

Guide for Concrete Highway Bridge Deck Construction

Reported by ACI Committee 345



American Concrete Institute®



American Concrete Institute®
Advancing concrete knowledge

First Printing
September 2011

Guide for Concrete Highway Bridge Deck Construction

Copyright by the American Concrete Institute, Farmington Hills, MI. All rights reserved. This material may not be reproduced or copied, in whole or part, in any printed, mechanical, electronic, film, or other distribution and storage media, without the written consent of ACI.

The technical committees responsible for ACI committee reports and standards strive to avoid ambiguities, omissions, and errors in these documents. In spite of these efforts, the users of ACI documents occasionally find information or requirements that may be subject to more than one interpretation or may be incomplete or incorrect. Users who have suggestions for the improvement of ACI documents are requested to contact ACI via the errata website at www.concrete.org/committees/errata.asp. Proper use of this document includes periodically checking for errata for the most up-to-date revisions.

ACI committee documents are intended for the use of individuals who are competent to evaluate the significance and limitations of its content and recommendations and who will accept responsibility for the application of the material it contains. Individuals who use this publication in any way assume all risk and accept total responsibility for the application and use of this information.

All information in this publication is provided “as is” without warranty of any kind, either express or implied, including but not limited to, the implied warranties of merchantability, fitness for a particular purpose or non-infringement.

ACI and its members disclaim liability for damages of any kind, including any special, indirect, incidental, or consequential damages, including without limitation, lost revenues or lost profits, which may result from the use of this publication.

It is the responsibility of the user of this document to establish health and safety practices appropriate to the specific circumstances involved with its use. ACI does not make any representations with regard to health and safety issues and the use of this document. The user must determine the applicability of all regulatory limitations before applying the document and must comply with all applicable laws and regulations, including but not limited to, United States Occupational Safety and Health Administration (OSHA) health and safety standards.

Order information: ACI documents are available in print, by download, on CD-ROM, through electronic subscription, or reprint and may be obtained by contacting ACI.

Most ACI standards and committee reports are gathered together in the annually revised *ACI Manual of Concrete Practice* (MCP).

American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
U.S.A.

Phone: 248-848-3700

Fax: 248-848-3701

www.concrete.org

ISBN 978-0-87031-410-0

 **Documents**
Document Sharing Hub

Guide for Concrete Highway Bridge Deck Construction

Reported by ACI Committee 345

Richard E. Weyers
Chair

Paul D. Carter
Secretary

Gerald H. Anderson
Michael C. Brown
Robert J. Gulyas*

Dena L. Guth
Alan B. Matejowsky
Harold R. Sandberg

Johan L. Silfwerbrand
Michael M. Sprinkel

Paul J. St. John
Jerzy Z. Zemajtis

*Deceased.

Consulting members

James C. Anderson
Byron T. Danley
Fouad H. Fouad
Allan C. Harwood

Martin E. Iorns
Yash Paul Virmani
Jeffrey P. Wouters

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.

ACI Committee Reports, Guides, Manuals, and Commentaries are intended for guidance in planning, designing, executing, and inspecting construction. This document is intended for the use of individuals who are competent to evaluate the significance and limitations of its content and recommendations and who will accept responsibility for the application of the material it contains. The American Concrete Institute disclaims any and all responsibility for the stated principles. The Institute shall not be liable for any loss or damage arising therefrom.

Reference to this document shall not be made in contract documents. If items found in this document are desired by the Architect/Engineer to be a part of the contract documents, they shall be restated in mandatory language for incorporation by the Architect/Engineer.

CONTENTS

Chapter 1—Introduction and scope, p. 2

- 1.1—Introduction
- 1.2—Scope

Chapter 2—Definitions, p. 2

- 2.1—Definitions

Chapter 3—Design and durability considerations, p. 2

- 3.1—General
- 3.2—Concrete and reinforcement materials
- 3.3—Positive protective systems
- 3.4—Arrangement and cover of reinforcement
- 3.5—Deck thickness
- 3.6—Deck drainage
- 3.7—Joint-forming materials and locations
- 3.8—Types and causes of deck cracking

ACI 345R-11 supersedes 345R-91 and was adopted and published September 2011.
Copyright © 2011, American Concrete Institute.

