

Report on Bridge Decks Free of Steel Reinforcement

Reported by ACI Innovation Task Group 3

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This document outlines procedures for the design of bridge decks free of steel reinforcement and requirements for design and installation of straps to restrain rotation of edge beams to achieve arching action in a deck slab.

Keywords: arching; bridge; composite action; corrosion; deck slab; fiber-reinforced concrete; reinforcement-free; transverse confinement; transverse constraint.

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PREFACE

The concept for the design of a steel-free bridge deck slab described in this report is patented. Therefore, use of the information in this document may require payment of royalties to the owners of the patents. At the time of printing, the United States and the United Kingdom have granted a patent for the steel-free cast-in-place bridge deck slabs, with a patent pending in Canada. The steel-free precast slab is also patented in the United States, and the global patent is pending. Interested parties are invited to submit information regarding the identification of an alternative(s) to this patented item to ACI Headquarters. Your comments will receive careful consideration at a meeting of the responsible standards committee, which you may attend.

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The inventor of the reinforcement-free bridge deck concept described in this report sponsored preparation of the report and provided reimbursement to the authors to assist in recovery of their costs and expenses related to travel to meetings; however, none of the authors received an honorarium.

ACI ITG-3-04 became effective September 10, 2004.
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Conclusions from research conducted since the document was first written are included in [Appendixes B and C](#).

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CHAPTER 1—INTRODUCTION

1.1—Purpose

The purpose of this document is to introduce a new technology to address corrosion-free concrete bridge deck slabs by eliminating the steel reinforcement. The concept for the design of a steel reinforcement-free bridge deck described in this document is patented. Therefore, authorization must be obtained from the owners of the patents before the information contained in this document can be used.*

Development of the steel-free bridge deck slab concept described in this report was prompted by the need for more durable bridge deck slabs. Traditionally, attempts have been made to alleviate the problem of corroding steel reinforcement by increasing the concrete cover, applying waterproofing membranes, or using epoxy-coated reinforcing steel, among other methods. While the use of these materials and methods has contributed to greater durability, the potential for deterioration due to corrosion of the reinforcing steel still exists. Eliminating the steel reinforcement in the deck slab can solve the corrosion problem.

This concept originated from recognizing that the conventional method for the design of bridge deck slabs is based on the flexural strength of the slab. The slab is assumed to be a transverse bending member supported by girders. This method leads to the need for a large amount of steel reinforcement in the deck slabs.

1.2—Scope and objectives

The scope of material presented in this document relates principally to delineating methodology for the design of reinforcement-free bridge decks. Other discussion includes guidance on special considerations for the construction and maintenance of this new type of bridge design. [Appendix D](#) provides research needs that require further investigation to facilitate continued understanding and improvement of this new emerging technology.

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Publication of this document satisfies the objective of disseminating as early as possible new emerging technology to the concrete industry. This objective is satisfied through the sharing of this new knowledge and experience in implementing the new reinforcement-free bridge deck designs. This sharing of new information and experiences helps accelerate technology transfer from the research arena to practice. The discussion that follows in this section serves to provide background on the origin of the idea for the development of the reinforcement-free bridge deck.

Extensive field and laboratory testing, done mainly in Ontario, Canada, more than two decades ago, showed that the amount of reinforcement in deck slabs could be reduced substantially by taking into account the arching action that develops within the slab. The 1979 Ontario Highway Bridge Design Code (OHBDC) included an empirical design method that took the arching action into account. The AASHTO LRFD Bridge Design Specifications (1998) also include a similar method for the design of deck slabs (Reference Clause 9.7.2). Using large-scale laboratory testing, further research into the arching action of deck slabs was initiated at Dalhousie University in Nova Scotia, Canada. Researchers found that suitable harnessing of the arching action could lead to the removal of the entire tensile reinforcement. The key to removing the tensile reinforcement from the deck slab is the provision of adequate transverse restraint, which generates compressive membrane forces so that under a concentrated load, the slab fails in a localized punching shear mode rather than in flexure. The terms “transverse constraint” and “transverse confinement” have been used interchangeably in this document.

