve the interphase between the polymer matrix and the fiber for strength and durability considerations. Combinations of fiber, sizing, and polymer matrix influence the short- and long-term mechanical properties of FRP reinforcement in concrete. The primary value of FRP reinforcement is its electrochemical inertness and resistance to corrosion in chloride-contaminated concrete, although it also has high tensile strength. The designer, however, should consider the long-term residual strength of FRP reinforcement. Factors that influence the strength reduction of FRP in concrete structures include the exposure to moist/wet, high-alkaline (pH 12.5 to 13.3) environment, creep, and freezing and thawing. Care should be used in selecting FRP reinforcement for concrete, particularly GFRP, as some GFRPs are not durable in the alkaline conditions of concrete.

Bridge decks have been designed with top and bottom FRP reinforcement mats (Hassan et al. 2000) or with top mat FRP reinforcement, and bottom mats of either carbon steel (Hassan et al. 2000) or epoxy-coated steel reinforcement (Bradberry 2001). Both design examples did not consider the long-term strength reduction of the FRP (Hanssan et al. 2000; Bradberry 2001). ACI Committee 440 recommends a long-term strength-reduction factor of 30% of ultimate strength. This may or may not be a conservative factor, however, considering the relatively short experience with FRP durability test methods and interpretation of results.

The use of FRP reinforcing materials in bridge decks has matured significantly in the past decade with the publication and use of several consensus guidelines. Fiber-reinforced polymer bars can be purchased from multiple suppliers based on standard material properties. Existing design methodology is used for FRP-reinforced bridge decks with minor adjustments to the design procedure. ACI Committee 440, along with AASHTO and CSA, provides specifications, test methods, design, and construction methods for the use of FRP in bridge decks (ACI 440R-07, ACI 440.1R-06, ACI 440.3R-04, ACI 440.5-08, ACI 440.6-08, AASHTO (2009), and CAN/CSA-S6 (CSA 2006).

5.6.6 *Microcomposite steel*—Microcomposite, multi-structural formable steel (MMFX-II™) has a higher corrosion resistance to chloride than the typical carbon reinforcing steel. The typical chromium content is approximately 9%, and it has previously been classified as an ASTM A615/A615M-09b, Grade 75 (Grade 520) reinforcing steel. The corrosion resistance of MMFX-II™ to chloride has been reported to be 4.5 times that of straight carbon reinforcing steel (Clemeña 2003; Clemeña and Virmani 2004). For bridge decks in Virginia, MMFX-II™ reinforcing steel is the most cost-effective solution compared to bare, ECR, and stainless steel for a 75-year design life (Weyers et al. 2006). Recently, ASTM developed specification ASTM A1035/A1035M-09 for low-carbon, chromium steel reinforcement, with minimum yield strengths of 100 to 120 ksi (690 to 830 MPa). Designers are cautioned that bars of greater yield strength may exhibit different tensile and ductility properties than conventional black bar, which should be accounted for in structural analysis. Welding of such steels may not be recommended (ASTM A1035/A1035-09). AASHTO has issued a specification for uncoated corrosion-resistant bars to be used as concrete reinforcement and dowels in MP 18M/MP 18-09. MP 18M/MP 18-09 permits minimum yield strengths of 60, 75, and 100 ksi (420, 520, and 690 MPa) and grades are designated accordingly (Grades 60 [420], 75 [520], and 100 [690]).

The work of Weyers et al. (2006) consisted of more than 15 years of research conducted in the laboratory, but primarily field performance studies. Service life performance, which is required for life-cycle cost analysis, was determined and validated from the performance of field structures (Kirkpatrick et al. 2002b). Costs were developed from the Virginia DOT bid prices. Subsequent field work confirmed these results (Williamson et al. 2007). These works used and cited work by Clemeña (2003), Clemeña and Virmani (2004), Trejo (2002), and Yeomans (1994b).

CHAPTER 6—PLACING, FINISHING, AND CURING

6.1—Placing

6.1.1 *General considerations*—The procedures outlined in ACI 304R-00 are applicable to the general problem of placing ordinary portland cement concrete under normal weather conditions. Only such additional points are made in this guide, as they are considered peculiar to or especially pertinent in the case of bridge decks. For example, if concrete is to be placed, finished, or cured in hot or cold weather, different methods may be used. Hot weather is any combination of high ambient temperature, high concrete temperature, low relative humidity, high wind speed, and solar radiation that tends to impair the quality of freshly mixed or hardened concrete by accelerating the rate of moisture loss and cement hydration, or otherwise cause detrimental results (ACI 305R-10). Alternatively, cold weather is a period when, for more than three consecutive days, the average daily air temperature is less than 40°F (5°C) and the air temperature is not greater than 50°F (10°C) for more than half of any 24-hour period (ACI 306R-10). Specific information can be found on hot and cold weather conditions and their effects on concrete in ACI 305R-10 and ACI 306R-10, respectively. In addition, concrete bridge decks differ from most concrete placements because of their relatively thin sections, high percentage and close spacing of reinforcing bars, and numerous points of stress reversal. Other considerations include exposure to abrasion, impact, and vibration of traffic.

The construction conditions associated with transporting, placing, finishing, and curing of concrete bridge decks are far from ideal. These conditions all contribute to the difficulties encountered in controlling the quality of the finished deck. Chemicals used to melt ice and snow are known to be aggressive to concrete and steel. In comparison to slabs-on-ground, decks are also subjected to more freezing-and-thawing cycles in the winter, as well as wider temperature variations experienced in the summer.

6.1.1.1 *Temperature controls*—During placing under normal conditions, the concrete temperature should be kept as low as practical to improve placement and structural qualities. Maximum temperatures of 60 to 95°F (16 to 35°C) are normally specified for ordinary portland cement concrete under ideal conditions. Depending on the volume of the placement and the anticipated thermal conditions within the placement, however, these temperatures may fluctuate (ACI 304R-00). Large flat surfaces, such as bridge decks, are prone to plastic shrinkage cracking. Plastic shrinkage cracking is a function of concrete temperature, relative humidity, and wind velocity (Menzel 1954).

During cold weather, with elevated concrete temperatures, decks may experience rapid moisture loss that could cause plastic shrinkage cracks. Therefore, concrete temperature controls should be executed to eliminate this problem. The larger the concrete section, the less rapidly it loses heat; therefore, lower minimum placement temperatures are recommended as concrete sections become larger (ACI 306R-10). For concrete placements of 12 in. (300 mm) and less, 12 to 36 in. (300 to 900 mm), 36 to 72 in. (900 to 1800 mm), and greater than 72 in. (1800 mm), minimum temperatures are 55, 50, 45, and 40°F (13, 10, 7, and 5°C), respectively.

For hot weather, scheduling concrete placements during times of the day or night when weather conditions are favorable is advised for relative ease of handling and placing, and to avoid the risk of plastic shrinkage and thermal cracking. Cold joints, poor consolidation, and uneven surface finishes can result if the concrete is placed faster than it can be properly consolidated and finished. More detailed information on placing concrete in hot weather conditions is found in ACI 305R-10.

6.1.2 *Transportation*—The methods discussed in ACI 304R-00 should be considered with additional stipulations due to the necessity of placing relatively small quantities of ordinary portland cement concrete over a large area. The transporting equipment should be customized to the consistencies of concrete proportioned for the job. For instance, admixtures can be used to improve the workability of concrete, provided that the selected *w/cm* is not exceeded. Some types of truck mixers, bucket gates, or pumps are slow, unworkable, or both, when harsh or very stiff mixtures are used. Approval of every piece of transporting equipment proposed for use on the project should depend on its ability to handle bridge deck concrete without segregation.

The rejection of concrete for a bridge deck often gives rise to a further complication at bulkheads. This results from a high percentage of reinforcement bars that makes the concrete at bulkheads difficult to place and creates the possibility of undesirable cold joints. Time spent checking equipment in advance and checking concrete at the batch plants has proven to be a good investment.

6.1.3 *Rate of delivery*—It is essential that concrete for bridge decks be delivered to the site at a uniform rate to avoid cold joints. Concrete should also be customized to the labor force and equipment that will be used in placing and finishing the bridge deck. On one major project in which specific records were kept, bridge deck concrete delivery was found to average 27.2 yd^3/h (20.8 m^3/h), with a standard deviation of 5.5 yd^3/h (4.2 m^3/h). Sufficient hauling units with at least one spare unit should be determined and established between the concrete producer and the placing and finishing contractor.

The difficulties of obtaining a satisfactory delivery rate can be overcome by mixing on the job. Other methods of mixing concrete, however, can serve equally as well when radio or other methods of communication are maintained between the batch plant and job site.

6.1.4 *Placing equipment*—The movement of concrete from the delivery point to the deck is often a delayed operation that should receive particular attention. Another phase of construction that needs to be observed is when mechanical strikeoff equipment is used and the delivery of concrete to the job site is inadequate. A variety of placing equipment that can assist with this step is available. Although belt conveyors, concrete buckets, and manual or motor-propelled buggies are chiefly equipment of the past, they are occasionally used in special cases. These devices are described in more detail in ACI 304R-00. Currently, pumping is the most dominant procedure used for placing different types of concrete, including ordinary portland cement, high-performance, silica fume, and fiber-reinforced concretes.

6.1.4.1 *Concrete pumps*—In the past, the capacities of pumps used for placing of concrete on bridge decks varied from 20 to 80 yd^3/h (15.3 to 61.2 m^3/h). Currently, the average rate of pumping is 180 yd^3/h (138 m^3/hour), with some pumps attaining a rate of over 200 yd^3/h (153 m^3/h). Rates are dependent on the height of lift, length of horizontal run, and number of pipe elbows used, plus type and size of concrete pump and pipe. Guidelines in ACI 305R-10 cover the attention that pumps require. Delivery of concrete by pumpline is shown in Fig. 6.1.4.1.

Inspection of steel pipes should be required before use. Hardline pipes should be clean and not severely dented. Couplings and gaskets are to be properly designed and capable of withstanding line pressures and surges. Clean couplings and gaskets after each use.

Flexible pipe, when used, should be of such material that kinking will not occur during its use. It should also be constructed so that excessive mortar leakage will not occur at pipe connections. The use of aluminum alloy pipe should be prohibited. The aluminum particles abraded by the aggregate particles will produce hydrogen gas as it reacts with the high-pH cement paste (Whiting and Nagi 1998).

6.1.5 *Vibration and consolidation*—ACI 304R-00 and ACI 309R-05 should be consulted for general requirements related to vibration. Deck requirements differ due to the concrete subsidence being restrained by closely-spaced and chair-supported reinforcing bars, as well as the head of concrete being low. In hot, windy weather, surface crusting is a problem that tends to promote early finishing. This, in turn, forces vibration operations to be completed before the subsidence of the concrete due to bleeding is complete. Occasionally, there is concern that concrete will be overvibrated or overfinished, resulting in more severe deterioration in surface scaling tests (Malisch et al. 1966). This deterioration demonstrates that the concrete was of a consistency so wet that it should not have been vibrated at all. In addition, it could imply that the finishers were working on the drying surface crust an hour or more before bleeding, and subsidence was completed.

It is essential that bridge deck concrete be thoroughly vibrated after the concrete has ceased to subside and late enough to assure close contact with the reinforcing bars. Revibration may be required if bleeding is prolonged, and generally occurs for a much longer time than would be expected. Surface revibration is also an effective method to close flexural cracks on the concrete surface and is effective to a depth of at least 4 in. (100 mm) from the concrete surface (Hilsdorf and Lott 1970). An evaporation reducer might be necessary to delay the time the finishers start, allowing vibration late enough to get proper consolidation. Retarding admixture can delay initial set time and permit later vibration, but cannot prevent surface crusting due to drying. For interim curing, fog sprays are helpful if they provide a true fog. ACI 308R-01 should be consulted for criteria for producing a true fog spray.

Fig. 6.1.4.1—Placement of concrete using a pumpline.

6.1.5.1 *Vibration of FRC*—Special care should be taken when vibrating FRC, as fibers can protrude out of the deck surface. To ensure that fibers will settle within the concrete, mechanical vibration should be considered. For instance, the use of vibrating, roller, or laser screeds consolidate the concrete while providing the required surface elevation of the slab. When using floats and trowels on fiber-reinforced slabs, however, the tools should be kept flat because their edges can cause fibers to spring out of the surface. In addition, burlap drags should not be used when texturing FRC, as they tend to lift the fibers and tear the concrete surface. ACI 544.3R-08 should be consulted for specific information on vibrating and consolidating FRC.

6.1.6 *Sequence of placing*—Concrete should be placed in a uniform heading in a line roughly parallel to the screed machine. Cracking can sometimes be reduced in continuous bridge decks by placing the concrete in a sequence designed to minimize the effect of form and falsework deflections. While this procedure is not as widely practiced as in the past, it is worth consideration. Several days might, however, be added to the time necessary for deck construction. Placing the center portions of the spans first reduces cracking created by the negative bending moment over the piers. Although concrete can also be placed longitudinally, this method could present design, cracking, and durability issues.

6.1.7 *Labor requirements and qualifications*—Arrangements should be made to ensure that sufficient competent labor is on hand to proceed properly with a concrete deck placement.

6.1.7.1 Labor requirements for deck placement vary according to the experience of the workers; the surface area of the placement; the placing and strikeoff equipment to be used; weather conditions; the speed of concrete delivery, including delivery from the batching area to the job site and from the delivery equipment to the deck forms; and the initial and final curing operations. Although crew sizes may vary, an example deck placement crew might consist of a paver operator, eight laborers, six finishers, and a foreman.

Fig. 6.1.8(a)—Vertical cracking in deck concrete aligned with reinforcing bar (right).