All rights reserved including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by electronic or mechanical device, printed, written, or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system of design, unless permission in writing is obtained from the copyright proprietor.

Chapter 4—Concrete materials, p. 10

- 4.1—General
- 4.2—Concrete-making materials
- 4.3—Chemical admixtures
- 4.4—Effects on concrete properties
- 4.5—Workability and consistency
- 4.6—Bleeding
- 4.7—Air content
- 4.8—Setting time
- 4.9—Shrinkage
- 4.10—Durability
- 4.11—Strength
- 4.12—Skid resistance

Chapter 5—Reinforcement, p. 17

- 5.1—General considerations
- 5.2—Reinforcement placement
- 5.3—Reinforcement supports and ties
- 5.4—Concrete cover over reinforcement
- 5.5—Cleanliness
- 5.6—Reinforcement type

Chapter 6—Placing, finishing, and curing, p. 22

- 6.1—Placing
- 6.2—Finishing
- 6.3—Curing

Chapter 7—Overlays, p. 31

- 7.1—Scope
- 7.2—Need for overlays
- 7.3—Required properties of overlays
- 7.4—Types of overlays
- 7.5—Design considerations
- 7.6—Construction considerations
- 7.7—Other considerations

Chapter 8—References, p. 35

- 8.1—Referenced standards and reports
- 8.2—Cited references

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 for rehabilitation, and milling of the concrete wearing surface.