Over the past 12 years, a large number of large-scale models of steel reinforcement-free deck slabs have been tested, mainly in Nova Scotia and Ontario. This research led to the construction, in December 1995, of the world’s first steel-free cast-in-place deck slab over the Salmon River, which forms part of the Trans-Canada Highway in Nova Scotia. Since then, four other Canadian bridges now incorporate concrete bridge deck slabs free of steel reinforcement. These are the Chatham Bridge in Chatham, Ontario; the Crowchild Trail Bridge in Calgary, Alberta; the Waterloo Creek Bridge in British Columbia; and the Lindquist Bridge in British Columbia, which is on a forestry road and incorporates a precast slab.

A reinforcement-free precast slab has also been used in the reconstruction of a wharf structure in Hall’s Harbour, Nova Scotia. The research conducted on reinforcement-free deck slabs in Canada over the past 12 years is reported in the technical literature (Section 10.1) along with the histories of bridges incorporating the new concept. Chapter 10 is a listing of the published papers referenced in this document.

In addition to the work done in Canada, research on the reinforcement-free deck slab is also being conducted in the United States (Seible et al. 1998) and Japan (Matsui et al. 2001).

1.3—Further research needs

While the reinforcement-free bridge deck design leads to elimination of the problem of corrosion from steel reinforcement,

it also raises several other concerns. Those concerns relate to the possibility that significant longitudinal cracking will be observed over time, and other types of cracks may lead to serviceability problems. Another area of concern is the potential for deterioration of the steel straps due to corrosion and the lack of their protection from a possible fire or impact. To address this issues, Appendices B, C, and D provide supplemental information and list areas of additional research that should be considered to answer the aforementioned concerns.

CHAPTER 2—DEFINITIONS AND ABBREVIATIONS

AFRP—aramid fiber-reinforced polymer.

CFRP—carbon fiber-reinforced polymer.

fibers—small-diameter filaments of materials of relatively high strength, which can be glass, carbon, aramid, or low-modulus polymer.

FRC—fiber-reinforced concrete; a fiber-reinforced composite in which the matrix is portland-cement concrete, and in which the fibers are discontinuous and uniformly and randomly distributed.

FRP—fiber-reinforced polymer; a fiber-reinforced composite with a polymeric matrix and continuous fiber reinforcement.

fiber volume fraction—the ratio of the volume of the fibers to the volume of the fiber-reinforced composite.

GFRP—glass fiber-reinforced polymer.

low-modulus fibers—fibers of thermoplastic polymer with a moduli of elasticity less than 1450 ksi (10 GPa), such as nylon, polyolefin, polypropylene, and vinylon.

matrix—the continuous material in a fiber-reinforced concrete or polymer component that contains aligned or randomly distributed fibers.

multispine bridge—a box-girder bridge in which the bottom flange is discontinuous in the transverse direction.

RC—reinforced concrete.

reinforcement—in this document, this term refers to bars that are added to concrete to enhance its tensile strength. Bars provided for only crack control are not referred to as reinforcement.

strap—a linear component of steel, FRP, or other material used to provide external transverse confinement in the reinforcement-free bridge deck slabs.

CHAPTER 3—DESIGN METHODOLOGY

The design methodology employing the concept described in this report relies mainly on transverse straps (Fig. 3.1 through 3.3), which may be made of steel, connected to the top flanges of adjacent girders for preventing their outward relative displacement; such displacement would normally occur when a load is applied on the slab between two girders. A combination of flange restraint against lateral movement and the cracking of concrete at the bottom of the slab leads to the formation of a shallow arch in the slab with the straps acting as ties. The degree of lateral restraint provided by the straps controls the relative lateral movement of adjacent girders and governs the ultimate load at which the slab fails in punching. The failure load can be several times greater

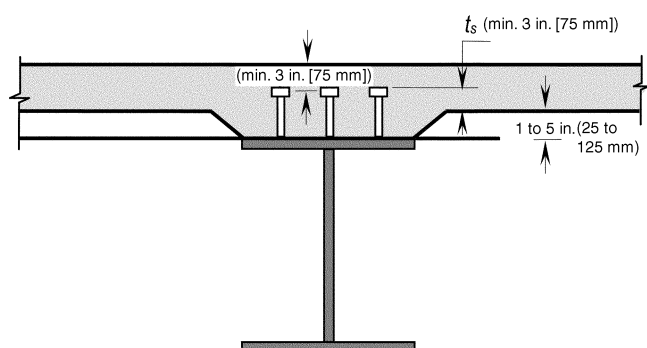


Fig. 3.1—Haunch between the deck slab and top of the supporting beam.