Fig. 6.1.8(b)—Horizontal and vertical cracking aligned with reinforcing bar in concrete sample extracted from a bridge deck.

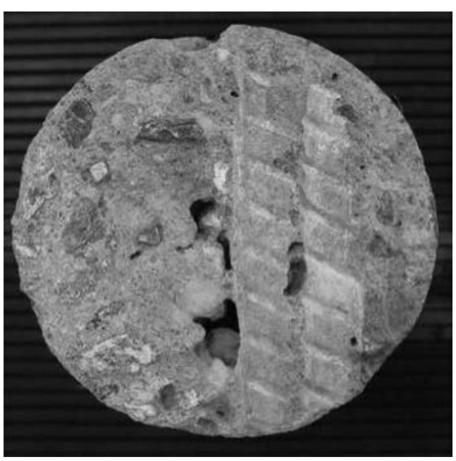

Fig. 6.1.8(c)—Evidence of entrapped air voids along the trace of lap-spliced reinforcement.

6.1.7.2 Minimum labor requirements are often established by union rules and maximum labor is a fundamental prerogative of contractors. Labor limits should not be set forth in the specifications. The judgment of an experienced supervisor is valuable in establishing labor requirements.

6.1.7.3 The individual on the contractor's force responsible for deck concreting should have a minimum of 2 years of experience for simple span bridges with lengths less than 100 ft (30.3 m) and skewed no more than 5 degrees from normal, and 5 years of experience for all other types of bridges.

6.1.8 *Special care of reinforcement during placing*—If reinforcing bars are properly positioned and securely tied, the freedom from spalling of a bridge deck may largely depend on the degree to which the bars are tightly encased in concrete. This should be done without cracks over bars (Fig. 6.1.8(a)) or horizontal cracks starting at the bars (Fig. 6.1.8(b)), or without voids or water channels along the bars. Because decks usually have closely-spaced bars, particularly where there are splices, caution should be taken that voids or cracks along bars (Fig. 6.1.8(c)) do not develop during bleeding and subsidence of the concrete. The best results are obtained with mixtures with low *w*/*cm*, ample and repeated vibration, and where finishing is delayed as long as possible.

6.2—Finishing

6.2.1 *General*—With respect to durability and riding quality, the most complex and important phase of bridge deck construction is finishing. Because decks are applied to support structures that are elevated and spanning considerable distances, those supporting members may flex during the placement and curing of the new deck concrete, requiring special construction techniques and controls (ACI 304R-00, ACI 305R-10, ACI 306R-10, ACI 308R-01, and "AASHTO LRFD Bridge Construction Specifications" (AASHTO 2004)).

After the concrete has been consolidated by vibration and struck off by machine, it should be further smoothed and consolidated with a longitudinal float of a suitable design. Weather conditions should also be considered when finishing, especially when using concrete containing SCMs. ACI 305R-10, ACI 306R-10, and ACI 308R-01 can be referred to for hot and cold weather finishing procedures. Curing is integral with finishing operations. Initial curing through the use of fog sprays or monomolecular films are often required (ACI 308R-01).

6.2.1.1 *Roughness*—Roughness can be periodic and varying in wavelength or it may occur as discrete discontinuities. Excessive sag and camber are deficiencies that cause long wavelength roughness. Roughness with short wavelength can appear early and result from inadequate cover over reinforcement or improper finishing. Roughness may develop subsequently with surface deterioration. Such short wavelength roughness may be periodic or random, depending on its cause.

Discontinuities at expansion joints or near abutment back walls result in sudden bumps. The bump may not be on the bridge or be caused by the bridge, but may be the result of pavement settlement at the back wall. Following the floating operation but while the concrete is still plastic, the contractor should test the slab surface for trueness with a straightedge 10 to 16 ft (3 to 5 m) long. The straightedge should be used to check the surface for bumps or depressions, and advanced along the deck in successive stages of not more than one-half the length of the straightedge, and the straightedge worked perpendicular to the direction of the placing screed. Any depressions should be filled immediately with freshly mixed concrete, struck off, consolidated, and refinished. High areas should be cut down and refinished.

6.2.1.2 *Finishing silica fume concretes*—Because of its physical properties, silica fume concrete requires different finishing operations than ordinary portland cement concrete. For instance, the absence of bleed water and adhesive nature of concrete with high silica fume dosages, which are 10 to 20% by weight, affects the screeding and troweling of slab surfaces and contributes to plastic shrinkage cracking. Plastic shrinkage cracking may be avoided by using proper initial finishing methods in conjunction with fog sprays (ACI 305R-10 and ACI 308R-01). Therefore, proper finishing precautions should be taken to avoid such problems. A general approach to finishing silica fume concrete is to underfinish, as opposed to overfinish, the concrete surface (Holland 1987). The minimum required finish is the best approach because it provides the most resistance to aggressive freezing-and-thawing and chemical environments (ACI 234R-06).

6.2.1.3 *Finishing FRC*—Only minor alterations in ordinary portland cement concrete procedures are needed to finish steel FRC. For flat-formed surfaces, no special attention is generally required. If chamfers or rounds have been provided at the edges and in corners, specific placing methods found in ACI 544.3R-08 should be conducted. Like silica fume concrete, care should be taken not to overwork the surface. Overworking may result in fine cracking in the surface of the deck.

6.2.1.4 *Finishing high-performance concrete*—High-performance concrete sets rapidly and, therefore, plans should incorporate solutions for last-minute problems when placing. Difficulties that delay the work can seriously affect the quality and characteristics of the concrete. Consequently, initial preparations should be made to transport, place, consolidate, and finish the concrete at the fastest possible rate (ACI 363R-10). Finishing problems may be prevented through the use of proper initial curing methods as fog sprays or monomolecular films (ACI 305R-10 and ACI 308R-01).

6.2.2 *Timing of operations*—The entire plan of operation, placing and finishing times, and the equipment of the contractor should be evaluated to ensure that the operation can be performed smoothly and efficiently. This phase should be carried out during the preconstruction meeting.

Final floating should be delayed as long as possible to allow for completion of bleeding of the concrete. This is necessary to prevent crusting, which is the formation of a weakened plane immediately below the finished surface. Crusting produces rapid scaling when the deck surface is exposed to deicers and freezing-and-thawing action.

6.2.3 *Manual methods*—Manual methods of strikeoff should not be used except where the use of a finishing machine is impractical or impossible. The manual method could be used on variable width sections or to finish adjacent to a temporary bulkhead. A typical scenario would be if the concrete was already deposited in the event of a breakdown of the mechanical finisher. When allowed, a manual strikeoff should be accomplished with a steel or steel-shod wood screed.

Fig. 6.2.3—Longitudinal floating of bridge deck with bull float.

Floating may be done manually or mechanically. Manual methods are commonly employed using plow-handled floats and long-handled bull floats. Both methods are used from work platforms that span the deck transversely, as shown in Fig. 6.2.3. Proper finishing, using manual methods, requires the skills of an experienced pavement finisher.

6.2.4 *Finishing aids*—The practice of sprinkling the struck surface of the deck to facilitate floating should be strictly prohibited. This practice may produce a surface that has an excessively high *w/cm* and low entrained air content. These conditions will contribute to rapid surface deterioration under the actions of traffic, freezing and thawing, and deicing chemicals.

To aid the finishing operations (floating), especially under hot, dry conditions, a monomolecular filming agent can be applied to the struck surface (Cordon and Thorpe 1965). The purpose of the filming agent is to prevent rapid evaporation of bleed water that can produce plastic shrinkage or crusting. Filming agents extend the period of time during which finishing operations can be carried out.

6.2.5 *Mechanical equipment*

6.2.5.1 Machinery used in the finishing of concrete placed on bridge decks consists of several types. Nomenclature varies because it is possible to describe this equipment in terms of either its direction of travel or the orientation of the striations imparted to the surface. Because the direction of motion and the orientation of the striations may be perpendicular to each other, the potential for conflicting nomenclature is apparent. For the purposes of this standard practice, the direction of striations will be used to designate the machine as longitudinal or transverse. The direction of travel of the entire machine will be used for secondary identification. The latter feature dictates the geometry of placement and, thus, influences progressive deflections. Depending on the specific design of the equipment, the motion of the strikeoff plate may not coincide with the direction of the entire machine.

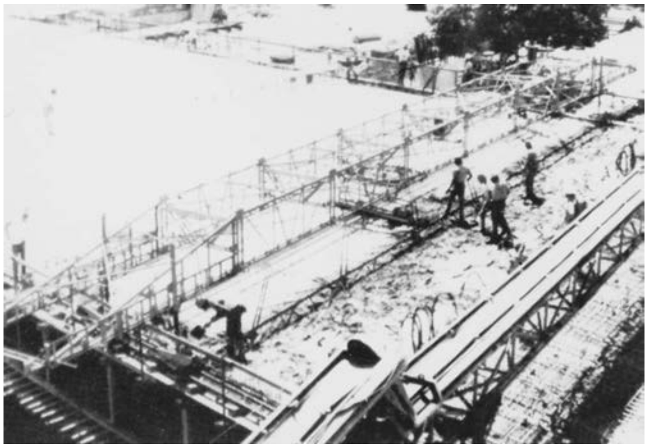

Fig. 6.2.5.1.1(a)—Longitudinal travel, longitudinal finish screed. Note the use of conveyor in foreground and work bridge behind screed for curing and other minor activities.

Fig. 6.2.5.1.1(b)—Close-up of longitudinal travel, longitudinal finish screed. Equipment is moving toward camera.

6.2.5.1.1 *Longitudinal finish, longitudinal travel*—Most commonly used is the combination strikeoff and finishing machine shown in Fig. 6.2.5.1.1(a) and (b). Supported in a structural frame, it is self-propelled on rails, and travels in a longitudinal direction parallel with traffic flow. Strikeoff and finishing machinery, which is suspended from this frame, is power-driven to perform the task of strikeoff and finishing to the established tolerances. The finishing is accomplished in a longitudinal direction as the power-driven vibrating or oscillating screed (float), or both, travels transversely across the deck.

6.2.5.1.2 *Transverse finish, longitudinal travel*—Another type of machine is supported on longitudinal rails and travels in the direction of traffic flow. This type of finish is accomplished by the transverse action of the power-driven vibrating or oscillating screed, or both. Strikeoff of fresh concrete is obtained through a strikeoff plate attached ahead of the finishing screed, moving placed concrete longitudinally. An example is shown in Fig. 6.2.5.1.2(a) with a closer view of a similar apparatus shown in Fig. 6.2.5.1.2(b).

6.2.5.1.3 *Longitudinal finish, transverse travel*—The frame supporting the strikeoff and finishing machinery is

Fig. 6.2.5.1.2(a)—Screed supported by longitudinal rails with transverse moving finishing element.

Fig. 6.2.5.1.2(b)—Close-up of transverse finishing element on screed.

mounted on rails placed transversely at a 90-degree angle to traffic flow or on adjacent decks. The strikeoff travels longitudinally like traffic flow; power-driven finishing is performed by a longitudinal oscillating screed while the machine travels transversely across the deck. An example is shown in Fig. 6.2.5.1.3.

6.2.5.1.4 Regardless of the type of equipment used, freshly placed concrete should be distributed uniformly ahead of the strikeoff and finishing machine and placed as close to its final position as practicable. Concrete should not be moved horizontally with vibrators or by other methods that cause segregation.

6.2.5.2 *Rails and guides*

6.2.5.2.1 *Equipment traveling longitudinally*—Adjustable screed supports provide the initial surfacing control and sets the final longitudinal profile. Therefore, it should be set to proper elevation with allowance for anticipated settlement, camber, and deflection of falsework. The elevation is required to form a bridge roadway deck true to the required grade and cross sections. The screed supports

Fig. 6.2.5.1.3—*Transverse travel, longitudinal finish screed.*

should be vertically adjustable and set by instrument. Temporary supports should be removable, with minimum disturbance of the concrete. The rails should be set above finished grade and extend beyond both ends of the scheduled length for concrete placement. This distance should be sufficient to permit the float of the finishing machine to fully clear the freshly placed concrete.

Figure 6.2.5.2.1 shows an idealized arrangement for a bridge deck strikeoff machine designed to travel longitudinally and incorporating several important features, including:

- Screed rail supports that are placed in an unfinished area requiring later concrete cover;
- Adjustable supports to allow for progressive deflections; and
- Screed rails located above the finished surface to avoid significantly disturbing the concrete when the rail is removed.

6.2.5.2.2 *Dead load deflections*—The issue of beam deflections during concreting poses a difficult problem for good bridge deck finishing. All beam deflections should be carefully calculated and compared at the deflection control points. Progressive longitudinal deflections should be carefully considered as concreting proceeds down the length of the span.

The problem of progressive deflections on a typical beam is illustrated by the deflection lines in Fig. 6.2.5.2.2, which are grossly exaggerated for clarity, for various conditions of loading. Screed rails should initially be set coincident with Line 1. If the rails become disturbed, they will require adjustment as the work progresses. Examples of variations similar to those shown in Lines 2, 3, or 4 should be considered in establishing the final grade lines. Note that, except for Lines 1 and 5, 1/4- and 3/4-point deflections are not equal.

The problem of transverse differential deflections is far more difficult to correct and cannot be precisely resolved in contemporary practice. Most fascia beams deflect less than interior beams; however, it is on these beams that screed rails are usually supported. Consequently, cross-slopes are altered as the beams are loaded. These differentials are usually greatest at midspan and nonexistent at span ends. Therefore, if complete deflection calculations are not available, it is best

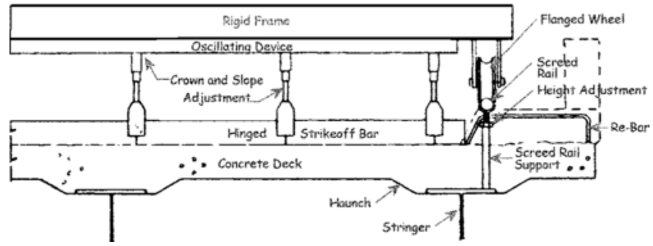

Fig. 6.2.5.2.1—Idealized arrangement of longitudinal travel, transverse finish equipment.