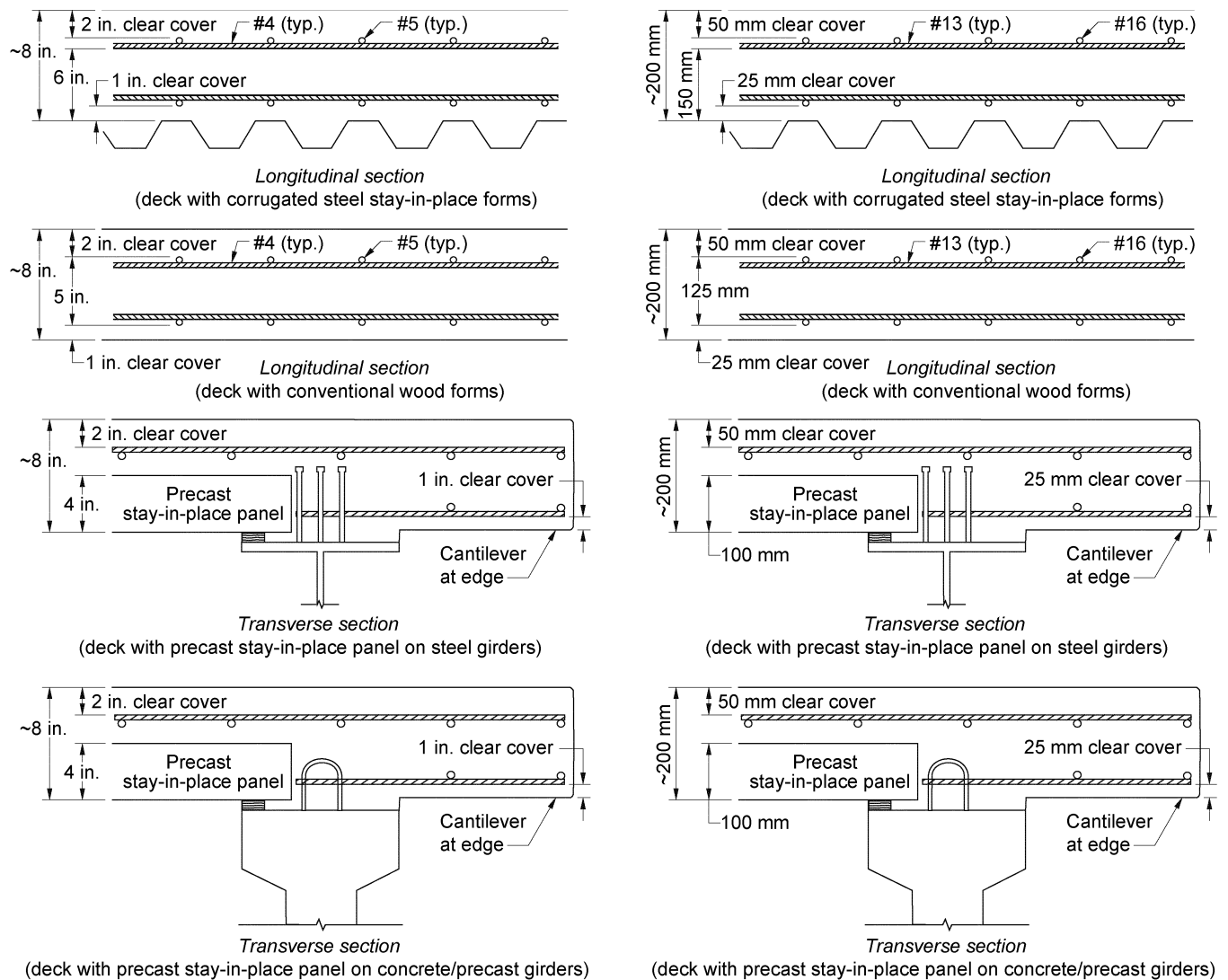


Fig. 3.5.1—Sections of several common reinforced concrete bridge configurations—U.S. Customary units (left) and SI units (right).

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.