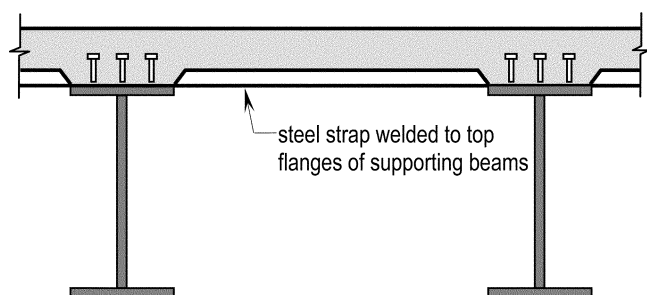


Fig. 3.2—Transverse confinement by directly connected straps.

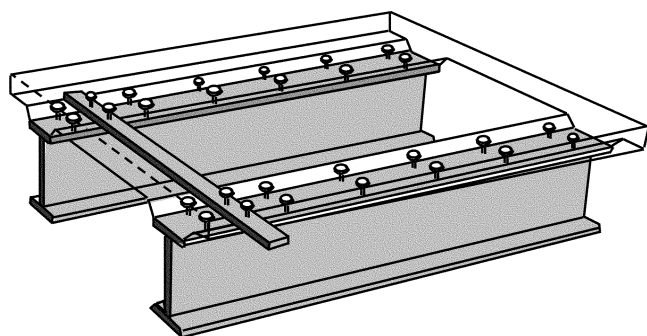


Fig. 3.3—Illustration of transverse confinement by partially studded straps.

than the load that causes flexural cracking of the slab, and is markedly greater than the failure load of an RC slab having the same dimensions. The design guidelines that follow are for cast-in-place and precast bridge deck slabs in accordance with the load and resistance factor design (LRFD) format of the AASHTO Specifications (1998). This method for designing bridge deck slabs can be modified suitably to address slabs of other structures, such as parking garages and marine wharves.

A cast-in-place or precast slab supported on beams satisfying the conditions set forth as follows need not be analyzed except for negative transverse moments due to loads on the overhangs and concrete parapets.

3.1—Composite action

The deck slab should be composite with parallel supporting beams in the positive moment regions of the beams.

The composite action of the deck slab with the supporting beams provides the necessary confinement in the longitudinal direction of the beams (Bakht, Mufti, and Jaeger 1998). The condition for the composite action in the positive moment regions should be considered in conjunction with [Section 3.10](#), which deals with the composite action in the negative moment regions. For precast deck slabs, the longitudinal confinement can be provided within the panel itself, eliminating the need for the composite action (Mufti, Banthia, and Bakht 2001).

A deck slab supported on multispine box girders without external bracing experiences significant transverse moments under eccentric live loads. Such transverse moments cannot be dealt with by the arching action in the slab. Accordingly, neither cast-in-place nor precast deck slabs without reinforcement should be used on multispine box girders unless the box girders are prevented from excessive relative rotation by means of suitable bracing between them.

3.2—Beam spacing

The spacing of the supporting beams, S , should not exceed 12 ft (3.7 m).

The largest spacing of the supporting beams of RC deck slabs designed by the empirical methods of AASHTO is 13.5 ft (4.11 m). Cast-in-place and precast deck slabs have been tested in the laboratory with a beam spacing of up to 13.1 ft (3.99 m). To be conservative, however, the maximum spacing of bridge girders is limited to 12 ft (3.7 m). The spacing can be increased if approval is obtained from the authority having jurisdiction over the bridge. The approval process can be facilitated by conducting tests on full-scale models. The spacing of the girders for the first reinforcement steel-free deck slab was 8.9 ft (2.71 m). Before this slab was built, its full-scale model was tested to failure (Newhook and Mufti 1996a). Subsequently, precast deck slabs with beam spacing of 11.5 ft (3.51 m) were tested at full-scale in the laboratory and implemented in the field (Sargent, Mufti, and Bakht 1999). As noted in [Chapter 7](#), the precast slab of a marine structure is supported on beams at a spacing of 13.1 ft (3.99 m).