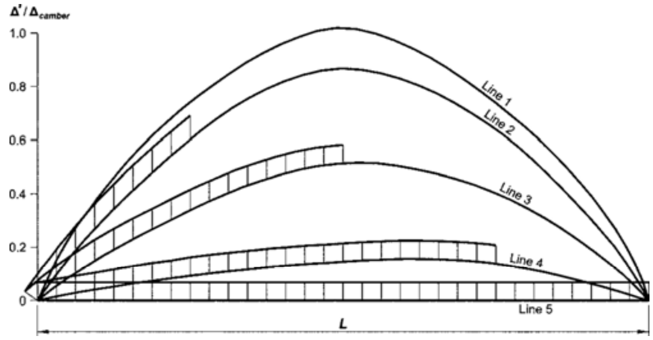

Fig. 6.2.5.2.2—Typical deflection characteristics of beam at various stages of loading by a screed traveling longitudinally; blocking indicates applied load (concrete placed).

to use the cross-sloped configuration of the span ends to ensure adequate deck thickness and sufficient cover over the reinforcement bar.

On sharply skewed bridges, the problem becomes considerably more complex, and consultation with the designer is advised before concreting begins. The finishing machine, when possible, should be set parallel in the skew of the bridge to avoid differential deflection on multi-girder bridges. On short spans or any relatively rigid spans with minor deflections, the problem can be ignored.

Special consideration should be given to bridges constructed in stages. Closure pours should not be placed until all formwork has been removed from the previously placed decks, or the theoretical adjusted deck elevations modified accordingly. Deflections and rotations from unbalanced loads during staged construction should also be accounted for.

6.2.5.2.3 *Equipment traveling transversely*—This type of machine is most often used on simple spans of 100 ft (30 m) or less, though it has been used on spans of greater length. The transverse screed rails supporting the machine are normally set to the finished grade at each end of the span. The finished elevation of intermediate points on the deck is set on the longitudinal strikeoff edge of the screeding machine. Assuming structural stability of the machine, these elevations remain fixed and are independent of the girder deflections occurring during concrete placement. Consequently, the thickness of the concrete deck is dependent on two major factors that should be recognized during construction. They are:

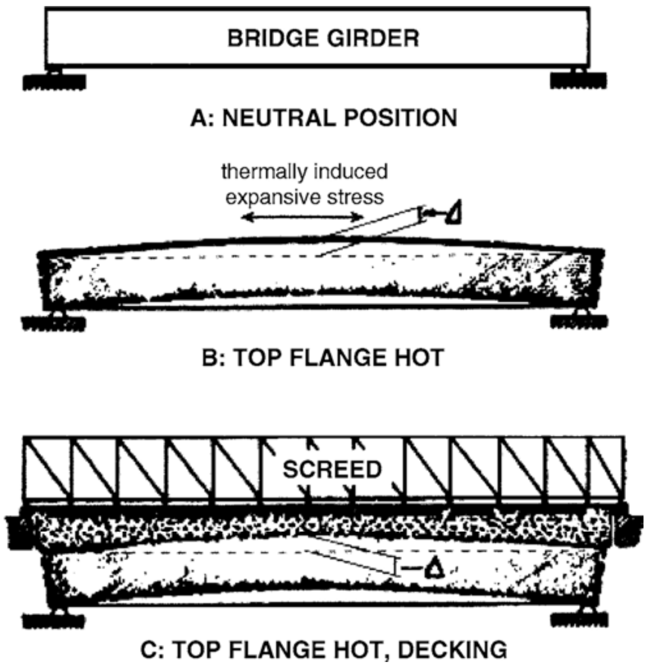

Fig. 6.2.5.2.3(a)—Effect of differential temperatures on deck screeded with a screed traveling transversely.

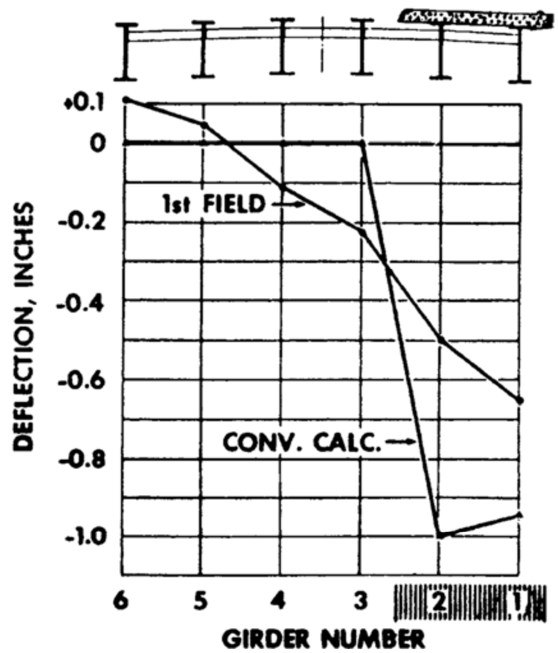

Fig. 6.2.5.2.3(b)—Comparison of conventionally calculated deflections with that measured in the field on a bridge finished with a screed traveling transversely.

1. The differential temperatures existing between the top and bottom flanges of the girders during concrete placement, as opposed to those that may have existed when the forming elevations were established; and
2. The transverse position of the concrete dead loading at the time a final screeding pass is made over a given point on the span.

The possible influence of differential temperatures is illustrated in Fig. 6.2.5.2.3(a). If no temperature differential between the top and bottom flanges exists, it would be in a thermally neutral position (Fig. 6.2.5.2.3(a)A). Due to solar radiation, differential temperatures will generate expansive forces in the upper flange, which are resisted by opposing forces in the lower flange. These deflections are more pronounced with longer span lengths, type, orientation and color of girders, weather conditions, and the time of year. The resulting effect is an upward deflection of the girder (Fig. 6.2.5.2.3(a)B). If the deck forms were established to grades complying with the neutral position of the girder, but the concrete deck was screeded to grade under differential thermal conditions, the thickness of the deck would be decreased (Fig. 6.2.5.2.3(a)C)).

The influence of concrete dead loading is illustrated in Fig. 6.2.5.2.3(b). Conventional design procedures for calculating dead-load deflections normally assume that each girder is free to deflect independently of other girders in a bridge span. However, under partial transverse loading conditions, such as the example shown, the conventional calculation method yields a midspan transverse deflection pattern markedly different from the actual field deflection pattern. Thus, if the concrete was struck off to grade over the first girder, the midspan deck thickness at this point would be decreased by the difference between the two deflection curves. In addition, the finished grade at this point will be low by an identical amount when all the deck concrete is placed.

Neither one of the two major factors discussed can be exactly compensated for during construction. However, their effects can be minimized by observing two practices:

1. Establish forming elevations when the thermal conditions on the girders approximates those anticipated at the time of concrete placement, or adjust the deck forms vertically at a time when the thermal condition of the girders approximates the condition expected to prevail at the time of concrete placement, or both. The precaution for vertical adjustment is most important because the in-place forming will shield the lower portion of the girders from solar radiation. Differential thermal effects can be virtually negated, of course, by concreting very early or late in the day; and
2. Delay the final strikeoff pass of the screeding machine over any given area at least three girder spaces, or preferably more, behind concrete placement. For exceptionally wide roadway widths of more than the equivalent of three 12 ft (3.7 m) traffic lanes, the bridge designer should be consulted.

6.2.5.3 Work platforms are commonly employed to aid in finishing operations. They typically span the deck in the direction parallel to the finishing machine and employ the same rails to facilitate movement. They should be required when manual finishing methods are used.

6.2.6 *Texturing*

6.2.6.1 Decks with deep surface textures will retain skid resistance longer than those with shallower textures. Satisfactory textures can be produced by rake, wire brooming, tining, and saw-cutting after curing (ACI 325.6R-88).

6.2.6.2 After the concrete has been brought to the required grade, contour, and smoothness, the texture should be applied. At the current state of technology, a broom finish may be the most practical method for obtaining a satisfactory texture. Wire brooms with bristles spaced to give a coarse texture are preferable. Due to the importance of securing proper drainage, transverse ridges are preferable, unless minimization of noise is important.

The broom strokes should be square across the slab from edge to edge, with adjacent strokes slightly overlapped. Brooming may be obtained either manually by skilled workers who pull it across the surface from a work platform, or mechanically by self-powered machinery traveling longitudinally with the power-driven broom moving transversely. Raking is done in a similar fashion as brooming, with rake tines that are stiffer than the wire tines. Machine tining is done with stiff wires. After hardening, texturing is done by saw-cutting. Saw-cutting has typically replaced tining due to surface raveling that occurs during tining.

6.2.6.3 Texturing should not be done on deck surfaces that are to be sealed with a waterproofing membrane.

6.2.6.4 *Texturing steel fiber*—Deck surfaces with steel fibers may be textured by brooming to provide a skid-resistant surface. Although another method is raking, caution should be taken during the process as fibers can be drawn out of the surface, thereby decreasing the structural integrity of the deck.

6.2.7 *Correction of defects*—After the first pass of the finishing machine, additional concrete should be added to honeycombed and low spots, then struck off again. Honeycombed and low areas should not be eliminated by tamping or grouting. The surface of finished concrete after floating should be checked with a 10 ft (3 m) straightedge that is placed parallel to the roadway centerline and at several positions from one edge of the deck to the other before moving to the next location. Successive locations should not exceed one-half the length of the straightedge. Depressions found should immediately be filled with fresh concrete, revibrated, struck off, and refinished. Any areas not corrected in the manner described previously may have to be corrected by grinding at a considerably greater cost later, and with attendant loss of surface texture.

6.2.8 *Poor skid resistance*—Surface friction measurements of highway pavements in the United States are typically made using a locked-wheel skid trailer that meets the requirements of ASTM E274/E274M-11. This procedure measures the frictional force on a locked test wheel as it is dragged over a wet pavement surface under constant load and at a constant speed, with its major plane parallel to the direction of motion and perpendicular to the pavement. The standard reference speed is usually 40 mph (64 km/h), and the results are expressed as a skid number, also known as friction number (FN) according to AASHTO terminology. Requirements vary by jurisdiction and highway classification, but generally pavement intervention is indicated when friction numbers fall below 30 under a full-scale test with either a smooth or a ribbed tire at a speed of 40 mph (64 km/h), designated SN40S or SN40R, respectively (Henry 2000). A well-textured new or rehabilitated pavement may have friction numbers (SN40R) above 60.

The FN of the bridge deck surface should not differ substantially from the pavement segments that it connects, and should have and retain the minimum value established for pavement surfaces. Published data for bridge decks are meager, but those available for pavements indicate that low skid resistance can be influenced by materials and construction practices, and by subsequently applied coatings. An example of a surface polished by heavy traffic is shown in Fig. 4.12. A detailed treatment of skid resistance and pavement roughness was reported by Henry (2000). More recent research has correlated microtexture and macrotexture of pavement surfaces using profilometry to determine an International Friction Index (IFI), and efforts have been made to correlate these data to skid numbers or friction numbers (Kowalski et al. 2010).

6.3—Curing

6.3.1 *General considerations*—The first few hours and days in the life of a concrete deck are critical where cracking, strength, and durability are concerned. A rapid increase in quality during this period, commonly referred to as the curing period, requires temperatures to be greater than 50°F (10°C) and with little or no loss of mixing water.

To ensure continued hydration at the optimum rate for a given temperature, the cement paste should be kept as nearly saturated as possible. Water should be available to compensate for evaporation from the surface, and to replenish water removed from the pores by the chemical process called self-desiccation. For a typical mixture, the amount of water needed during the first week to replenish depletion due to self-desiccation is about one part water to 24 parts cement by weight. Evaporation of curing water, however, reduces the deck surface temperature. ACI 308.1-98 states that the concrete temperature should not be permitted to decrease at a rate greater than 5°F (3°C) per hour during the initial 24-hour period after casting.

Particular attention should be given to the equipment that will be used to accomplish the cure. All equipment and facilities should be ready so that the curing may begin immediately as soon as the concrete is ready.

Temperature and weather should also be considered when curing concrete. For hot weather conditions, attempts should continue to protect the concrete from drying conditions that are conducive to higher evaporation. ACI 305R-10 recognizes that “Drying conditions can also occur at low ambient temperature, with slower set times, lower relative humidity and wind, all of which are conducive to higher evaporation.” The concrete should be kept in a uniformly moderate temperature condition. Lack of adequate precautionary measures to protect concrete can affect the surface and concrete quality. Adequate protection of the concrete should be accomplished by using methods and materials described in ACI 305R-10, ACI 306R-10, and ACI 308R-01 for hot and cold conditions. Concrete should be protected from drying too rapidly so adequate hydration can occur. When concrete that is warmer than 60°F (16°C) is exposed to air at 50°F (10°C) or higher, the preferred technique is to use steam for both heating and preventing excessive evaporation

(ACI 306R-10). The concrete can be exposed to the air when the air temperature within the enclosure has fallen to 50°F (10°C), provided that the relative humidity is not less than 40%. If the humidity is low, moisture should be added to heated air to increase the humidity.

There are many different spray-on compounds that are available for curing concrete, including newer products that are intended to address the environmental concerns associated with high volatile organic compound (VOC) contents. Although concrete cured with water or plastic sheeting has been found to yield the best results when retaining water for hydration, promoting concrete strength, and reducing permeability (Whiting and Snyder 2003), there are many other methods that may be used for special situations and can be found in ACI 308R-01.