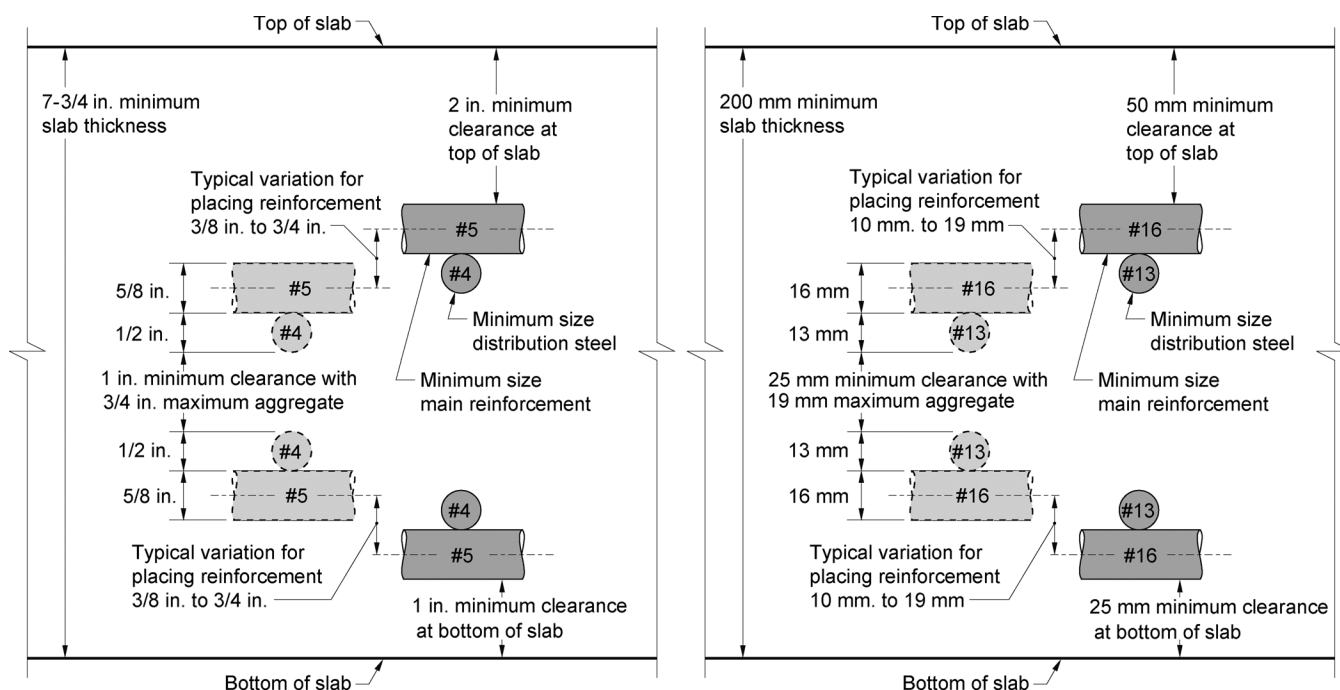


Fig. 3.5.3—Guidelines for vertical placement of reinforcement in bridge decks—U.S. Customary units (left) and SI units (right).

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



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.

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 bonding 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



Fig. 3.7.7—Example of modular joint system.

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.

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 308R-10. 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. (1 mm) wide;
2. Medium: Between 0.04 in. (1 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.

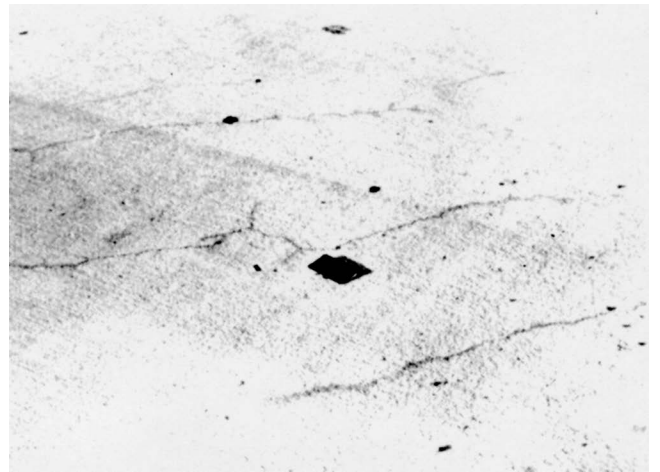


Fig. 3.8(b)—Diagonal cracking.

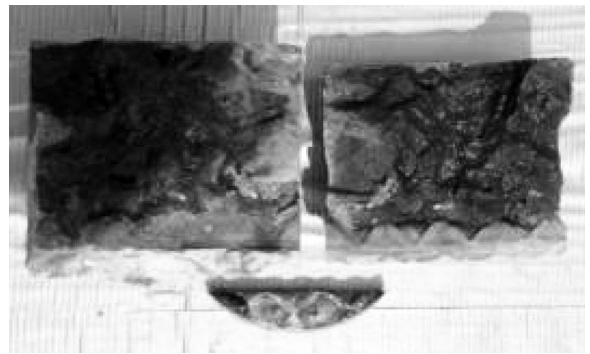


Fig. 3.8(c)—Faces of crack in core from a concrete deck.

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 ft² (3300 m²) 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

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 ₃ S	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

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.

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 ft²/lb (547 m²/kg), versus 1800 ft²/lb (369 m²/kg) for Type I (Tennis 1999). For comparison, the average Blaine fineness values of Types II and V were 1840 and 1820 ft²/lb (377 and 373 m²/kg), respectively. ASTM specifications allow for a higher SO₃ 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 (C₃S) 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 Panousaki 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 carbon fly ash. 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/C1157M-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/ft³ (1120 to 1500 kg/m³). 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 ft²/lb (20,000 m²/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/ft³ (130 to 430 kg/m³) (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 ft²/lb (400 to 600 m²/kg), specific gravity ranges from 2.85 to 2.95, and the bulk density varies from 66 to 86 lb/ft³ (1050 to 1375 kg/m³) (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-07). 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