The use of beam spacing greater than 12 ft (3.7 m) should be based on tests of full-scale models and not on the basis of analysis alone.

3.3—Slab thickness

The deck slab thickness t should be at least 6.5 in. (165 mm), and should not be less than $S/15$, where S is the spacing of the supporting beams, in inches (mm).

To predict the failure load of deck slabs under concentrated loads, an analytical model was developed by Mufti and Newhook (1998b). The limiting ratio of beam spacing to slab thickness required in this condition was partially established by using this method, which has been validated with experimental results. Also according to this method and verified by experiment, even a 6 in. (150 mm) thick slab provides sufficient strength to the deck slab supported on beams spaced at 6.6 ft (2.01 m) (Bakht and Lam 2000). To be conservative, however, a minimum slab thickness of 6.5 in. (165 mm) was used. Structures that are not subjected to heavy vehicular

loads, such as parking garages, can have thinner slabs, but only after confirmatory tests on full-scale models have been conducted.

3.4—Diaphragms

Unless justified by rigorous analysis, the supporting beams should be connected with transverse diaphragms, or cross-frames, at a spacing of not more than 26 ft (7.9 m).

The transfer of wheel loads to beams remote from the live loads takes place primarily through the transverse flexural action of the deck slab. For most bridges, the transverse moments induced in the deck slab by this mechanism are positive, with tension occurring at the bottom of the slab. For such cases, the transverse moments are sustained by the straps at the underside of the precast and cast-in-place deck slabs. In certain cases, an eccentrically placed vehicle may induce small negative transverse moments in the deck slab, resulting in tension in the top surface of the slab that might lead to longitudinal cracks at its top surface. To accommodate such cases, this condition requires the provision of transverse diaphragms. This requirement is very conservative and can be superseded after additional analytical investigation. Another reason for providing the diaphragms, or cross-frames, is to provide additional redundancy in the bridge against failure of one of the girders. The condition for diaphragms does not apply to structures that do not carry heavy vehicular loads, such as parking garages and some marine structures.

3.5—Haunches

As illustrated in Fig. 3.1, the height of the haunch between the deck slab and the top of a supporting beam should be between 1 and 5 in. (25 and 125 mm), and the projection of the shear connecting devices in the deck slab, t_s , should be at least 3 in. (75 mm). In addition, the cover between the top of the shear connecting devices and the top surface of the deck slab should be a minimum of 3 in. (75 mm) when the slab is not exposed to moisture-containing chlorides. Otherwise, either this cover distance should be a minimum of 4 in. (100 mm), or the shear connectors should be provided with an approved coating.

The upper limit of the height of the haunch of a cast-in-place or precast slab ensures that the haunch is not sheared off under transverse loads generated by the arching action (Bakht and Lam 2000). The limit for the minimum projection of shear studs in the slab provides for an effective transfer of the interface shear from the supporting beams to the slab.

With the help of comparisons between experimental and analytical results, the effective height of the deck slab arch is the distance between its crown and the center of gravity of its transverse confining system, being the bottom transverse bars in RC slabs and straps in the cast-in-place and precast slabs (Bakht, Mufti, and Jaeger 1998). The reference Mufti and Newhook (1998b) contains a table of comparison between analytical and experimental failure loads. The effective thickness of the slab is greatly influenced by the height of the haunch.