6.3.1.1 *Silica fume concrete*—Because of the typically low water content and *w*/*cm*, silica fume concrete requires additional attention to curing as compared to ordinary portland cement concrete. To obtain the full benefits of silica fume concrete, initial curing procedures found in ACI 308R-01, including the use of fog sprays, and proper curing procedures found in ACI 234R-06 should be followed.

6.3.1.2 *High-performance concrete*—It is highly recommended that high-performance concrete be cured as soon as the concrete has been finished, beginning with initial curing described in ACI 308R-01 and the use of fog sprays, or severe cracking will be inevitable (Bickley and Mitchell 2001). Cracking will occur because of the low *w*/*cm* used in the concrete. Total immersion of the finished concrete unit in water is the preferred method of curing, but there are many other processes stated in ACI 363R-10, such as ponding, fog spraying, and sprinkling. For high-performance concrete to attain its full strength, it should be cured for a minimum of 14 days (Nassif and Suksawang 2002).

6.3.1.3 *Fiber-reinforced concrete*—Fiber-reinforced concrete should be cured similarly to ordinary portland cement concrete with the exception of constant observation of temperature change and weather conditions. Otherwise, plastic shrinkage cracking will occur, as FRC is placed in thin sections and has high cement content. If extreme conditions occur, placements should be shaded from the sun and sheltered from the wind for protection.

6.3.2 *Curing methods*—An ideal curing medium or agent will prevent any substantial loss of moisture. Unfortunately, there is no ideal curing agent, but a number of methods by which concrete decks can be kept in a moist condition and at a favorable temperature. The most popular methods supply additional moisture to the surface with continuous application of water, minimize moisture loss by sealing the surface with a membrane-curing compound, or cover the surface with a moisture barrier material. The preferred method for a low-*w*/*cm* concrete used for bridge decks is continuous water curing. ACI 308.1-98 states that the temperature of the curing water should not be lower than 20°F (10°C) cooler than the surface temperature of the concrete at the time the water and concrete come in contact.

6.3.2.1 *Continuous water curing*—Continuous water curing can be maintained by a continuous spray, ponded water on the surface, or by a surface covering of absorbent material that is kept saturated.

When the continuous water method is used to cure concrete, it is essential that the surface of the concrete not be allowed to dry out once the curing period begins. Continuity is important because of volume changes due to alternate wet and dry periods. This promotes the development of pattern cracking. The need for continuous curing is greatest during the first few hours after placement.

Prewetting moisture-retaining material before it is placed ensures an ideal weight. If placed dry, there is danger that absorption of water from the deck will cause surface damage. To minimize the change for damage, the deck surface should be thoroughly wet before placing the material. The material should also be thoroughly wet when placed and placed in a manner that it does not mar the surface.

6.3.2.2 *Membrane curing*—There are three advantages of membrane curing over continuous water curing. Membrane curing is:

1. Generally applied earlier;
2. Not cut off sharply; and
3. Extended over a much longer period.

There are two primary disadvantages of membrane curing. Membrane curing:

1. Is an attempt to prevent evaporation, without replacing water consumed by hydration and evaporation; and
2. Does not offer the cooling effect afforded by continuous water curing.

For hot weather concreting, white pigmented curing compounds are preferred over clear or lightly tinted compounds because they allow less heat to build up from solar radiation and offer better visual evidence of uniform application.

Only curing compounds that meet the requirements of ASTM C309-07 should be used on bridge deck concrete. Because of the lower allowable water loss, compounds that meet the requirements of federal specifications are preferable (U.S. Army Corps of Engineers 1990). Curing compound coverage and rate of application are critical to curing efficiency. Application rate varies with product; some products require twice the material as others.

6.3.2.3 *Sheet materials*—Curing by materials such as plastic sheets, water-resistant paper, and plastic bonded to absorbent is effective only if the deck surface is thoroughly wet down. This should occur just before the barrier material is laid. Air is not permitted to circulate under the material. The moisture barrier curing may be difficult to control in windy areas, but surfaces should be protected properly by securing the moisture barrier.

Plastic curing sheets are often combined with a wet absorbent, such as burlap. The plastic sheet reduces evaporation, and the wet absorbent adds water and thus reduces the surface temperature to desirable ranges that prevent cracking. ACI 308.1-98 recommends minimum 4 mil (0.10 mm) polymer sheet that conforms to ASTM C171-07. The sheet color should be black if the daily high ambient temperature is less than 60°F (15°C), white if greater than 85°F (30°C), and transparent if the temperature is between 60 and 85°F (15 and 30°C).

6.3.3 *Time of application*

6.3.3.1 When placing deck concrete in hot weather, it is necessary to keep the operation confined to a small area. The application should proceed on a front, with the minimum exposure surface against which concrete is to be added. A fog nozzle should be used generously to cool the air, the forms, and steel immediately ahead. This is necessary to lessen rapid evaporation from the concrete surface before and after each finishing operation. Excessive fog spraying, which washes the fresh concrete surface or causes water to stand on the surface during floating or troweling, should be avoided.

Without fog spray between the finishing operations in hot weather, particularly if it is windy and humidity is low, water may evaporate from the surface faster than it will rise naturally to the surface through bleeding, creating growing tension in the surface that often causes irregular, plastic shrinkage cracking.

6.3.3.2 Membrane curing should begin as soon as the bleed water sheen leaves the concrete surface, but before the surface dries. If the sheen is not uniform over the area or if for some reason application of the membrane curing is delayed, the surface should be kept wet by fogging. The surface should be damp when the membrane material is placed.

6.3.3.3 When the temperature is near freezing, the use of plastic coverings, wet burlap, water-resistant paper covering, or similar curing methods should be employed. Refer to ACI 306R-10 for further information.

6.3.4 *Duration*—The period of positive or controlled curing that follows the setting of concrete is intended to ensure obtaining a reasonable strength at an early age and prevent the formation of surface cracks due to rapid loss of water while the concrete is low in strength. For bridge decks, the minimum curing period should be no less than 7 days.

In cold weather concreting, if the temperature is less than 50°F (10°C), the curing period should be extended if heat is not applied to the concrete. An extended curing period or heat is especially important for increasing the strength of a deck over the supports of continuous structures, thereby minimizing stress cracking when the span falsework is struck.

6.3.5 *Cracking*—After construction, many concrete bridge decks develop transverse cracks that may increase maintenance costs and shorten the service life of a bridge. Also, cracks can increase the rate of corrosion of steel reinforcing bars, deteriorate concrete, damage components beneath the deck, and damage aesthetics. Modifications to current methods of bridge design, material selection, and construction techniques can be taken to reduce the number of transverse cracks in bridge decks (Krauss and Rogalla 1996). For example, engineers can calculate the proper design to reduce stress within different girder types and styles, allowing for careful selection of construction materials. Selecting a concrete that has a low modulus of elasticity, high creep, low coefficient of thermal expansion, low heat of hydration, and high thermal conductivity allows the control of shrinkage and thermal strains that cause stresses. During bridge construction, the major cause of cracking is the curing process (Krauss and Rogalla 1996), as plastic shrinkage cracks occur when there is not enough water for the hydration of the cement. While all concretes are prone to this cracking, high-performance concretes are especially sensitive to water loss and poor curing practices due to their chemical composure. Therefore, plastic shrinkage cracking is more likely to occur in high-performance concrete. It has been discovered that the application of burlap or mats 10 to 15 minutes after concrete placement should be carried out (Praul 2001) for crack reduction. Although the burlap may leave indentations or impressions in the fresh concrete, it will achieve enhanced durability. The Maine DOT experimented with the use of concrete that contained a pozzolan as a proposed replacement for granite curbing (Praul 2001). It was found that the sections extruded, sprayed with curing compound, finished, and then covered with wet burlap displayed cracks every 3 ft (0.9 m). Alternatively, sections that were immediately covered and then finished by removing isolated areas of the cover exhibited cracks every 15 ft (4.6 m). Therefore, it was illustrated that a longer period of immediate wet curing produced higher-quality concrete (Darwin 2003). This method should be analyzed to determine whether the long-term benefits of extended curing outweigh the added costs of longer placing and finishing operations.

6.3.6 *Scaling*—Scaling, such as that shown in Fig. 4.10, is loss of surface mortar, and is usually associated with the use of deicer chemicals. Severity is normally expressed qualitatively by terms such as light, medium, heavy, or severe. Gradual loss of surface by abrasion is sometimes difficult to distinguish from scaling. Scaling can be locally severe but, in the absence of studded tires, is generally not a serious problem if accepted concreting practices are followed.

6.3.7 *Related information*—Further information on curing can be found in ACI 308R-01, ACI 305R-10, and ACI 306R-10.

CHAPTER 7—OVERLAYS

7.1—Scope

Chapter 7 discusses overlays placed on a cured bridge deck as a wearing and protective shield against water, chemicals, abrasion, or low skid resistance. Concrete overlays are usually 1.25 in. (30 mm) or more in thickness, and polymer overlays may be as thin as 0.25 in. (6 mm). This chapter does not include considerations of coatings and penetrating sealers, such as silanes, used to inhibit chloride penetration.

Throughout Chapter 7, no distinction will be made as to the age of the bridge at the time of overlay placement. Also, no attempt will be made to discuss the relative merits of various overlays to prevent deterioration of concrete bridge decks, as it is assumed that overlays will only be placed on structurally sound surfaces, regardless of age.

7.2—Need for overlays

7.2.1 *Waterproof barrier*—The primary reason for the use of overlays is the prevention and repair of spalling on concrete bridge decks. Such spalling is the result of expansive forces built up within the deck concrete by the products of corrosion of reinforcement steel. Such corrosion is advanced by the presence of moisture and chlorides. Cracks over the reinforcement or porous concrete can accelerate the rate of deterioration. Thus, where cracks or porous concrete are evident and deicers are used, some type of waterproof barrier should be provided, or spalling can be anticipated.

Again, it is reiterated that careful attention to good design and construction practices, as set forth elsewhere in this standard practice, should significantly reduce the propagation of cracks and prevent the acceptance of low-quality concrete. Where repair costs have become excessive or good practice is known to have been compromised, however, an overlay may be a cost-effective means of extending service life.

7.2.2 *Skid resistance*—Bridge decks, like all roadway surfaces, should be adequately skid-resistant. Occasionally, rapid surface wear, due to construction deficiencies and use of inadequate skid-resistant aggregates, reduces skid resistance. Overlays provide a means for correcting this deficiency.

7.2.3 *Wearing course*—The use of studded tires has markedly increased the abrasive wear on some bridges. Consequently, overlays may be considered as a sacrificial wearing course because the loss through abrasion of an overlay would not reduce the section modulus or the critical clear cover over reinforcing bars in the structural slab. Overlays can be replaced with relative ease and cost.

7.2.4 *Reduction of wheel load effect*—Asphaltic concrete overlays are commonly used to provide wheel load distribution and a smooth riding surface that helps reduce impact. Because water and chemicals can penetrate asphalt overlays, a waterproofing membrane is placed on the deck surface before placing the asphalt.

7.3—Required properties of overlays

The required properties of overlays depend on their intended purpose, as discussed previously.

7.3.1 *Properties required of all overlays*—Several properties are generally required of all overlays, regardless of the reasons leading to their use.

7.3.1.1 Adhesion to concrete or bond is a fundamental requirement for most overlays. Without adhesion, overlays soon delaminate, which, at best, presents an unsightly appearance and, at worst, requires extensive repair to provide an acceptable wearing and protective surface.

7.3.1.2 Cohesion or resistance to shear within the overlay itself is necessary to resist the stresses induced by the turning and braking of the heaviest trucks. This resistance may be relevant when considering the use of unreinforced thermoplastic materials, such as asphalt.

7.3.1.3 Skid resistance is a fundamental requirement of an overlay, whether or not that is the purpose for which it was intended, because the overlay becomes the road surface. This property requires the use of abrasion-resistant aggregates in polymer concrete overlays. Grooving with a diamond-blade saw cut of hardened concrete or texturing of plastic concrete is usually required when placing hydraulic cement concrete overlays.

7.3.1.4 Durability, used herein as resistance to abrasion, deformation, and decay, is another important property. Many materials, such as bitumens, soften under high temperatures and become subject to rutting. Such rutting may be imperceptible in the roadway, but creates an undesirable bump at bridge joints. Polymer overlays may become brittle with age or when oxidized, and thus may not retain the properties for which they were intended. Extended service histories should be investigated for any proposed overlay.

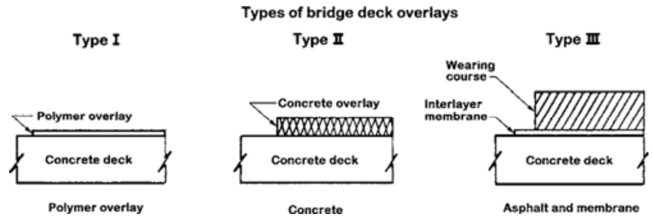

Fig. 7.4—Types of bridge deck overlays.

7.3.2 *Properties required of waterproof barriers*—In addition to the properties listed previously, waterproof barriers should be designed that are appropriate for the conditions that could lead to the intrusion of moisture and chloride ions (Manning 1995).

7.3.2.1 Impermeability is an important property of waterproof barriers. Materials may be impermeable in lab test conditions, but may be affected by ultraviolet radiation or by the heat from asphalt paving. Introducing aggregates for skid resistance or as bulk fillers may also create interconnected voids that admit water. Some construction techniques induce foaming and porosity that may increase water intrusion.

7.3.2.2 Crack resistance is another important requirement of a waterproof barrier. Development of cracks in concrete is one of the conditions leading to the use of a waterproof barrier. Hence, barrier materials should be capable of bridging such cracks in the underlying deck and remain waterproof. Reflective cracking in bridge decks is a much greater problem on long-span, cast-in-place decks. The most prevalent longitudinal cracking occurred as reflective cracks in thin concrete wearing courses over longitudinal joints of adjacent precast, prestressed box girder spans, or in areas where resistance to subsidence was offered by longitudinal reinforcement, void tubes, or other obstructions.

7.3.2.3 Bridge decks expand and contract with temperature change, and overlays placed on them should do likewise without loss of bond. Where thermal incompatibilities exist between the deck concrete and the overlay or membrane, shear stresses will be created by temperature change. These stresses are proportional to the overlay or membrane thickness. Such stresses may exceed the bond strength of the overlay or membrane or the shear strength of the deck concrete, resulting in the failure of the overlay or membrane's effectiveness. Thus, the coefficient of expansion of any overlay or membrane material is a significant property where substantial temperature changes occur.

7.4—Types of overlays

Overlays can be grouped into three categories (Fig. 7.4):

- Type I: Thin polymer overlays;
- Type II: Hydraulic cement concrete overlays; and
- Type III: Membrane and asphalt concrete overlays.

7.4.1 *Thin polymer overlays*—Thin polymer overlays have design thicknesses of 0.25 to 1 in. (6 to 25 mm), and therefore add minimal dead load to structures (AASHTO 1995). Their primary function may be to increase skid resistance or to act as surface membranes to minimize penetration of water and chloride ions. They should generally be applied to dry concrete surfaces. They usually involve durable, abrasion-resistant aggregates bonded together by various binders including asphaltic emulsions, polymer resins, and polymer-modified cements. Thin overlays are generally not recommended for badly spalled or deteriorated decks. Specialized expertise may be needed to properly apply these systems. A detailed discussion of these systems is beyond the scope of this document, but additional information may be found in ACI 548.1R-09, ACI 548.5R-94, ACI 548.8-07, ACI 548.9-08, and ACI 503R-93; and AASHTO Guide Specifications (AASHTO 1995). Polymer overlays can perform well for approximately 15 to 30 years (Sprinkel 2003a).

7.4.2 *Hydraulic cement concrete overlays*—This type of overlay typically varies from 1.25 to about 2.5 in. (30 to 65 mm) in thickness. Much thicker overlays have been constructed to improve drainage and ride quality. Much thicker overlays, however, should be avoided unless special consideration is given to the design of the deck (Section 7.4.3) and reinforcement of these thick overlays (Sprinkel 2003b). Latex-modified concrete and low-slump dense concrete overlays have been used on bridge decks since the 1960s (Sprinkel 1992). ACI 548.3R-09 and ACI 548.4-93 have useful information on latex-modified concrete overlays. The use of silica fume concrete overlays and rapid-strength-gain concrete began in the late 1980s (Sprinkel 1988, 1999). Many other types of overlays with or without steel and plastic fibers, with shrinkage-reducing admixtures, with corrosion-inhibiting admixtures, and with pozzolans and slag cement have been used successfully (Sprinkel 2001, 2005). The primary function of these systems is to replace deteriorated concrete or asphalt-wearing surfaces with an economical, durable, crack-resistant, low-permeability material without significantly increasing the dead load on the structure. The relative advantages and disadvantages of the systems may vary from one region to another, depending on local economic, climatic, and design factors. The choice of a system should involve consideration of the actual problems. Shrinkage and surface cracking of concrete overlays are likely to be significant factors in cold climates where deicing salts are used, as compared with milder climates with little use of deicing salts. Shrinkage cracking is also a significant factor in dry and windy climates. High-slump mixtures that are higher than 4 in. (100 mm) slump are not recommended for decks with longitudinal grades exceeding 2%. The use of steel fibers is generally intended to improve concrete toughness. Steel fibers are to meet ASTM A820/A820M-06 Type I or II specifications. Steel fibers may be continuously deformed or Z-hooked end deformed, a minimum of 1.5 in. (40 mm) in nominal length, and added at a dosage rate of 80 lb/yd^3 (47 kg/m^3) (Wisconsin DOT 2000). In general, deformed fibers significantly improve concrete toughness or energy absorption. Fibers with deformations at the ends appear to be more effective than those with deformations over the entire length (Banthia and Trottier 1995). Before use, the field experience of any particular system should be investigated. Steel fibers used in a white-topping project, which is a concrete overlay on asphalt pavement, failed to hold cracked sections together because of corrosion of the fibers in the cracks. Polyolefin fibers performed the best on the white-topping project (Sprinkel 2000). Hydraulic cement concrete overlays can perform well for 30 years (Sprinkel 2003a).

7.4.3 *Membrane and asphalt concrete overlays*—This type of overlay involves a waterproofing membrane covered with one or two courses of asphaltic concretes (Manning 1995). The total depths usually range from 2 to 4 in. (50 to 100 mm). Two courses of asphalt are recommended so that the top course can be replaced after approximately 15 years without damaging the membrane. The membrane will likely be damaged and have to be replaced when all of the asphalt is replaced. When available, the economics of asphalt may make this a good option for using the good riding quality and shock-absorbing qualities of the material (Sprinkel 2004). Membranes are not recommended for repairing badly delaminated decks with corroded reinforcing bars close to the surface.

There are many types of membranes, including hot-applied, rubberized membranes; sheet membranes; and liquid-applied, polymer membranes. The membranes should be capable of bonding to concrete, bridging cracks, waterproofing, and bonding to asphaltic concrete overlays without being affected by 300°F (150°C) asphalt. Some membranes require protection boards and two passes of asphaltic concrete to minimize damage during compaction; these systems may not be suitable for repair of existing bridges that were not designed for the extra dead load. Some sheet membranes may not bond well to concrete, or may debond at later dates if exposed to heat and sunlight. This creates vapor pressure and weakened bond due to temperature. Liquid-applied membranes may require special expertise. Some jurisdictions require warranties on membrane installation.

7.4.3.1 Wearing courses are generally asphaltic concretes. The design of such courses is beyond the scope of this guide.

An asphaltic concrete overlay should not be used directly on a portland cement concrete deck without a waterproofing membrane. All asphaltic concrete mixtures are inherently porous and readily conduct water and chlorides to the portland cement concrete deck, where they cannot be flushed off. Such impounded brine greatly accelerates bridge deck deterioration, which is then difficult to observe or measure below the asphalt. The permeability of asphaltic concrete greatly increases with age. Asphalt concrete overlays can perform well for 15 years (Sprinkel 2003a). When the overlay includes surface and intermediate mixtures, the surface mixture can be replaced without damaging the membrane, and the membrane may not have to be replaced for 30 years (Sprinkel 2003a).

7.5—Design considerations

For Type I and most Type II overlays, no special design considerations are usually necessary for the concrete bridge deck. On the other hand, for some Type II and all Type III overlays, the designer should carefully consider several details.

Thick concrete overlays, as well as membrane and asphaltic concrete systems, may increase the dead load on an existing deck. If so, structural design calculations should be reviewed, particularly on long-span structures.

In addition, a thick concrete or an asphalt overlay may require raising the bridge deck joints and surface drainage facilities to meet the new grade. The raised end joints, together with the effect of the bridge curbs, may create a void into which the overlay is placed. While water that permeates this wearing course should not affect a properly constructed interlayer membrane, it could, on freezing, disrupt the wearing course itself. For this reason, some designers prefer to install small-diameter subsurface drains to conduct the water that ponds below the asphalt through the deck slab. To prevent the leakage from causing deterioration of the deck underside, the drains should extend slightly below the deck or be surrounded by a drip groove. They should also be located so as to miss dripping on the supporting girders, or they may be extended to drip below the level of the girders.

7.6—Construction considerations

7.6.1 *Deck construction to accommodate overlays*—Where the use of overlays is anticipated, texturing of the plastic hydraulic cement concrete surface should be avoided. Sheet membranes generally bond better to smooth concrete, while thin overlays may bond better to the light roughness created by light brooming. A light grit-blast, shot-blast, or hydro-blast of the deck surface within 24 hours of placing the overlay is recommended to clean the surface, remove carbonated concrete, and to lightly texture the surface. A grout is often used to enhance the bond between the overlay and deck concrete. Applying a thin uniform layer of grout to a heavily textured surface is difficult, so grout should be avoided when the texture is heavy. Hydro-blasting an older deck is one example of this.

Manufacturer's recommendations should be consulted. For Type II and Type III overlays, deck surface tolerances for screeding and flatness need be less stringent than where Type I or no overlays are anticipated. Minor irregularities in profile and cross-slope can be corrected by the subsequent concrete or asphaltic concrete overlay.

Some curing compounds may inhibit the bond strength between Type I and Type II overlays and the deck surface. Where such materials are used, sand-blasting or shot-blasting should be required before applying the overlay.

On larger overlay projects in particular, it is recommended that the prepared surface be evaluated for bond strength using the procedure in ACI 503R-93. Alternately, test patches of overlay can be placed on the prepared surface and tested for bond strength using ACI 503R-93. The testing of the test patches can identify problems with the deck concrete, surface preparation, grout, and overlay materials and construction. Problems should be addressed so that a successful overlay is constructed (Sprinkel 2003b).

7.6.2 *Constructing the overlay*—Nearly all Type I and Type II overlays require scrupulous cleaning of the deck surface prior to application. Sand-blasting, shot-blasting, or hydro-blasting are generally preferred, although hydro-blasting is not recommended before applying most polymer materials. Manufacturer's recommendations should be checked. Shot-blasting involves less risk or human error than sand-blasting, and is often preferred. Surface preparation for Type III overlays is also dependent on the kind of membrane selected. Resinous membranes for Type III overlays may require the same degree of surface preparation as Type I and Type II overlays. Bitumen membranes may require only careful sweeping.

Type I overlays should be placed on a dry surface. The degree of surface dryness required for Type III overlays is dependent on the type of membrane material. Most polymers will not bond well to a moist surface. Asphalt will not bond well to a wet surface. In contrast, the emulsions often used with reinforced membrane systems may bond better to a moist surface than to a dry one. Manufacturer's instructions should be consulted.

Type II overlays generally bond best to surfaces that are saturated surface-dry. For low-slump dense concrete and latex-modified concrete, a bonding slurry is typically broomed on just ahead of the concrete placement. The effectiveness of bonding slurries has been questioned, however (Silfwerbrand and Paulsson 1998).

The ambient temperature is significant for nearly all overlays. Virtually all common materials require temperatures above freezing, and most above 50°F (10°C), to affect proper cure. One exception is the prefabricated sheets.

In the absence of specific information, a good rule of thumb is that all Type I overlays bond best to a clean, dry, (except emulsions), and warm deck.

7.6.2.1 Type I overlays may be applied by spraying or pouring the liquid binder. Aggregates are then broadcast over the surface. Another method is to premix the aggregates and binder, and screed the overlay, sometimes in narrow longitudinal strips. Sometimes the premix system is preceded by a primer coat. Aggregates are typically broadcast over the screeded surface.

7.6.2.2 Type II overlays are usually applied by screeding in place. Low-slump overlays require mobile concrete mixers and special screeds. Other overlays placed at 2 to 4 in. (50 to 100 mm) slump involve conventional screeds. High-amplitude air screeds or the use of air screeds with mixture slumps higher than 4 in. (100 mm) are not recommended due to their effect on the concrete air-void system and resulting freezing-and-thawing resistance of the overlay. Concrete overlays with HRWRAs should not be over-vibrated or over-finished to avoid durability problems.

7.6.2.3 Type III overlays are constructed according to the kind of membrane used. Membranes similar to Type I overlays are applied as in Section 7.6.2.1. Bitumen membranes are similarly applied except that mesh, usually of fiberglass, may be embedded rather than aggregate. Some types of prefabricated sheets are rolled in place after applying a suitable tack coat. Emulsion-based tack coats are preferred, because volatiles from asphalts may cause blistering in the sheets. Water vapor freed from emulsions may also cause blistering if adequate time is not permitted for the emulsion to cure properly. Some types of sheet membranes are applied by using torches to melt the bottom layer as the sheet is rolled into place.

Wearing courses are typically placed with conventional rubber-tired equipment and care so as not to damage the membrane. Many types of bitumen used in built-up, mesh-reinforced layers are vulnerable to damage and may require hand application of a binder course, followed by the surface wearing course.

7.7—Other considerations

Not all bridges have the same design and exposure conditions, so the resulting bridge deck problems are not always similar, and neither are the solutions. Several factors should be considered when choosing an overlay.

7.7.1 *Geographic and climatic factors*—Annual rainfall, maximum and minimum expected temperatures, annual ranges of humidity, and annual number of freezing-and-thawing cycles are all significant factors relating to expected service life that vary from region to region. Dry climates generally result in greater shrinkage and cracking of Type II overlays. Warm, wet climates are conducive to rapid rates of steel corrosion. Cold climates create tensile stresses from temperature change and cause many materials to become brittle and fail when subjected to live load stresses. Salt may be present in the aggregates of some regions, or may come from bodies of saltwater or from deicing chemicals used in northern regions. Abrasive surface wear may be greatly increased by the presence of studded tires or tire chains. Some regions are beginning to experience acid rain. Rates of carbonation also vary regionally. Both acid rain and carbonation lower the pH level of the concrete, which may result in increased reinforcing steel corrosion.