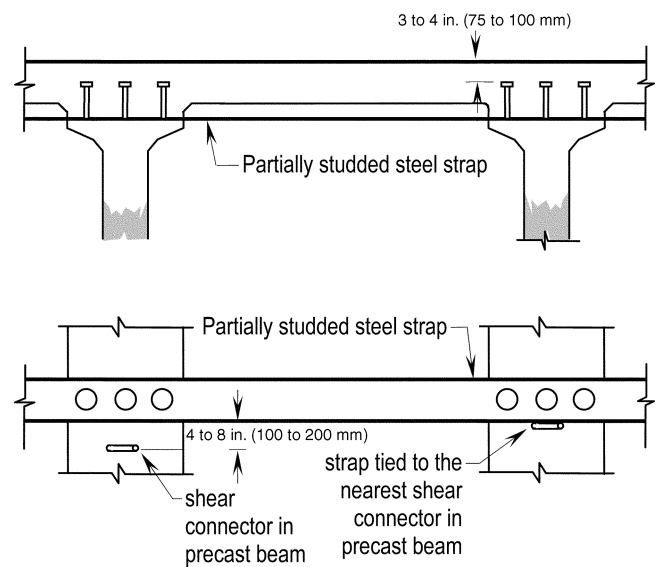


Fig. 3.4—Details of transverse confinement by partially studded straps.



Fig. 3.5—Photograph showing straps embedded in an Arch-Panel.

3.6—Transverse confinement

The top flanges of all adjacent supporting beams should be restrained by an external transverse confining system comprising straps or bars that are perpendicular to the supporting beams and that are:

1. Connected directly to the top of the flanges, as in welded steel straps (Fig. 3.2);
2. Connected indirectly, as in partially studded straps (Fig. 3.3 and 3.4);
3. Partially embedded in precast slabs (Fig. 3.5); or
4. Proven by approved full-scale laboratory testing.

The arching action in a reinforcement-free cast-in-place and precast deck slab is harnessed by transverse confinement, which occurs by restraining the relative lateral movement of the supports of the slab (the top flanges of the beams). Bakht and Lam (2000) and Bakht and Aly (1997) have shown that the transverse confinement can be provided by a number of methods, including the use of welded steel straps. In a precast slab of a forestry bridge, the transverse confinement was provided by steel straps partially embedded in the precast concrete deck slab (Fig. 3.5); these straps were provided with two or three shear studs at each of their embedded ends.

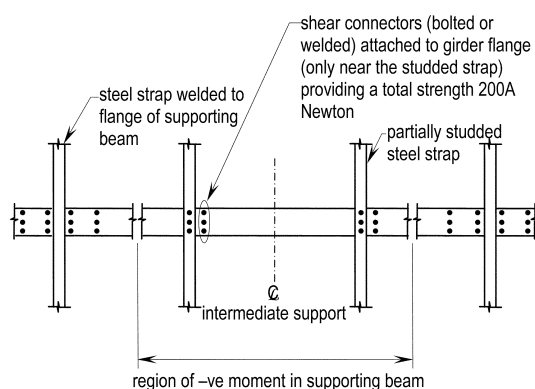


Fig. 3.6—Transverse confinement in longitudinal negative moment regions.

Guidance on testing the full-scale laboratory model of the deck slab can be found in Bakht and Lam (2000), Khanna, Mufti, and Bakht (2000), Newhook and Mufti (1996a), and Thorburn and Mufti (1995). The length of the model, measured parallel to the beams, should be at least twice the girder spacing, and the concentrated test load should be well away from the transverse free edges of the slab, especially if they are not stiffened.

3.7—Strap spacing

The spacing of straps, S_l , should not be more than 4 ft (1.2 m).

The requirement for the maximum spacing of the straps ensures that the lateral confinement is reasonably uniform along the length of the beams and also provides for the loss of a few straps. In the event a vehicle hits the underside of the bridge and more than two straps are lost, they should be replaced before permitting traffic above the damaged straps.

3.8—Strap size

Each strap should have a minimum cross-sectional area A in.², given by

$$A = \frac{F_s S^2 S_l}{Et} \quad (3-1)$$

where the factor F_s is 0.87 ksi for outer panels and 0.73 ksi for inner panels; the girder spacing S , the strap spacing S_l , and the slab thickness t are in inches; and E is the modulus of elasticity of the material of the strap in psi.