CHAPTER 8—REFERENCES

8.1—Referenced reports and standards

American Association of State Highway and Transportation Officials (AASHTO)

M31/M31M-10 Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement

M85-09 Standard Specification for Portland Cement

M240-10 Standard Specification for Blended Hydraulic Cement

M284/M284M-09 Standard Specification for Epoxy Coated Reinforcing Bars

M295-07 Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete

M302-06 Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars

M307-07 Standard Specification for Silica Fume Used in Cementitious Mixtures

MP 18M/MP 18-09 Standard Specification for Uncoated, Corrosion-Resistant, Deformed and Plain Alloy, Billet-Steel Bars for Concrete Reinforcement and Dowels

T11-05 Standard Method of Test for Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing

T19M/T 19-09 Standard Method of Test for Bulk Density (“Unit Weight”) and Voids in Aggregate

T21-05 Standard Method of Test for Organic Impurities in Fine Aggregates for Concrete

T26-79 Standard Method of Test for Quality of Water to Be Used in Concrete

T27-06 Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates

T71-08 Standard Method of Test for Effect of Organic Impurities in Fine Aggregate on Strength of Mortar

T84-10 Standard Method of Test for Specific Gravity and Absorption of Fine Aggregate

T85-10 Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate

T96-02 Standard Method of Test for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Lost Angeles Machine

T104-99 Standard Method of Test for Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate

T106M/T106-09 Standard Method of Test for Compressive Strength of Hydraulic Cement Mortar (Using 50-mm or 2-in. Cube Specimens)

T113-06 Standard Method of Test for Lightweight Pieces in Aggregate

T131-10 Standard Method of Test for Time of Setting of Hydraulic Cement by Vicat Needle

T161-08 Standard Method of Test for Resistance of Concrete to Rapid Freezing and Thawing

T303-00 Standard Method of Test for Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction

American Concrete Institute (ACI)

117-10 Specifications for Tolerances for Concrete Construction and Materials and Commentary

201.1R-08 Guide for Conducting a Visual Inspection of Concrete in Service

201.2R-08 Guide to Durable Concrete

211.1-91 Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete

211.2-98 Standard Practice for Selecting Proportions for Structural Lightweight Concrete

211.4R-08 Guide for Selecting Proportions for High-Strength Concrete Using Portland Cement and Other Cementitious Materials

212.3R-10 Report on Chemical Admixtures for Concrete
212.4R-93 Guide for the Use of High-Range Water-Reducing Admixtures (Superplasticizers) in Concrete
214R-11 Guide to Evaluation of Strength Test Results of Concrete
221R-96 Guide for Use of Normal Weight and Heavyweight Aggregates in Concrete
221.1R-98 Report on Alkali-Aggregate Reactivity
223R-10 Guide for the Use of Shrinkage-Compensating Concrete
232.1R-00 Use of Raw or Processed Natural Pozzolans in Concrete
232.2R-03 Use of Fly Ash in Concrete
233R-03 Slag Cement in Concrete and Mortar
234R-06 Guide for the Use of Silica Fume in Concrete
304R-00 Guide for Measuring, Mixing, Transporting and Placing Concrete
304.2R-96 Placing Concrete by Pumping Methods
304.5R-91 Batching, Mixing, and Job Control of Lightweight Concrete
305R-10 Guide to Hot Weather Concreting
306R-10 Guide to Cold Weather Concreting
308R-01 Guide to Curing Concrete
308.1-98 Standard Specification for Curing Concrete
309R-05 Guide for Consolidation of Concrete
325.6R-88 Texturing Concrete Pavements (withdrawn)
345.1R-06 Guide for Maintenance of Concrete Bridge Members
363R-10 Report on High-Strength Concrete
365.1R-00 Service Life Prediction
440R-07 Report on Fiber-Reinforced Polymer (FRP) Reinforcement for Concrete Structures
440.1R-06 Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars
440.3R-04 Guide Test Methods for Fiber-Reinforced Polymers (FRP) for Reinforcing or Strengthening Concrete Structures
440.5-08 Specification for Construction with Fiber-Reinforced Polymer Reinforcing Bars
440.6-08 Specification for Carbon and Glass Fiber-Reinforced Polymer Bar Materials for Concrete Reinforcement
503R-93 Use of Epoxy Compounds with Concrete
504R-90 Guide to Sealing Joints in Concrete Structures (withdrawn)
544.3R-08 Guide for Specifying, Proportioning, and Production of Fiber-Reinforced Concrete
548.1R-09 Guide for the Use of Polymers in Concrete
548.3R-09 Report on Polymer-Modified Concrete
548.4-93 Standard Specification for Latex-Modified Concrete (LMC) Overlays
548.5R-94 Guide for Polymer Concrete Overlays
548.8-07 Specification for Type EM (Epoxy Multi-Layer) Polymer Overlay for Bridge and Parking Garage Decks
548.9-08 Specification for Type ES (Epoxy Slurry) Polymer Overlay for Bridge and Parking Garage Decks

ASTM International

A276-10 Standard Specification for Stainless Steel Bars and Shapes
A706/A706M-09b Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A767/A767M-09 Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement
A615/C615M-09b Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
A775/A775M-07b Standard Specification for Epoxy-Coated Reinforcing Steel Bars
A820/A820M-06 Standard Specification for Steel Fibers for Fiber-Reinforced Concrete
A955/A955M-11 Standard Specification for Deformed and Plain Stainless-Steel Bars for Concrete Reinforcement
A1035/A1035M-09 Standard Specification for Deformed and Plain, Low-carbon, Chromium, Steel Bars for Concrete Reinforcement
C29/C29M-09 Standard Test Method for Bulk Density ("Unit Weight") and Voids in Aggregate
C33/C33M-11 Standard Specification for Concrete Aggregates
C40-04 Standard Test Method for Organic Impurities in Fine Aggregates for Concrete
C87/C87M-10 Standard Test Method for Effect of Organic Impurities in Fine Aggregate on Strength of Mortar
C88-05 Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate
C94/C94M-11 Standard Specification for Ready Mixed Concrete
C109/C109M-11 Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)
C117-04 Standard Test Method for Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing
C123-04 Standard Test Method for Lightweight Particles in Aggregate
C127-07 Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate

C128-07a	Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate
C131-06	Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
C136-06	Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates
C150/C150M-11	Standard Specification for Portland Cement
C171-07	Standard Specification for Sheet Materials for Curing Concrete
C191-08	Standard Test Methods for Time of Setting of Hydraulic Cement by Vicat Needle
C227-10	Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)
C260/C260M-10a	Standard Specification for Air-Entraining Admixtures for Concrete
C289-07	Standard Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method)
C295-08	Standard Guide for Petrographic Examination of Aggregates for Concrete
C309-07	Standard Specification for Liquid Membrane Forming Compounds for Curing Concrete
C311-11	Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use in Portland-Cement Concrete
C330/C330M-09	Standard Specification for Lightweight Aggregates for Structural Concrete
C457/C457M-10a	Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete
C494/C494M-10a	Standard Specification for Chemical Admixtures for Concrete
C539-84(2011)	Standard Test Method for Linear Thermal Expansion of Porcelain Enamel and Glaze Frits and Ceramic Whiteware Materials by Interferometric Method
C586-05	Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks as Concrete Aggregates (Rock-Cylinder Method)
C595/C595M-11	Standard Specification for Blended Hydraulic Cements
C618-08a	Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
C666/C666M-03 (2008)	Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing
C672/C672M-03	Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals
C845-04	Standard Specification for Expansive Hydraulic Cement
C989-10	Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars
C1137-97	Standard Test Method for Degradation of Fine Aggregate Due to Attrition (superseded) (historical standard)
C1157/C1157M-10	Standard Performance Specification of Hydraulic Cement
C1202-10	Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
C1240-10a	Standard Specification for Silica Fume Used in Cementitious Mixtures
C1260-07	Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)
C1293-08b	Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction
C1602/C1602M-06	Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete
C1603-10	Standard Test Method for Measurement of Solids in Water
D3398-00(2006)	Standard Test Method for Index of Aggregate Particle Shape and Texture
D7205/D7205M-06	Standard Test Methods for Tensile Properties of Fiber Reinforced Polymer Matrix Composite Bars
E274/E274M-11	Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire

8.2—Cited references

AASHTO, 1995, "Guide Specifications for Polymer Concrete Bridge Deck Overlays," American Association of State Highway and Transportation Officials, Washington, DC, 68 pp.

AASHTO, 1998, "LRFD Bridge Design Specifications," second edition, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO, 2000, "Guide Specification on Alkali-Silica Reaction," Section 56, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO, 2004, "LRFD Bridge Construction Specifications," second edition, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO, 2009, "LRFD Bridge Design Specification for GFRP Reinforced Concrete Decks and Traffic Railings—Revision 1," American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO, 2010, "LFRD Bridge Design Specifications (5th Edition) with 2010 Interims," American Association of

State Highway and Transportation Officials, Washington, DC, 1822 pp.

Antiohos S.; Giakoumelos, D.; and Tsimas, S., 2004, "Strength Development of Ternary Blended Cement with High-Calcium Fly Ash and Amorphous Silica," *Special Publication*, V. 221, pp. 153-168.

Attiogbe, E. K.; Nmai, C. K.; and Gay, F. T., 1992, "Air-Void System Parameters and Freeze-Thaw Durability of Concrete Containing Superplasticizers," *Concrete International*, V. 14, No. 7, July, pp. 57-61.

Banthia, N., and Trottier, J.-F., 1995, "Concrete Reinforced with Deformed Steel Fibers, Part II: Toughness Characterization," *ACI Structural Journal*, V. 92, No. 2, Mar.-Apr., pp. 146-154.

Bennett, J. E.; Bushman, J. B.; Clear, K. C.; Kamp, R. N.; and Swiat, W. J., 1993, *Cathodic Protection of Concrete Bridges: A Manual of Practice*, SHRP-S-372, National Research Council, Washington, DC, 148 pp.

Bergren, J. V., and Brown, B. C., 1975, "An Evaluation of Concrete Bridge Deck Surfacing in Iowa," Iowa Department of Transportation, Ames, IA, Apr., p. 11.

Bhatty, J. I., 2004, "Cement Manufacturing and Composition," *Research and Development Bulletin Serial* No. 2796, Portland Cement Association, Skokie, IL, 29 pp.

Bhatty, J. I.; Miller, F. M.(G.); and Kosmatka, S. H., eds., 2004, *Innovations in Portland Cement Manufacturing*, SP400, Portland Cement Association, Skokie, IL, 1387 pp.

Bickley, J. A. and Mitchell, J. D., 2001, "A State-of-the-Art Review of High-Performance Concrete Structures Built in Canada: 1990-2000," Cement Association of Canada, Ottawa, ON, Canada, May, p. 114.

Bradberry, T. E., 2001, "Concrete Bridge Decks Reinforced with Fiber-Reinforced Polymer Bars," *Transportation Research Record 1770*—Design of Structures 2001, Paper No. 01-3247, Transportation Research Board, Washington, DC, pp. 94-104.

Brown, M. C., and Weyers, R. E., 2003, "Corrosion Protection Service Life of Epoxy-Coated Reinforcing Steel in Virginia Bridge Decks," *Final Contract Report* VTRC 04-CR7, Virginia Transportation Research Council, Charlottesville, VA, Sept., 63 pp.

Buckler, J. G.; Barton, F. W.; Gomez, J. P.; Massarelli, P. J.; and McKeel, Jr., W. T., 2002, "Effect of Girder Spacing on Bridge Deck Response," VTRC 01-R6, Virginia Transportation Research Council, Charlottesville, VA, 32 pp.

Burg, R. G., and Ost, B. W., 1994, "Engineering Properties of Commercially Available High-Strength Concretes," *Research and Development Bulletin* RD104T, Portland Cement Association, Skokie, IL, 62 pp.

Burg, R. G., and Panoussaki, I., 1993, "Comparison of the Performance of Concrete Made with ASTM and AASHTO Type II Cements," *PCA R&D Serial* No. 1981, RP320.01T, Portland Cement Association, Skokie, IL.

Cady, P. D., and Weyers, R. E., 1983, "Chloride Penetration and the Deterioration of Concrete Bridge Decks," *Journal of Cement, Concrete, and Aggregates*, V. 5, No. 2, Winter, pp. 81-87.

Cady, P. D., and Weyers, R. E., 1984, "Deterioration Rates of Concrete Bridge Decks," *Journal of Transportation Engineering*, ASCE, V. 100, No. 1, Jan., pp. 34-44.

Cady, P. D., and Weyers, R. E., 1992, "Predicting Service Life of Concrete Bridge Decks Subject to Reinforcement Corrosion," ASTM STP 1137, Jan., pp. 20-31.

Canadian Standards Association (CSA), 2006, "Canadian Highway Bridge Design Code (CAN/CSA-S6)," 10th edition, Canadian Standard Association, Mississauga, ON, Canada, 800 pp.

Cheng, T. C., and Johnston, D. W., 1985, *Incidence Assessment of Transverse Cracking in Concrete Bridge Decks: Construction and Material Considerations*, FHWA/NC/85-002, Center for Transportation Engineering Studies, Dept. of Civil Engineering, North Carolina State University, Raleigh, NC, V. 1, June.

Clear, K. C., 1975, "Reinforcing Bar Corrosion in Concrete: Effect of Special Treatments," *Corrosion of Metals in Concrete*, SP-49, American Concrete Institute, Farmington Hills, MI, pp. 77-82.

Clear, K. C., 1976, "Time-to-Corrosion of Reinforcing Steel in Concrete Slabs," *Report* No. FHWA-RD-76-70, FHWA, Washington, DC, Apr., p. 38.

Clear, K. C., 1992, "Effectiveness of Epoxy-Coated Reinforcing Steel," *Concrete International*, V. 14, No. 5, May, pp. 58-62.

Clear, K. C., 1998, "Effectiveness of Epoxy-Coated Reinforcing Steel, Final Report," Canadian Strategic Highway Research Program, Ottawa, ON, Canada, Dec., p. 128.