In metric units, the cross-sectional area A in mm² is given by the following equation

$$A = \frac{F_s S^2 S_l}{Et} \quad (\text{in SI})$$

where F_s is 6.0 MPa for outer panels and 5.0 MPa for inner panels; S and S_l are in mm; t is in mm; and E is in MPa.

The requirement for the minimum cross-sectional area for the straps was developed through full-scale laboratory experiments and confirmed by analytical studies (Thorburn and Mufti 1995). For the analytical study, the method of Mufti and Newhook (1998b) was used. As suggested by E

being in the denominator of Eq. (3-1), the cross-sectional area of the strap is governed by its axial stiffness and not by its strength. The value of F_s is lower for inner panels because of the substantial additional restraint provided to them by the slab in the adjacent panels, and because of bending between adjacent diaphragms in their own plane.

When the first frequency of vibration of a strap matches that of the bridge, the straps will vibrate excessively with the passage of every vehicle on the bridge. The steel straps in the Salmon River Bridge have a cross section of 4 x 1/2 in. (100 x 13 mm), with the latter dimension being the thickness. These straps vibrate visibly. Because of very low levels of stresses induced in the straps, there is no concern for the fatigue of their connections. The problem of strap vibration was eliminated in subsequent cast-in-place and precast slabs by increasing the thickness of the straps and reducing their widths.

3.9—Strap connection

The direct or indirect connection of a strap to the supporting beams should be designed to have a factored shear strength in kips of at least 29A, where A is in in.² In metric units, the factored shear strength of the connection in Newtons is required to be at least 200A, where A is in mm.²

The tensile force induced in a strap, even under specified failure loads, is significantly smaller than its tensile strength; therefore, the strength of its connection to the top flanges of the beams need not be as high as its tensile strength. As noted previously, the straps are required for their axial stiffness and not for their strength. Because the stresses induced in the straps are small under service loads, their fatigue resistance is not of concern. The factored connection strength is obtained by multiplying the nominal strength with the appropriate resistance factors. The effectiveness of an indirect strap connection to girders can be obtained experimentally by tests described by Bakht and Aly (1997).

3.10—Strap connection in negative moment regions

As shown in Fig. 3.6, in a negative moment region of a supporting beam, where the beam is not made composite with the slab, shear connecting devices should be provided on the beam in the vicinity of the straps, with a minimum total factored shear strength in kips (N) of 29 A (200 A), with A being in in.² (mm²). As shown in Fig. 3.4, such shear connecting devices should be within 4 in. (100 mm) of the nearest strap.

In continuous-span steel girder bridges, mechanical shear connectors are not always provided in the negative moment regions because of concerns for the fatigue resistance of the top flange in flexural tension. Even in such cases, this condition requires that some connectors be provided so that the tensile force in the straps is transferred effectively to the top flanges of the beams. Figure 3.6 shows that such connectors may be bolted to the top flanges to ensure that the fatigue category of the detail does not reduce the permissible fatigue stresses significantly.

3.11—Edge stiffening

Composite edge beams should have a suitably high flexural rigidity EI in the plane of the deck slab to stiffen the transverse

edges of the deck slab. Recommended edge stiffening details are given in [Section 5.1](#).

The arching action in the deck slab is harnessed in the longitudinal direction by making the slab composite with the beams and by providing edge beams with sufficiently high flexural stiffness in the plane of the slab (Bakht and Agarwal 1995).

3.12—Reinforcement for transverse negative moment

The deck slab should incorporate appropriate tensile reinforcement for transverse negative moments resulting from gravity loads on deck slab overhangs and horizontal loads on railings or concrete parapets.

Negative transverse moments induced by loads on the railings and those on the cantilever overhang do not generate arching action in the deck slab. When such negative moments are present, the deck slab should be suitably reinforced by transverse tensile reinforcement, which can be of coated steel, stainless steel, AFRP, CFRP, or GFRP.

The transverse negative moment reinforcement in the cantilever overhang should be extended into the adjacent panel as well. When the deck slab is subjected to uplift forces, such as wave action under a marine structure, reinforcement should be provided in the slab to resist such forces.