Clemeña, G. G., 2002, "Testing of Selected Metallic Reinforcing Bars for Extending the Service Life of Future Concrete Bridges," Summary of Conclusion and Recommendations, VTRC 03-R7, Virginia Transportation Research Council, Charlottesville, VA, Dec., 12 pp.

Clemeña, G. G., 2003, "Investigation of the Resistance of Several New Metallic Reinforcing Bars to Chloride-Induced Corrosion in Concrete," VTRC 04-R7, Virginia Transportation Research Council, Charlottesville, VA, 24 pp.

Clemeña, G. G., and Virmani, Y. P., 2004, "Comparing the Chloride Resistances of Reinforcing Bars," *Concrete International*, V. 26, No. 11, Nov., pp. 39-49.

Clifton, J. R.; Beeghly, H. F.; and Mathey, R. G., 1974, "Nonmetallic Coatings for Concrete Reinforcing Bars," *Final Report* No. FHWA-RD-74-18, Federal Highway Administration (FHWA), Washington, DC, Feb., p. 87.

Cohen, M. D.; Zhou, Y.; and Dolch, W. L., 1992, "Non-Air-Entrained High-Strength Concrete—Is it Frost Resistant?" *ACI Materials Journal*, V. 89, No. 4, July-Aug, pp. 406-415.

Concrete Reinforcing Steel Institute (CRSI), 2009, *CRSI Manual of Standard Practice*, 28th edition, Schaumburg, IL, 144 pp.

Cope, B. L., and Ramey, G. E., 2001, "Reducing Drying Shrinkage of Bridge-Deck Concrete," *Concrete International*, V. 23, No. 8, Aug., pp. 76-82.

Cordon, W. A., and Thorpe, J. D., 1965, "Control of Rapid Drying of Fresh Concrete by Evaporation Control," ACI JOURNAL, *Proceedings* V. 62, No. 8, Aug., pp. 977-986.

Covino, Jr., B. S.; Cramer, S. D.; Holcomb, G. R.; Russell, J. H.; Bullard, S. J.; Dahlin, C.; and Tinnea, J. S., 2000, "Performance of Epoxy-Coated Steel Reinforcement in the Deck of the Perley Bridge," *Final Report*, Ministry of Transportation, ON, Canada, Mar., p. 102.

Dakhil, F. H.; Cady, P. D.; and Carrier, R. E., 1975, "Cracking of Fresh Concrete as Related to Reinforcement," ACI JOURNAL, *Proceedings* V. 72, No. 8, Aug., pp. 421-428.

Darwin, D., 2003, "Q&A – What Can Be Done to Reduce Cracking in a Concrete Bridge Deck?" *HPC Bridge Views*, No. 30, Nov.-Dec., 4 pp.

Davis, H., 1990, "Studies of the Performance of Fusion Bonded Epoxy Coated Reinforcement During Construction Process," *Protection of Concrete*, Proceedings of the International Conference, University of Dundee, Scotland, UK, R. K. Dhir and J. W. Green, eds., E&FN Spon, pp. 281-292.

Detwiler, R. J.; Bhatty, J. I.; and Bhattacharja, S., 1996, "Supplementary Cementing Materials for Use in Blended Cements," *Research and Development Bulletin* RD112, Portland Cement Association, Skokie, IL, 108 pp.

Fanous, F.; Wa, H.; and Pape, J., 2000, "Impact of Deck Cracking of Durability," Center for Transportation Research, Iowa State University, Ames, IA, 115 pp.

Fitch, M. G.; Weyers, R. E.; and Johnson, S. D., 1995, "Determination of End of Functional Service Life for Concrete Bridge Decks," TRB 1490, National Research Council, pp. 60-66.

French, J. W., and McKeel, W. T., 2003, "An Evaluation of Bridge Deck Joint Sealing Systems in Virginia," Virginia Transportation Research Council, Charlottesville, VA, June, 9 pp.

Gebler, S. H., and Klieger, P., 1986, "Effect of Fly Ash on Some of the Physical Properties of Concrete," *Research and Development Bulletin* No. RD089, Portland Cement Association, Skokie, IL, 48 pp.

Gillis, H. J., and Hagen, M. G., 1994, "Field Examination of Epoxy-Coated Rebars in Concrete Bridge Decks," *Report* No. MN/RD-94-14, Minnesota Department of Transportation, St. Paul, MN, 4 pp.

Gruner, P. W., and Plain, A. G., 1989, "Use of Type K Shrinkage Compensating Cement in Concrete for Bridge Decks," Paper 89-63, International Bridge Conference Proceedings, Pittsburgh, PA, pp. 331-334.

Gruner, P. W., and Plain, A. G., 1993, "Type K Shrinkage Compensating Cement in Bridge Deck Concrete," *Concrete International*, V. 15, No. 10, Oct., pp. 43-47.

Gulyas, R.; McCabe, K.; and Katlin, J., 2008, "High Performance Bridge Deck: The Use of ASTM C845 Type K Shrinkage Compensating Cement with Light Weight Aggregate Concrete for Optimized Internal Curing and Excellent Durability and Performance Benefits," Paper 71, Concrete Bridge Conference Proceedings, National Concrete Bridge Council, St. Louis, MO, May 4-7, 20 pp.

Hasan, H. O.; Ramirez, J. A.; and Cleary, D. B., 1995, "Field Evaluation of Concrete Bridge Decks Reinforced with Epoxy-Coated Steel in Indiana," *Paper* No. 950200, 74th Annual Meeting, Transportation Research Board, Washington, DC, 21 pp.

Hassan, T.; Abdelrahman, A.; Tadros, G.; and Rizkalla, S., 2000, "Fiber Reinforced Polymer Reinforcing Bars for Bridge Decks," *Canadian Journal of Civil Engineering*, V. 27, Oct., pp. 839-849.

Helmuth, R. A., 1987, *Fly Ash in Cement and Concrete*, SP040T, Portland Cement Association, Skokie, IL, 203 pp.

Henry, J. J., 2000, "Evaluation of Pavement Friction Characteristics," *NCHRP Synthesis* 291, Transportation Research Board, National Academy Press, Washington DC.

Hilsdorf, H. K., and Lott, J. L., 1970, "Revibration of Retarded Concrete for Continuous Bridge Decks," *NCHRP Report* 106, 67 pp.

Hogan, F. J., and Meusel, J. W., 1981, "Evaluation for Durability and Strength Development of a Ground Granulated Blast-Furnace Slag," *Cement, Concrete, and Aggregates*, V. 3, No. 1, Summer, pp. 40-52.

Hoke, J. H.; Pickering, H. W.; and Rosengarth, K., 1981, "Cracking of Reinforced Concrete," *ILZRO Project* ZE-271, International Lead Zinc Research Organization, Research Triangle Park, NC, 64 pp.

Holland, T. C., 1987, "Working with Silica-Fume Concrete," *Concrete Construction*, V. 32, No. 3, Mar., pp. 261-266.

Hossain, A. B.; Islam, S.; and Copeland, K. D., 2007, "Influence of Ultrafine Fly Ash on the Shrinkage and Cracking Tendency of Concrete and the Implications for Bridge Decks," Proceedings of the Transportation Research Board Annual Meeting, Paper #07-0022, Transportation Research Board, Washington, DC, Jan., 15 pp. (CD-ROM)

Johansen, V. C.; Taylor, P. C.; Peter C.; and Tennis, P. D., 2005, "Effect of Cement Characteristics on Concrete Properties," EB226, second edition, Portland Cement Association, Skokie, IL, 48 pp.

Keller, W. J., 2004, "Effect of Environmental Conditions and Structural Design on Linear Cracking in Virginia Bridge Decks," MS thesis, Civil and Environmental Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA, 192 pp.

Kirkpatrick, T. J.; Weyers, R. E.; Anderson-Cook, C. M.; and Sprinkel, M. M., 2002a, "Probabilistic Model for the Chloride-Induced Corrosion Service Life of Bridge Decks," *Cement and Concrete Research*, V. 32, No. 12, Dec., pp. 1943-1960.

Kirkpatrick, T. J.; Weyers, R. E.; Anderson-Cook, C. M.; and Sprinkel, M. M., 2002b, "Impact of Specification Changes on Chloride-Induced Corrosion Service Life of Bridge Decks," *Cement and Concrete Research*, V. 32, No. 8, Aug., pp. 1189-1198.

Kosmatka, S. H.; Kerkoff, B.; and Panarese, W. C, 2002, "Design and Control of Concrete Mixtures," *Engineering Bulletin* 001, 14th edition (Revised 2003), Portland Cement Association, Skokie, IL, 358 pp.

Kowalski, K. J.; McDaniel, R. S.; and Olek, J., 2010, "Identification of Laboratory Techniques to Optimize Superpave HMA Surface Friction Characteristics: Final Report," *IHRB Project TR-450*, School of Civil Engineering, Purdue University, IA, The Iowa Highway Research Board, 289 pp.

Krauss, P. D., and Rogalla, E. A., 1996, "Transverse Cracking in Newly Constructed Bridge Decks," *NCHRP*

Report 380, Transportation Research Board, Washington DC, 56 pp.

Lerch, W., 1957, "Plastic Shrinkage," ACI JOURNAL, *Proceedings* V. 53, No. 8, Apr., pp. 797-802.

Liu, Y., and Weyers, R. E., 1998, "Modeling the Time-to-Corrosion Cracking in Chloride-Contaminated Reinforced Concrete Structures," *ACI Materials Journal*, V. 95, No. 6, Nov.-Dec. pp. 675-681.

Maabn, E., and Sorensen, B., 1986, "The Influence of Microstructure on the Corrosion Properties of Hot-Dip Galvanized Reinforcement in Concrete," *Corrosion*, V. 41, No. 4, Apr., pp. 187-196.

McDonald, D.; Pfeiffer, D.; and Sherman, M., 1998, "Corrosion Evaluation of Epoxy-Coated, Metallic-Clad, and Solid Metallic Reinforcing Bars in Concrete," *Final Report* FHWA-RD-98-153, Federal Highway Administration, Washington, DC, Dec., p. 127.

Malhotra, V. M.; Ramachandran, V. S.; Feldman, R. F.; and Aïtcin, P. C., 1987, *Condensed Silica Fume in Concrete*, CRC Press, Inc., Boca Raton, FL, 221 pp.

Malisch, W. R.; Raecke, D. A.; Fischer, D. M.; Lott, J. L.; Kennedy, T. W.; and Kesler, C. E., 1966, "Physical Factors Influencing Resistance of Concrete to Deicing Agents," *NCHRP Report* 27, 41 pp.

Manning, D. G. 1995, "Waterproofing Membranes for Concrete Bridge Decks," *Synthesis of Highway Practice* 220, NCHRP, Washington DC.

McDonald, D. B.; Sherman, M. R.; Pfeifer, D. W.; and Virmani, P., 1995, "Stainless Steel Reinforcing as Corrosion Protection," *Concrete International*, V. 17, No. 5, May, pp. 65-70.

Mehdi, M., 2001, "Ternary and Quaternary Cements for Sustainable Development," *Concrete International*, V. 23, No. 4, Apr., pp. 35-42.

Menzel, C. A., 1954, "Causes and Prevention of Cracks Development in Plastic Concretes," *Proceedings,* Portland Cement Association Annual Meeting, Skokie, IL, pp. 130-136.

Ministry of Transportation Ontario, 1998, "Covermeter User's Guide for Concrete Cover Survey," Engineering Materials Office, Concrete Section, Downsview, ON, Mar., p. 17.

Mokarem, D. W.; Meyerson, R. M.; and Weyers, R. E., 2003, "Development of Concrete Shrinkage Performance Specifications," *VTRC Report 04-CR1*, Virginia Transportation Research Council, Charlottesville, VA, 40 pp.

Nassif, H., and Suksawang, N., 2002, "Effect of Curing Methods on Durability of High-Performance Concrete," *Transportation Research Record: Journal of the Transportation Research Board*, No. 1798, TRB, National Research Council, Washington, DC, pp. 31-38.

Neville, A. M., 2001, "Silica Fume in a Concrete Specification: Prescribed, Permitted, or Omitted?" *Concrete International*, V. 23, No. 3, Mar., pp. 73-77.

Ozyildirim, H. C., and Sprinkel, M. M., 2000, "Evaluation of Hydraulic Cement Concrete Overlays Placed on Three Pavements in Virginia," VTRC 01-R2, Virginia Center for Transportation Innovation and Research, Charlottesville, VA.

PCA Durability Subcommittee, 1998, "Guide Specification to Control Alkali-Silica Reactions," IS415, Portland Cement Association, Skokie, IL, 8 pp.

Pedeferri, P., 1998, "Behavior of Stainless Steel in the Rehabilitation of Corrosion Damaged Infrastructures and Effects of Galvanic Coupling Between Carbon Steel and Stainless Steel," *Rehabilitation of Corrosion Damaged Infrastructure*, NACE International, Houston, TX, pp. 1-7.

Pedeferri, P.; Bertolini, L.; Bolzoni, F.; and Pastore, T., 1997, "Behavior of Stainless Steels in Concrete," *Proceedings of the International Seminar: The State of the Art of the Repair and Rehabilitation of Reinforced Concrete Structures*, W. F. Silva-Araya, O.T. De Rincón, and L. P. O'Neill, eds., ASCE, Reston, VA, pp. 192-206.

Perenchio, W. F., and Klieger, P., 1978, "Some Physical Properties of High Strength Concrete," *Research and Development Bulletin* No. RD056.01T, Portland Cement Association, Skokie, IL, 7 pp.

Perfetti, G. R.; Johnston, D. W.; and Bingham, W. L.; 1985, "Incidence Assessment of Transverse Cracking in Concrete Bridge Decks: Structural Considerations," *Report* No. FHWA/NC/85-002, Center for Transportation Engineering Studies, Dept. of Civil Engineering, North Carolina State University, Raleigh, NC, V. 2, 201 pp.

Perregaux, G. R., and Brewster, D. R., 1992, "In-Service Performance of Epoxy-Coated Steel Reinforcement in Bridge Decks," *Technical Report* 92-3, New York DOT, Albany, NY, 11 pp.

Pfeifer, D. W.; Landgren, Jr., R. J.; and Zoob, A., 1987, "Protective Systems for New Prestressed and Substructure Concrete," FHWA/RD-86/193, Federal Highway Administration, Washington, DC, Apr., 113 pp.

Phillips, M.V.; Ramey, G. E.; and Pittman, D. W., 1997, "Bridge Deck Construction Using Type K Cement," *Journal of Bridge Engineering*, V. 2, No. 4, Nov., pp. 176-182.

Pinto, R. C. A, and Hover, K. C., 2001, "Frost and Scaling Resistance of High-Strength Concrete," *Research & Development Bulletin RD122*, Portland Cement Association, Skokie, IL, 70 pp.

Portland Cement Association, 1970, "Durability of Concrete Bridge Decks—A Cooperative Study," *Final Report* EBN067E, Skokie, IL, 36 pp.

Portland Cement Association, 2000, "Survey of Mineral Admixtures and Blended Cements in Ready Mixed Concrete," Portland Cement Association, Skokie, IL, 16 pp.

Praul, M. F., 2001, "Curing for HPC Bridge Decks—Bring on the Water!" *HPC Bridge Views*, No. 15, Hydrometeorological Prediction Center, Silver Spring, MD, May-June, 1 p.

Purvis, R., 2003, "Bridge Deck Joint Performance," *NCHRP Synthesis* 319, A Synthesis of Highway Practice, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC, 47 pp.

Pyc, W. A.; Weyers, R. E.; and Sprinkel, M. M., 1998, "Corrosion Protection Performance of Corrosion Inhibitors and Epoxy-Coated Reinforcing Steel in a Simulated

Concrete Pore Water Solution," *Report* No. VTRC98-R42, VTRC, Charlottesville, VA, June, p. 46.

Pyc, W. A.; Weyers, R. E.; Weyers, R. M.; Mokarem, D. W.; and Zemajtis, J., 2000, "Field Performance of Epoxy-Coated Reinforcing Steel in Virginia Bridge Decks," *Report* No. VTRC 00-R16, Virginia Transportation Research Council, Charlottesville, VA, p. 38.

Ramey, G. E., 2001, "Deck Thickness Considerations for CIP Reinforced Concrete Bridge Decks," *Practice Periodical on Structural Design and Construction*, V. 6, No. 1, Feb., pp. 35-47.

Ramey, G. E.; Pittman, D. W.; and Webster, G. K., 1999, "Shrinkage-Compensating Concrete for Bridge," *Concrete International*, V. 21, No. 4, Apr., pp. 29-34.

Ramniceanu, A.; Weyers, R. E.; and Mokarem, D. W., 2010, "Bridge Deck Concrete Volume Change," VTRC Report 10-CR5, Virginia Transportation Research Council, Charlottesville, VA, Feb., 31 pp.

Saucier, K. L.; Tynes, W. O.; and Smith, E. F., 1965, "High Compressive-Strength Concrete—Report 3, Summary Report," *Miscellaneous Paper* No. 6-520, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Sept., 87 pp.

Schiessl, P., 1988, "Corrosion of Steel in Concrete," RILEM Technical Committee 60-CSC Report, RILEM, Bagneux, France, 96 pp.

Scully, J. R.; Marks, C. A.; and Hurley, M. F., 2003, "Testing of Selected Metallic Reinforcing Bars for Extending the Service Life of Concrete Bridges: Testing in Solutions," *Report* No. FHWA/VTRC-03-CR11, Virginia Transportation Research Council, Charlottesville, VA, 2003, 48 pp.

Shiathas, C.; Muntasser, T. Z.; and Nwaubani, S. O., 2003, "A Comparative Study of the Properties and Durability of Binary and Ternary Cementitious Systems," *Sixth CANMET/ACI International Conference on Durability of Concrete*, SP-212, American Concrete Institute, Farmington Hills, MI, June, pp. 459-474.

Silfwerbrand, J., and Paulsson, J., 1998, "The Swedish Experience: Better Bonding of Bridge Deck Overlays," *Concrete International*, V. 20, No. 10, Oct., pp. 56-61.

Smith, L. L.; Kessler, R. J.; and Powers, R. G., 1993, "Corrosion of Epoxy-Coated Rebar in a Marine Environment," *Circular* No. 403, Transportation Research Board, Washington, DC, Mar., pp. 36-45.

Sprinkel, M. M., 1988, "High Early Strength Latex Modified Concrete Overlay," *Transportation Research Record* No. 1204, Transportation Research Board, Washington, DC, 42 pp.

Sprinkel, M. M., 1992, "Twenty Year Performance of Latex-Modified Concrete Overlays," *Transportation Research Record* No. 1335, Transportation Research Board, Washington, DC.

Sprinkel, M. M., 1999, "Very-Early-Strength Latex-Modified Concrete Overlay," *Transportation Research Record* No. 1668 – Concrete in Pavements and Structures, Transportation Research Board, Washington, DC, pp. 18-23.

Sprinkel, M. M., 2001, "Evaluation of High Performance Concrete Overlays Placed on Route 60 Over Lynnhaven Inlet in Virginia," *Proceedings*, PCI/FHWA Convention, Orlando, FL, Sept.

Sprinkel, M. M., 2003a, "Twenty-Five Year Experience with Polymer Concrete Bridge Deck Overlays," *Polymer in Concrete: The First Thirty Years*, SP-214, American Concrete Institute, Farmington Hills, MI, pp. 51-61.

Sprinkel, M. M., 2003b, "High Performance Concrete Overlays for Bridges," *Proceedings*, PCI/FHWA Convention, Orlando, FL, Oct. 19-22.

Sprinkel, M. M., 2004, "Deck Protection Systems for Post-Tensioned Segmental Concrete Bridges," *Proceedings*, American Segmental Bridge Institute (ASBI) 16th Annual Convention, Dallas, TX.

Sprinkel, M. M., 2005, "Latex-Modified Concrete Overlay Containing Type K Cement," Performing Organization Report No. VTRC 05-R26, Virginia Transportation Research Council, Charlottesville VA., Apr.

Stratfull, R. F.; Jurkovich, W. J.; and Spellman, D. L., 1975, "Corrosion Testing of Bridge Decks," *TRB Research Record* No. 539, Transportation Research Board, Washington, DC, Dec., pp. 500-559.

Swartz, B. D., and Schokker, A. J., 2008, "Design Guide Development for Post-Tensioned Bridge Decks," *PTI Journal*, Post-Tensioning Institute, Farmington Hills, MI, Aug.

Tennis, P., 1999, "Portland Cement Characteristics—1998," *Concrete Technology Today*, PL992, V. 20, No. 2, Portland Cement Association, Skokie, IL.

Tia, M.; Subramanian, R.; Brown, D.; and Broward, C., 2005, "Evaluation of Shrinkage Cracking Potential of Concrete Used in Bridge Decks in Florida," *Research Report* No. BC-354 RPWO# 26, Florida State Dept. of Transportation, Tallahassee, FL, Sept., 129 pp.

Tonini, D. E., and Dean, S. W., Jr., eds., 1976, "Chloride Corrosion of Steel in Concrete," ASTM STP 629, ASTM International, West Conshohocken, PA, July, 193 pp.

Trejo, D., 2002, "Evaluation of the Critical Chloride Threshold and Corrosion Rate for Different Steel Reinforcement Types," Department of Civil Engineering, Texas A&M University, College Station, TX, July, 68 pp.

U.S. Army Corps of Engineers (USACE), 1990, "Specifications for Membrane-Forming Compounds for Curing Concrete," *Handbook for Concrete and Cement*, CRD-C-300-90, Washington, DC, June.

Vassie, P. R., 1984, "Reinforcement Corrosion and Durability of Concrete Bridges," *Proceedings of the Institution of Civil Engineers*, Part 1, V. 76, Paper 8798, pp. 713-723.

Weed, R. M., 1974, "Recommended Depth of Cover for Bridge Deck Steel," *Transportation Research Record* No. 500, Transportation Research Board, Washington, DC, pp. 32-35.

West Virginia DOT, 1994, "Evaluation of Bridge Decks Using Epoxy Coated Reinforcement," *Research Report* MIR 1261603, Charleston, WV, 11 pp.

Weyers, R. E., 1995, "Protocol for In-Service Evaluation of Bridges with Epoxy-Coated Reinforcing Steel," *NCHRP Final Report* 10-37B, National Research Council, Washington, DC, Nov., p. 112.

Weyers, R. E., 1998, "Service Life Model for Concrete Structures in Chloride Laden Environments," *ACI Materials Journal*, V. 95, No. 4, July-Aug., pp. 445-453.

Weyers, R. E.; Cady, P. D.; and McClure, R. M., 1984, "Cost-Effective Decision Models for Maintenance, Rehabilitation, and Replacement of Bridges," *TRB Research Record* 950, National Research Council, Washington, DC, V. 1, No. 950, Sept. 24-26, pp. 28-33.

Weyers, R. E.; Conway, J. C.; and Cady, P. D., 1982, "Photoelastic Analysis of Rigid Inclusions in Fresh Concrete," *Journal of Cement and Concrete Research*, V. 12, No. 4, pp. 475-585.

Weyers, R. E.; Fitch, M. G.; Larsen, E. P.; and A1-Qadi, I. L.; Chamberlin, W. P.; and Hoffman, P. C., 1994, "Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques—Service Life Estimates," *Strategic Highway Research Program Report* SHRP-S-668, National Research Council, Washington, DC, 357 pp.

Weyers, R. E.; Prowell, B. D.; Sprinkel, M. M.; and Vorster, M., 1993, "Concrete Bridge Protection, Repair, and Rehabilitation Relative to Reinforcement Corrosion: A Methods Application Manual," SHRP-S-360, National Research Council, Washington, DC, 268 pp.

Weyers, R. E.; Pyc, W.; Sprinkel, M. M.; and Kirkpatrick, T. J., 2003, "Bridge Deck Cover Specifications," *Concrete International*, V. 25, No. 2, Feb., pp. 61-64.

Weyers, R. E.; Sprinkel, M. M.; Pyc, W.; Zemajtis, J.; Liu, Y.; and Mokarem, D., 1997, "Field Investigation of the Corrosion Protection Performance of Bridge Decks and Piles Constructed with Epoxy-Coated Reinforcing Steel in Virginia," *Report* No. VTRC 98-R4, Transportation Research Council, Charlottesville, VA, Oct., p. 38.

Weyers, R. E.; Sprinkel, M. M.; and Brown, M. C., 2006, "Summary Report of the Performance of Epoxy-Coated Reinforcing Steel in Virginia," VCTIR 06-R29, Virginia Center for Transportation Innovation and Research Council, Charlottesville, VA, Feb., p. 35.

Whiting, D. A., and Nagi, M. A., 1998, *Manual on Control of Air Content in Concrete*, EB116, Portland Cement Association, Skokie, IL.

Whiting, D. A.; Todres, A.; Nagi, M.; Yu, T.; Peshkin, D.; Darter, M.; Holm, J.; Andersen, M.; and Geiker, M., 1993, "Synthesis of Current and Projected Concrete Highway Technology," *Strategic Highway Research Program Report* No. SHRP-C-345, National Research Council, Washington, DC, 294 pp.

Whiting, D. M., and Schmitt, J., 1987, "Durability of In-place Concrete Containing High-Range Water-Reducing Admixtures," *NCHRP Report* No. 296, National Research Council, Washington, DC, Sept., 72 pp.

Whiting, N. M., and Snyder, M. B., 2003, "A Study in the Effectiveness of Portland Cement Concrete Curing Compounds," *Journal of the Transportation Research Board* 1834, pp. 59-68.

Williamson, G.; Weyers, R. E.; Brown, M. C.; and Sprinkel, M. M., 2007, "Bridge Deck Service Life Prediction and Costs," Final Contract Report, VTRC08-CR4, VTRC, Charlottesville, VA, Dec., 77 pp.

Wisconsin DOT, 2000, "Notice to all Contractors: Proposal #31: 1077-05-71; IM 2001 (088), Article 27, Concrete Masonry, Steel Fibers, Bridges, Item 90034A," Division of Transportation Infrastructure Development, Bureau of Highway Construction, Madison, WI, Nov.

Yeomans, S. R., 1994a, "A Conceptual Model for the Corrosion of Galvanized Steel Reinforcement in Concrete," *Corrosion and Corrosion Protection of Steel in Concrete*, R.N. Swamy, ed, Sheffield Academics Press, Proceedings of International Conference, University of Sheffield, July, pp. 1299-1309.

Yeomans, S. R., 1994b, "Performance of Black, Galvanized, and Epoxy-Coated Reinforcing Steel in Chloride-Contaminated Concrete," *Corrosion*, V. 50, No. 1, pp. 72-81.

Zemajtis, J.; Weyers, R. E.; and Sprinkel, M. M., 1998, "Corrosion Protection Service Life of Low-Permeable Concretes and Low-Permeable Concrete with a Corrosion Inhibitor," *TRB Research Record* Issue No. 1642, Transportation Research Board, Washington, DC, pp. 51-59.

Zemajtis, J.; Weyers, R. E.; and Sprinkel, M. M., 1999, "Performance Evaluation of Corrosion Inhibitors and Galvanized Steel in Concrete Exposure Specimens," *Report* No. VTRC 99-CR4, Virginia Transportation Research Council, Charlottesville, VA, Jan., p. 73.

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