3.13—Reinforcement in longitudinal negative moment

For continuous-span bridges, the deck slab should contain longitudinal negative moment reinforcement in at least those segments in which the flexural tensile stresses in concrete under service loads are larger than $0.6f_{cr}$, where f_{cr} in psi is calculated in accordance with AASHTO LRFD Specifications (1998), as follows for f'_c , also in psi

$$f_{cr} = 6.32(f'_c)^{0.5} \quad (3-2)$$

In metric units, f_{cr} in MPa is calculated as follows for f'_c , also in MPa

$$f_{cr} = 0.5(f'_c)^{0.5} \quad (\text{in SI})$$

The requirement for longitudinal negative moment reinforcement ensures that transverse cracks developing at the top of the slab are controlled.

If AFRP or GFRP is used as the longitudinal reinforcement, as noted by Tadros, Tromposch, and Mufti (1998), it should be treated as secondary reinforcement only and should not be taken into account in the calculation of the section properties of the beam in the negative moment region.

3.14—Fibers in concrete

The concrete of the slab should contain low-modulus fibers so that the residual strength index R is at least 0.15 ([Appendix C](#)).

The test method for determining the residual strength index is described in [Appendix A](#). Discussion on the significance of this index is provided in [Chapter 4](#).

3.15—Crack control

The slab should contain a mesh or grid of GFRP to control the width and number of cracks.

The phenomenon of cracking in cast-in-place and precast slabs, as related to the reinforcement-free concept described herein is discussed in [Sections 5.4.1](#) through [5.4.4](#). The recommended approach to arrest cracking is presented in [Section 5.4.4](#).

CHAPTER 4—MATERIALS

The design and in-service behavior of cast-in-place and precast reinforcement-free bridge deck slabs does not depend on special or exotic materials. Because the body of these deck slabs can be devoid of steel reinforcement, it is necessary to control the cracks that develop in concrete during its early life due to volumetric changes. These volumetric changes are compensated through the incorporation of low-modulus synthetic fibers.

The fiber volume fraction should be such that the post-cracking residual strength index R of the FRC is at least 0.15. The index R is the ratio of the maximum bending stress that an FRC beam can sustain after initial cracking and the initial cracking stress of the same specimen, so that $R = f_{pcr}/f_{cr}$. Both the cracking and postcracking strengths of FRC, f_{cr} , and f_{pcr} , respectively, are obtained from bending tests. If a beam specimen has no fibers, then R will be zero. When the tensile strength of FRC is greater than that of plain concrete, R is greater than 1.0. The fact that the required minimum value of R is significantly less than 1.0 confirms that the addition of the low-modulus fibers does not increase the tensile strength of the concrete. The addition of high-modulus fibers, such as carbon fibers, does increase the tensile strength of concrete, in which case the value of R is larger than 1.0. Details of the tests for determining the value of R are given in [Appendix A](#). A similar method for determining the residual strength index is specified in ASTM C 1399-98.

Because of its predominantly compressive stress regime, the behavior of a well-confined deck slab is only slightly affected by the presence of large and frequent cracks. Agarwal and Ip (1992) tested a 9 in. (230 mm) thick RC slab with a large number of cracks wider than 0.02 in. (0.5 mm) and having a herring-bone type of pattern. By comparing the results of tests on this slab with those on a similar crack-free deck slab, they concluded that the presence of cracks does not affect the load-carrying capacity of the deck slab. Bakht and Selvadurai (1996) have also confirmed that a 6.9 in. (175 mm) thick, reinforcement-free deck slab without any fibers was able to sustain two million passes of each of 11, 15, 19, and 22 kips (49, 67, 85, and 98 kN) wheels without significant loss of stiffness. The low-modulus fibers in reinforcement-free deck slabs have practically no influence on the strength or stiffness of the slab. Pulsating load tests on two precast deck slabs designed using the concept described in this report have shown that the initiation of cracks in these slabs is related to the fiber volume fraction. Slabs with lower dosages of fibers develop cracks earlier than slabs with higher dosages (Mufti, Banthia, and Bakht 2001). As discussed in [Sections 5.4](#) and